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– Water handling during backfill installation

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Contents

1　Introduction

This memorandum describes different methods for handling the inflow water, assesses the level of the knowledge, evaluates the potential usability of the described methods based on the assumed long term safety impact, and gives recommendations for future testing.

Most of the methods selected for evaluation have been developed under SKB and Posiva assignments by companies such as Clay Technology AB, B+Tech Oy and VTT. One additional methodology was developed by NAGRA in the Euratom FEBEX II project. Methods for water inflow handling were also collected from literature in order to identify those in current use in civil engineering and evaluated as to how they might be applied to disposal facility usage. The methods were evaluated based on their recognized long term safety, effectiveness while the tunnel remains open and maturity of the technology used as well as occupational safety aspects and usability in repository environment.

Methods are divided into five topic areas as follows and presented in more detail in Sections 3 through 7:

- Methods for handling water inflow rates up to around 1.0 l/min are presented in Section 3. These methods deviate only slightly from the basic KBS3-V concepts and are mainly based on optimization of existing methodologies, excluding the geotextiles and removable tube.

- Grouting methods and their limitations in different inflow scenarios are discussed in Section 4.

- Different types of dam construction are evaluated in Section 5. Within a repository dams would be required near high-inflow fracture locations and when tunnel total inflow exceeds approximately 4.0 l/min in the Posiva tunnel concept or 5.5 l/min in the SKB tunnel concept. Dams in this situation are meant to separate high inflow locations from the rest of the tunnel and to allow for storage (and removal) of inflow water.

- Methods for water management in extreme inflow cases are examined in Section 6. Whether these methods are relevant or can be used in such high inflow situations are discussed. Whether such features can be detected and where possible avoided, using piloting/exploratory boreholes is not considered in this discussion.

- Methods used in conventional civil/geological engineering applications are reviewed and discussed in Section 7 with respect to their applicability in a repository application.

As an outcome of the report, recommendations are provided based on the assumed cost, potential long term safety effects and potential benefits of the method for further development and testing actions.

2 Background

Depending on the flow rate, characteristics of the pellet fill and the backfill block-pellet geometry and volumes, the water inflow coming into the tunnel may be stored in the open macro void volume of the pellet fill. The probability of this decreases with increasing water inflow rate and piping of the water inflow through the backfill can be expected to take place in the deposition tunnels. Therefore, water handling methods are expected to be needed independent of the site (Forsmark or Olkiluoto) or the details of the backfill design.

For low, fracture-related, water inflow rates (up to 0.5–1.0 l/min) from a single feature, there exist concepts for handling these without harmful effects such as local erosion or creation of flow paths. However even small inflow features may cause problems for the backfilling of the deposition tunnel if there are many individual inflow fractures.

For inflow rates higher than 1.0 l/min per fracture feature there are no tested methods for water control in a repository environment. However, as mentioned above, there are many ideas on how such a situation could be handled. Long term safety effects of these methods are in some cases uncertain and should be studied in more detail.

The inflow rates, patterns and subsequent need for water inflow management may differ between the SKB facility proposed for Forsmark and Posiva's Onkalo site. As a result the options for water inflow management differ slightly for each location. Sections 2.1 and 2.2 briefly summarize the anticipated inflow rates and distribution within the deposition tunnels at the Forsmark and Onkalo sites respectively.

2.1 Inflow prediction for Forsmark site

The expected inflow to deposition holes and deposition tunnels has been modelled and is reported in Svensson and Follin (2010) and in Börgesson et al. (2015). The basic model used to generate these estimates is the same as used in the SKB SR-Site document (Svensson and Follin 2010) but some minor modifications have been introduced.

The results from the Svensson and Follin (2010) report regarding expected inflow to deposition tunnels (see Figure 2-1), can be summarized as follows:

Out of a total of 207 tunnels,

- approx. 30 tunnels will have a total inflow < 1 l/min,
- approx. 90 tunnels will have a total inflow > 1 l/min,
- approx. 50 tunnels will have a total inflow > 5 l/min,
- approx. 30 tunnels will have a total inflow > 10 l/min,
- only a few tunnels can have a maximum inflow of 50 l/min.

This means that in about 25 % of the tunnels, the inflow will be higher than the maximum limit (5 l/min) previously set in the reference design.

In addition to the inflow rates, the effect of grouting has also been estimated by numerical modelling. The diagram provided in Figure 2-1 shows that the estimated effect of grouting only discernibly changes inflow rates that were initially larger than 1 l/min. Even at very high inflow rate (10 l/min) the flow reductions resulting from grouting are only modest, raising the question of the usefulness of this approach as a routine methodology for controlling water inflow in any but the most extreme situations. However, the results of these predictions are uncertain, since the model used has not been validated.
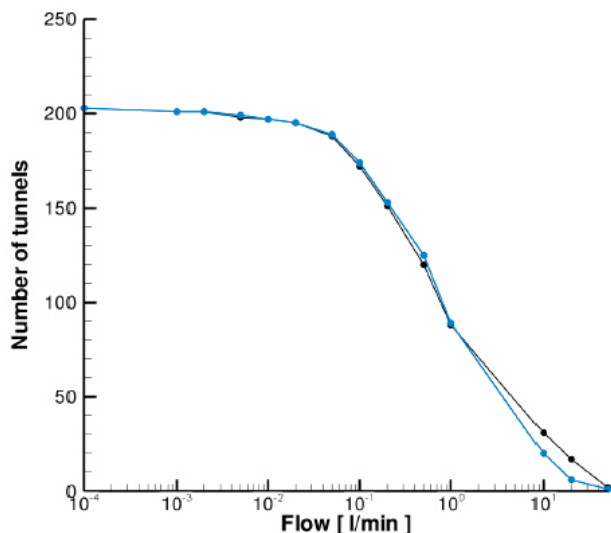
Figure 2-1. *The number of deposition tunnels plotted versus total inflow to the tunnel. Black line represents "No grouting" and blue "Grouted" (Börgersson et al. 2014).*

Other modelling efforts examining the effects of grouting have been done for Forsmark site (Joyce et al. 2013). The results were similar to those presented here, and therefore are not presented in this document.

2.2 Inflow prediction for ONKALO

Two versions of Olkiluoto hydrogeological DFN-model (Discrete Fracture Network) have been produced. The first one has been used to model groundwater flow under open repository conditions (Hartley et al. 2010). The results of that modelling effort include estimates of the inflow to deposition holes, deposition tunnels and tunnel sections. Correlation of the flow observations in the pilot holes, excavated tunnel and deposition holes was addressed. The sensitivity of the modelling results on different model variants, effect of the EDZ, criteria applied for exclusion of deposition holes as well as correlation of the flows under open conditions and in post closure conditions were also studied.

The hydro-DFN model has been updated based on additional data and also to reflect the changes in the geological DFN model and deterministic hydrogeological modelling (Hartley et al. 2013). Although the newer report mainly concentrates on the development of the DFN model, it also includes some flow calculations and discusses the differences to the previous hydrogeological DFN-model and some of the modelling results.

Inflow estimates based on Hydro DFN 2008

Figure 2-2 shows an example of the distribution of inflows to the 9 m deposition tunnel sections (corresponding to a typical distance between the canisters). Although the modelling has been done for 9 m long sections, the rest of the document will use 6 m sections which is the daily length of backfill installation (in SKB concept). Distribution of the inflow to the deposition tunnels based on the results by Hartley et al. (2010) is shown in Figure 2-3. All of these results are calculated without applying any exclusion criteria due to high inflows, additionally the effect of grouting has not been considered.

Further, the following conclusions can be made regarding the anticipated inflow to deposition Tunnels at the Onkalo site:

• 89 % of the deposition tunnels have some detectable inflow, 78 % of the deposition tunnels have inflow greater than 0.01 l/min, 66 % greater than 0.1 l/min; 41 % over 1 l/min and 18 % over 10 l/min (no grouting assumed).

- Most of 9 m deposition tunnel sections are dry, only 18 % of the deposition holes have some inflow.

- The maximum allowed inflow to a tunnel from a single fracture is 0.25 l/min according to the RSC (Rock Suitability Classification). According to the modelling results less than 5 % of the 9 m deposition tunnel sections have inflow higher than the limit (0.3 l/min and assuming no grouting). Such tunnel sections however can be found in approximately 60 % of the tunnels. In many cases the higher inflows seem to be connected to occurrence of larger conductive fractures and fractures around them.

- The distances between the transmissive fractures and inflow points along the tunnel were also analysed. The results suggest a strong clustering of flow conductive fractures with transmissivities less than 10^{-8} m²/s. These clusters are generally swarms with widths estimated to be 10–20 m and the small fractures carry water from nearby large connected open fractures. The spacing between clusters may be large, only 1–2 clusters per tunnel are expected at Onkalo.

- Similar to deposition holes, the presence of an EDZ increases the number of tunnel sections having inflow (24 %), but its effects on the tunnel sections with inflow higher than 0.001 l/min are minor. In this case the EDZ has an effective hydraulic transmissivity of 1 10^{-8} m²/s and a thickness of 0.3 m.
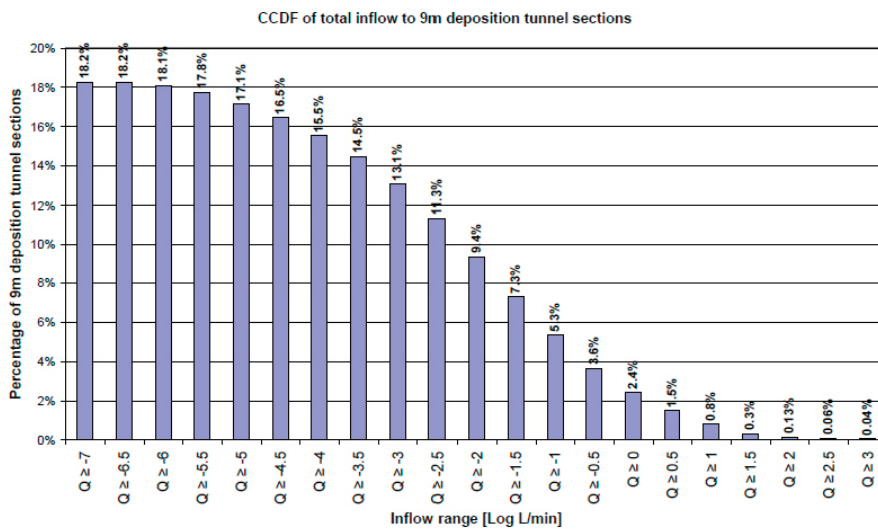
Figure 2-2. *Complementary cumulative distribution of the total inflow to each 9 m section of the deposition tunnel (Hartley et al. 2010).*
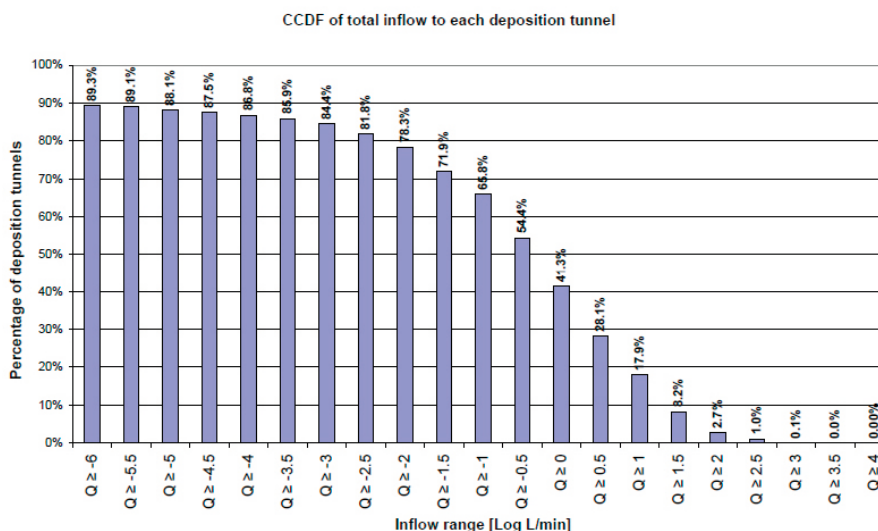
Figure 2-3. *Complementary cumulative distribution of the total inflow to each deposition tunnel (Hartley et al. 2010).*

Based on the inflow modelling reports (Svensson and Follin 2010, Hartley et al. 2010), Posiva's tunnels seem to be dryer than those anticipated in Sweden by SKB. On the other hand, Posiva's tunnel dimensions mean that there is a smaller available void space to store the water entering the excavations. However it should be noted, that both predictions are preliminary and based on modelling with uncertainties.

According to the Svensson and Follin (2010), an estimated 30–40 % of the tunnels at an SKB repository site will need some inflow handling methods to be applied. Tunnels with extreme inflow rates (30–50 l/min) are likely to be encountered (approx. 10 such tunnels based on current statistical assessments) unless those can be detected from probe boreholes and not excavated.

According to the Hartley et al. (2010) an estimated 20–25 % of the tunnels for Posiva's repository will need some inflow water handling methods to be applied. They also estimate only 8 % of the tunnels will have a total inflow rate above 30 l/min.

3 Methods to store and distribute the inflow

Methods for water storage and distribution described in this chapter are based on using existing structures to store the maximum-possible amount of water in the backfill. Available macro void space in the backfill is located in the pellet layer, and is 45–50 % of the volume occupied by the pellet filling. Therefore the macro void volume is ~2.66 m^3/m of tunnel (SKB tunnel) or 1.98 m^3/m of tunnel (Posiva tunnel), based on ideal tunnel excavation dimensions.

3.1 Pellet manufacture and optimization

There are two techniques commonly used in the industry to produce compacted pellets of various materials to be considered for backfill pellets; roller compaction and extrusion. The overall conclusion from the pellet pressing tests is that pellet extrusion is considered to be the preferred method of manufacture, producing a sufficiently robust bentonite pellets for backfill of deposition tunnels. Fine-tuning of the production process, especially identifying the optimal water content, must be performed for every new material (Johnsson and Sandén 2013). This has not been considered as a problem for the materials planned to be used in backfill purposes.

3.1.1 Level of knowledge

Pellet optimization projects have been undertaken by SKB (Johnsson and Sandén 2013) and Posiva (Marjavaara et al. 2013). Optimization undertaken by Posiva has however been done on materials intended for different purposes than deposition tunnel backfilling (buffer gap filling and backfill).

The general conclusion from the laboratory pellet tests (Johnsson and Sandén 2013) is that extruded pellets with a diameter of 6 mm seem to have the best overall characteristics. Regarding materials, Asha NW BFL-L and IBECO RWC BF were superior to MX-80 in backfill pellet manufacturing purposes. The extruded pellets were the most resistant to erosion, but all material and pellet types are within the limits of the theoretical model describing the erosion rate by Sandén and Börgesson (2010). The extruded pellets also resulted in a fill that had higher water flow resistance and seemed to slow the wetting front better than the roller compacted pellets according to the comparisons in the initial water storing tests. Installation tests using shotcrete equipment showed that extruded pellets were also much more durable than compacted pellets (Johnsson and Sandén 2013), which may be caused by the higher water content. In order to have a smooth pellet production and high quality pellets, extruded pellets require 13–20 % water content.

Based on the laboratory tests, the 6 mm extruded pellets were chosen as the preferred pellet type for the large scale test. A large scale test verified that the pellet type has desirable characteristics regarding erosion and water storing capacity (Johnsson and Sandén 2013).

Pellets are relatively well optimized in shape and material. Methods for pellet installation may need some further development, due to extensive dust generation using conventional methodologies.

In the geotextile tests undertaken in 2012 (Koskinen and Sandén 2014), the effect of sieving out the fine particles from pellet products was observed. This fines-removal option should be studied further, since it may provide significant operational benefits for little effort.

3.1.2 Long term safety aspect

Pellet optimization does not have any major effect on long term safety of the disposal facility as long as the installed density is not lowered significantly and the proneness to erosion is taken into account.

3.2 Geotextile

Geotextile is planned to be used in inflow areas. Its purpose is to distribute water inflow more evenly from water bearing fracture zone to the backfilled tunnel, thus storing more water into the pellet section and avoiding local erosion. Geotextile would be used approximately 2 m to both sides of the fracture zone, depending on the inflow rate.

3.2.1 Level of knowledge

Use of geotextile for water distribution purposes has been studied in 2012–2013 (Sandén and Koskinen 2014). According to these tests, the presence of a geotextile layer on the rock-pellet interface at the location of an inflow feature seems to increase the water storing capacity of the pellet filling by at least 30 %, but data is limited. According to the tests so far conducted, the best performance of the geotextile occurs when point-wise water inflow rates are higher than ~0.3 l/min, but these results are not conclusive. Due to the uncertainties associated with results obtained thus far, there is still a need to repeat some tests as well as do tests with higher inflow rates.

According to the laboratory-scale tests done by Clay Technology AB, the specific geotextile material used seems to have only limited impact on how well water is redistributed (Koskinen and Sandén 2014). The results observed seem more a function of creating a preferential flow path between tunnel wall and pellet front. The geotextile then distributes the inflow into larger area and pellets are able to take up water at a gradual pace.

Different methods to fasten the geotextile onto wet and dusty rock surface were tested in 2012–2013 (Koskinen and Sandén 2014). Two types of glue were tested, without success. Glue was also considered to be potentially hazardous to long term safety; therefore no more effort was put into testing this approach. Cement was also tested but it proved to be impractical due to the large quantities required. The best alternative identified was use of express nails. The express nails are hollow nails designed to be used in hard rock or concrete surface, picture of the express nails can be seen in Figure 3-1. The fastening of the geotextile with express nails was quick and easy. The attachment between rock and geotextile was durable. Some additional detailed development for this fastening approach is needed, but it is not considered to be a high priority activity.

Figure 3-1. *Picture of an express nail.*

3.2.2 Long term safety aspect

As long as the geotextile is made out of glass fiber, there should not be material composition issues related to long term safety. Fastening methods using steel or plastics (which can be hazardous in long term safety point of view), should not be an issue since the quantities used are very small.

Because the geotextile acts as preferential flow path, the amount of geotextile must be kept at minimum. Safety distance from geotextile to closest deposition hole must be decided so as to prevent preferential flow paths for contaminants or microbe growth near the canister borehole.

3.3 Temporary drainage pipe

Temporary removable drainage pipe(s) would be used in conjunction with geotextile, to allow for short-term drainage of excess inflow water away from the pellet front. The pipe would be temporarily attached to geotextile. The maximum length of the pipe is as-yet undefined, but probably something in neighbourhood of 10 meters will be the upper limit of length (depending on the backfill installation section length). If the pipe is much longer, the force required to pull out the pipe could increase to values too high for the approach to be practical.

3.3.1 Level of knowledge

Based on the KBS-3H design the removal of long pipes from bentonite filling is possible. The same methods as proposed in KBS-3H can be used in removing the pipe from pellet front.

Geotextile material also needs to be selected correctly so that the inflow water will flow downwards into the pipe. If the geotextile is designed to distribute water, it is possible that the water will not readily flow downwards into the pipe. However the most likely outcome is that the water will flow to the pipe since it is the flow path of least resistance once a saturated wetting front is established in the pellet fill.

The attachment between the pipe and geotextile needs to be carefully designed, it should be strong enough to survive the pellet installation, but weak enough to allow the pipe removal.

The attachment should be studied in more detail. Basically there are three options to make the attachment: (1) selecting the geotextile material correctly, (2) making the attachment so that it will break, and (3) designing the pipe to have a sharp edge that will cut the geotextile. All of the possible methods have some challenges, designing and testing of the removability should be done, if the method will be used. Further development is recommended, since the method is likely to increase significantly the capacity of the geotextile method.

3.3.2 Long term safety aspect

Since the pipe will be removed and the expected diameter of the pipe is only around 20 mm, there should not be any long term safety affects. The only identified risk is that the pipe cannot be removed in one piece. This risk should be minimal as long as pipe strength is overdesigned and material is correctly selected. In order to avoid any long term safety effect should a section become lodged in the clay, the pipe material shall not have potential for substantial adverse reaction with the bentonite.

3.4 Artificial wetting of pellet surface

A method considered, and also tested during pellet installation in the steel tunnel tests (Koskinen and Sandén 2014), was to wet the outermost pellet surface after installation of a backfill section. The pellet installation is made by blowing the pellets into the slots. The main part of pellets in a section is installed without adding any water but during installation of the final pellet layer, water can be added at the nozzle. The idea with this method is that water flowing from the inside of the pellet filling towards the front will hit the wetted pellet "wall" which is much tighter than the rest of the pellet filling, and the water will therefore back up into the dry parts of the filling. With this method a larger

part of the pellet filling will be used for water storing. Results from the tests performed at Äspö HRL show that the wetted "wall" seems to work according to these ideas.

The method has one drawback, there will be entrapped air that will pressurize and have subsequent violent release when the seal breaches. However, this is not necessarily considered to be problematic, as long as there is more than one backfilling section between the breaching one and front of the backfill.

3.4.1 Level of knowledge

Method has been tested in numerous tunnel simulation tests (Dixon et al. 2008a, b, Riikonen 2009). Adding water during blowing of the pellets is common practice when direct wall adhesion is desired. Water addition during spraying also reduces dust generated in pellet installation. Water addition is also common practice in shotcreting of the tunnel walls in civil engineering. Therefore the method is well developed and understood and no more research is needed at present.

3.4.2 Long term safety aspect

Since the only added material is water, and it will not significantly alter the basic design there are no long term safety effects.

4 Slowdown and delaying the inflow of water

Methods discussed in this chapter are based on the desire to slow down the water inflow rate, compartmentalize the local inflow from rest of the backfill and delay the start of the inflow to the pellet-filled volume.

4.1 Pre-grouting

Pre-grouting of water bearing fractures is a well-known method used to decrease water inflow to tunnels. The technique is planned to be used during the future construction of deposition tunnels in the repository at Forsmark. The grouting will probably be made as a selective pre-grouting i.e. the results from analyses of pilot holes will be used in order to decide if grouting should be used. Pre-grouting decreases the water inflow into a tunnel but there is no guarantee that water inflow can be prevented or what degree of improvement can be achieved.

4.1.1 Level of knowledge

Pre-grouting with colloidal silica has been used in Äspö HRL as well as in ONKALO. Although some difficulties have been encountered, the overall performance and installation technique are well known (Funehag 2008, Hollmén et al. 2013).

Due to the materials and geological conditions, pre-grouting is not 100 % effective even in the best of circumstances.

4.1.2 Long term safety aspect

Colloidal silica gel has been considered to be acceptable from the long term safety point of view, since it has low pH and does not react with other EBS components.

4.2 Post-grouting

Post-grouting procedure is similar to pre-grouting, with one major difference, the grouting agent is injected after the excavation is opened. Post-grouting is rarely as effective as the pre-grouting even in optimal conditions. Post-grouting becomes more difficult as the inflow water pressure and inflow rate rises. Due to long term safety requirements regarding materials used underground, the grouting agent is limited to colloidal silica which is less effective than cement based materials. The difference is caused by the properties related to the grouting methods and colloidal silica's tendency to flow out of the fracture more easily than cement materials.

4.2.1 Level of knowledge

This technique has been tested by SKB in a number of different projects e.g. in the KBS-3H project where horizontal TBM-tunnels with a diameter of 1.85 m were successfully post-grouted by use of a Mega-Packer (Eriksson and Lindström 2008). Such equipment is not readily usable for deposition tunnels in the KBS-3V geometry, but the principle might be possible to adapt for use in vertical deposition bore holes.

The use of post-grouting in blasted KBS-3V deposition tunnels is possible, but will, require further development before it can be considered for routine use.

In many cases, following post-grouting, the inflow rate may drop for a brief period but after while returns to the original level (Sievänen and Hagros 2002). This is usually caused by redirection of the flowpaths to previously unused routes. However, the time needed to control the inflow is not very

long, just enough to enable installation of the backfill. With high inflow water pressure, such as in deposition level, the inflow of water tends to remain constant even if post-grouting is attempted.

4.2.2 Long term safety aspect

Use of cementitious materials for post-grouting has been considered to be too harmful for long term safety, and therefore it is not recommended. However, if post-grouting with colloidal silica can be developed into a reliable technology, there should not be any adverse long term effects. If not using grouting leads to need for a permanent concrete based dam, it should be estimated whether the amount of cement used for post grouting would be smaller compared to the concrete plug (see Section 5). The alternative with less foreign materials to the repository should be preferred.

5 Dam construction

All deposition tunnels are planned to be closed with a concrete plug. The demands on these plugs will be very high e.g. they should withstand high pressures (water pressure and swelling pressure from the backfill), they should be water tight and they should have a lifetime of more than one hundred years. It is of course also possible to build this kind of plug inside the deposition tunnels in order to cut off a water bearing fracture zone but this will be very expensive and time consuming. The proposals in this chapter assume that no deposition tunnels will be abandoned and so temporary plugs will be needed.

Since the demands on temporary plugs positioned in deposition drifts are lower (only during operations within that tunnel), it is possible to instead of using the standard tunnel end plug design, build simplified plugs with a design adapted for the actual conditions. Such plugs could also be used in cases of unscheduled operational stoppage during the installation phase. A suggestion for a plug classification is provided in Table 5-1.

In this chapter, a number of different plug designs are suggested. None of the designs are, however, investigated in detail and they should therefore be considered as conceptual regarding both design and requirements.

Table 5-1. Suggestion for different plug classes for discussion purposes only (Sandén and Börgesson 2014).

Plug class	Planned/Unplanned	Life time	Strength requirements	Design
Class I	Planned	>100 years	10 MPa (Swelling pressure + full hydrostatic pressure)	Concrete plug with bentonite sealing and drainage layers
Class II	Planned	Approx. 1 year	6 MPa (Low swelling pressure + full hydrostatic pressure)	Concrete plug with drainage layer
Class III	Unplanned	Approx. 2 months	0.5 MPa (Low swelling pressure)	Concrete beams + frame work of steel, drainage layer
Class IV	Unplanned	Approx. 2 weeks	50 kPa Earth pressure from gravel 8/16 mm and pellets	Concrete beams, drainage layer

5.1 Tunnel end plug (Class I and Class II)

SKB (SKB 2010) and Posiva (Keto et al. 2013) have developed their own tunnel end plug designs. Although there are differences in these designs, both are expected to be able to withstand 10 MPa swelling pressure and their design lifespan is upward of 100 years.

The demands on a tunnel end plugs (Class I) are very high. They should withstand high pressures (water pressure and swelling pressure from the backfill), they should be water tight and they should have a functional lifetime of more than hundred years. The conceptual design of such a plug is shown in Figure 5-1. The properties and requirements of this plug type are investigated in other projects and so will not be further discussed in this report.

If a deposition drift crosses a fracture zone with high water inflow rates it could be necessary to build a plug in order to cut off the fracture zone. Such a plug, Class II, can be of a more simple construction than the tunnel end plug, and the demands regarding life-time and strength can be lower. The design could be similar to a Class I plug, but some of the parts can be excluded, such as the bentonite sealing blocks and extra drainage sections. A disadvantage is that this type of plug requires a lot of preparation work on the rock, and this must of course have been made before the backfilling of the deposition tunnel starts.
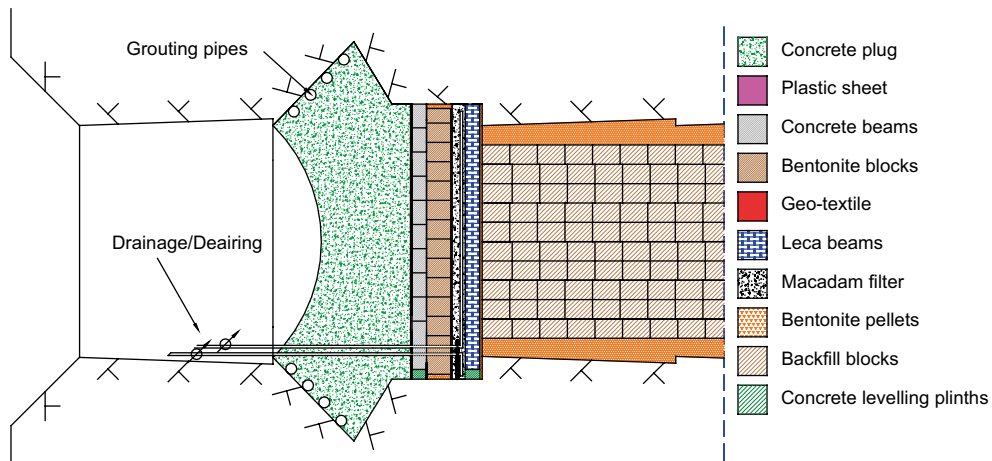
Figure 5-1. *Schematic drawing of a tunnel end plug (class I).*

5.1.1 Level of knowledge

SKB has constructed a prototype tunnel end plug and Posiva is planning to construct another in first half of 2015. The know-how level related to construction and installation of the tunnel end plug is relatively high, and there is confidence that the same design could be used to isolate a high inflow section of deposition tunnel. However, the design is too complicated, expensive and time consuming to build for it to be viable as a standard option for water handling.

5.1.2 Long term safety aspect

Materials and design are already accepted for use in deposition tunnels. Therefore no major unresolved long term safety aspects can be expected. However, the amount of extra cement and other foreign materials used in permanent concrete dams needs to be taken into account and a method not requiring large amount of concrete should be preferred.

5.1.3 Light tunnel end plug (Class II)

Basically the light tunnel end plug (Class II) is similar in design to the tunnel end plug (Class I), however some less-vital components intended to provide for long term mechanical stability have been omitted. This will mean that it would be a little bit quicker to construct and also less expensive.

The know-how level regarding this component is good since the tunnel end plugs have been tested. Testing results from tunnel end plugs can be applied to this lighter version as well. It is anticipated that should they become necessary to install, this type of plug would remain in the tunnel and back-filling operations would restart on the downstream side of the plug (Sandén and Börgesson 2014).

5.2 Light concrete plug (Class III and Class IV)

In some situations it could be necessary to build a plug within the deposition drift on short notice. One example of such a situation occurs when the backfill installation process cannot proceed according to the plans and an extended interruption occurs, e.g. mechanical failure of the backfill installation equipment or if the backfill block production is insufficient for operations. In order to not risk the previously installed backfill being so affected by inflowing water that its functional requirements are not fulfilled, it could be necessary to build a plug within a few days (Class IV). If it is foreseen that the installation of backfill cannot be continued within a time of maximum two weeks, it will probably be necessary to strengthen the construction in order to secure the plug for a time of a couple of months (Class III) (Sandén and Börgesson 2014).

5.2.1 Level of knowledge

Examples of design of plugs of class IV are provided in Figure 5-2 and 5-3. The main plug consists of large concrete beams. The beams are anchored to the rock by reinforcement bars cemented in boreholes in the rock and then welded to steel plates cast in advance into the beams. Inside the beams there is a drainage layer of gravel 8/16 mm. A steel tube should be installed in the bottom of the plug, running through the beams and into the drainage layer. This design makes it possible to drain water from the inside of the plug and prevent high water pressures from building up. This type of construction has already been built as a part of the Plug project, see photo provided in Figure 5-3 (Sandén and Börgesson 2014). It is judged that this type of plug can be built within a few days. When a decision to continue the backfilling has been taken, the valve on the drainage tube should be closed. The design means that a quantity of steel and concrete will be left in the tunnel.
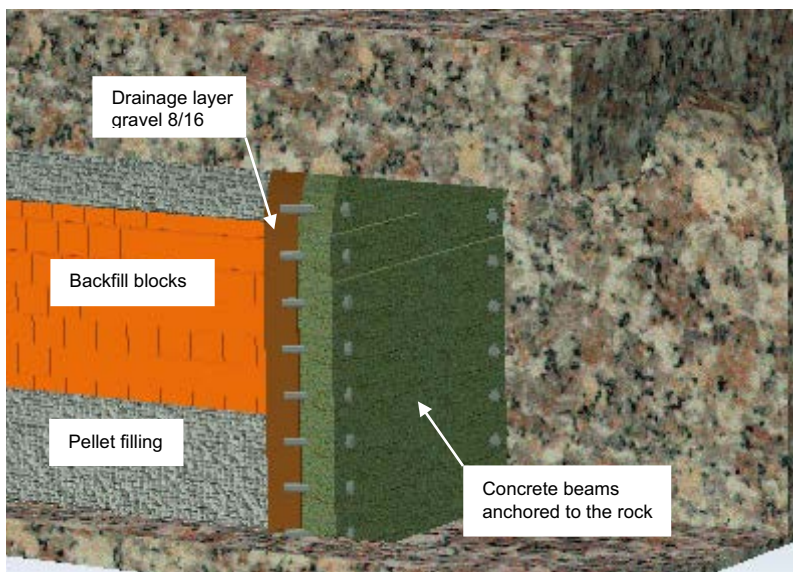

Figure 5-2. Suggestion for design of a class IV plug (Sandén and Börgesson 2014).

Figure 5-3. Photo taken during the installation of the plug test at Äspö. The photo shows the concrete beams outside the bentonite sealing (Sandén and Börgesson 2014).

The demands regarding strength are rather low on this kind of plug. The plug should withstand the earth pressure from the gravel 8/16 mm (commercially available gravel with grain size distribution between 8 and 16 mm) and the pellet filling inside the tunnel i.e. approx. 50 kPa in total. There may also be a very limited swelling force developed by the backfill in the tunnel that must be supported.

There is a need to investigate what happens when the hydraulic and swelling pressure exceeds the strength of the temporary plug. This effect and time window needs to be compared to backfill installation process. It is important to show that the backfill has progressed far enough in the tunnel to when temporary plug's strength is exceeded so that there are no risk for negative effects on the backfill operation or the initial state.

5.2.2 Long term safety aspect

Use of the gravel 8/16 mm, steel and concrete lowers the average backfill density achieved in the tunnel section being isolated and therefore might increase the saturated and density equilibrated water conductivity of the backfill. The magnitude of the effect is dependent on the length of the isolated section and if it is kept short the effects may be minimal. Alternatively, if the isolated section is short the benefit of installing the isolating plug is questionable.

Calculations and possibly some simulations are needed to determine ideal length of the isolated section in order to avoid inducing unacceptable long term conditions. The effects of the ideal length isolation in specific inflow rate on to the long term safety can then be estimated. The intended minimum distance between a plug and the closest deposition holes is the most important parameter in the estimation.

5.2.3 Fortified light concrete plug (Class IV)

If the backfilling process cannot restart within a couple of weeks of its interruption, there may be a substantial swelling pressure from the backfill inside the plug. This will make it necessary to strengthen the plug. This can e.g. be made by mounting of a steel frame outside the plug (Figure 5-4). The frame can be welded together on site and then anchored to the rock. How this type of plug can be designed to take a reasonable high load needs to be investigated more thoroughly.

5.2.4 Long term safety aspect (Class IV)

Similar to the Class III type concrete plug, except the amount of the steel is higher.
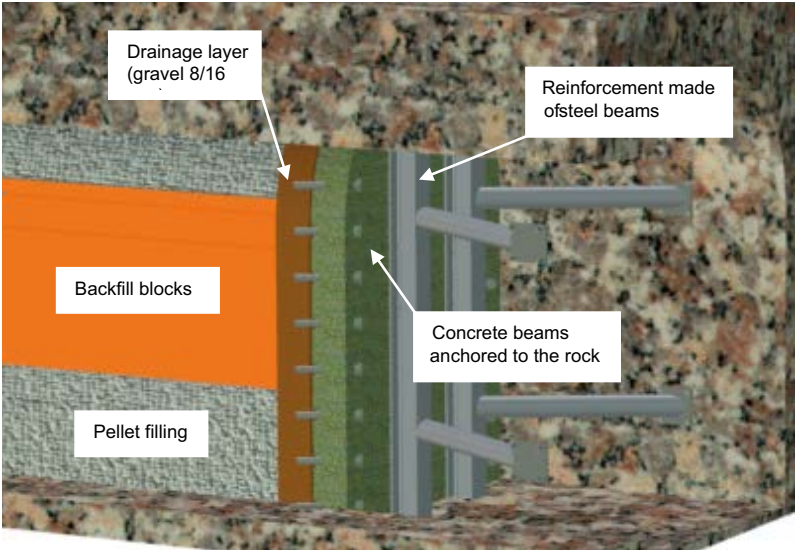

Figure 5-4. *Suggestion for design of a class III plug (Sandén and Börgesson 2014).*

5.3 Shotcrete plug (FEBEX II)

Shotcrete plug construction was developed by Nagra and tested in FEBEX II project. In this plug concept the shotcrete is sprayed directly to bentonite backfill. The thickness of the FEBEX II plug was 2.5 m, however it was designed for much greater mechanical stress than is predicted in water handling situation. FEBEX II plug also used steel fiber reinforcement, which needs to be assessed regarding the long term safety aspect. In general shotcrete plugs can be designed to work as dome plugs where the force is mainly taken by compression forces in the concrete. This results in a rather long plug and specific high tolerance requirements on the concrete and rock interface. For other types of shotcrete plugs used generally the force is taken by the friction between rock and concrete interface and the plug also needs to endure tensile forces.

The construction would be fast, and require only minimal advance preparation. Therefore it would be suitable method to create short term operationally-expedient plugs in the tunnel in case of machine failure or some other unseen delay in the emplacement of the backfill.

Method development is needed if a drainage layer is needed between the backfill and plug. In that case some metal (or other) mesh should be attached between the gravel 8/16 mm and plug in order to have confined space for the gravel.

5.3.1 Level of knowledge

Basic concept has been successfully tested in Grimsel URL as part of the FEBEX II project. A picture from the Grimsel test can be seen in Figure 5-5. Detail planning is needed to determine if the metal mesh is needed, and the ideal thickness of the plug.

5.3.2 Long term safety

If fiber reinforcement is required in the shotcrete, the material of the fiber can be glass fiber. Glass fiber has been considered to have only small if any impact to the long term safety.

Other possible materials that could be used in this type of plug are low-pH shotcrete rather than conventional formulations and the metal mesh to separate the gravel 8/16 mm drainage layer. Materials ultimately selected for use should not cause any significant effect on long term safety, as long as each of the materials chosen are carefully evaluated and confirmed as suitable for use.
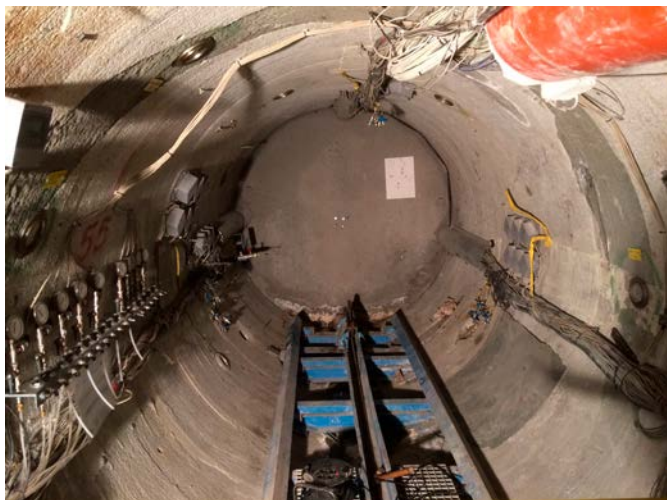

Figure 5-5. Picture of the finished FEBEX II shotcrete plug (Nagra).

5.4 Bentonite/copper plate dam (B+TECH design)

In this concept a flow retaining dam wall is constructed using a copper sheet (see Figure 5-6). The copper wall prevents water from flowing through the foundation layer towards the tunnel mouth and the section under ongoing backfilling.

The anticipated thickness of the copper sheet is three millimetres. It is attached to fixing points that are fastened on the rock surface. The fixing points are attached to a smooth rock surface that has been prepared by drilling, wire sawing or small amount of low pH cement mortar. The nominal height of the wall is 3 m but it can be adjusted as required. The wall is dimensioned so that it can be installed efficiently with the forces from possible water pressure and swelling in isolated region carried by an adjacent supporting dam section. As an option, if drainage is required a water drainage pipe can be positioned at the bottom of the copper sheet if drainage is required during installation of the innermost portion of the seal and the dam wall. The concept was designed at B+tech on request from Posiva in 2011 but is not published.

However, since the copper plate does not go all the way to the sealing of the tunnel, the method is likely not to function as designed. The Dam would require constant drainage which is not practical compared to the concrete based plug presented above.

5.4.1 Level of knowledge

This design is only at a conceptual design state and needs to be further developed and tested if it is considered for use in deposition tunnels. In order to be a viable option with respect to the concrete plugs (Class III and IV), the upper part of the dam needs to be machined to fit perfectly to the sealing of the tunnel. This would cause additional expenses, work, and delay the backfill emplacement process although major part of the work can be done in advance.

Additional mechanical durability can be achieved by a using temporary steel plate and supporting beams. These could subsequently be removed prior to restarting the backfilling process.

5.4.2 Long term safety aspect

Dam construction should not have major long term safety effects, since all the materials are also used in other KBS-3 sealing components. If the dam is constructed out of MX80 clay the average EMDD (Effective Montmorillonite Dry Density) tends to be higher at least locally meaning that there should be a lower hydraulic conductivity and higher swelling pressure in this location.
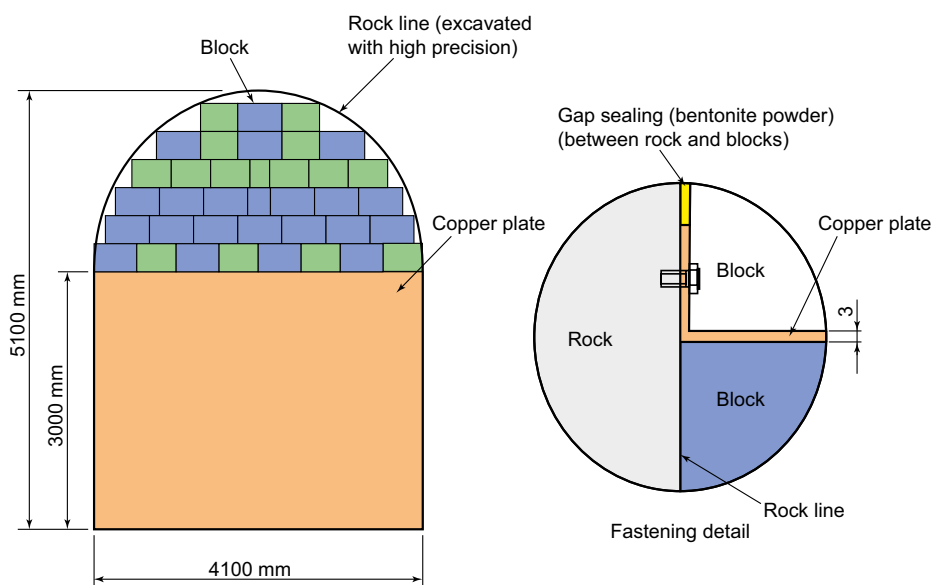

Figure 5-6. Schematic picture of the copper wall dam.

5.5 Bentonite dam

The tunnel zone where the bentonite dam would need to be built must be ground or wire-sawed so that the surface tolerance is less than 20 mm. In order to be able to grind the surfaces smooth, the construction site must be in unfractured rock. The length of the bentonite dam must be at least 2 m, but portioned to the adjacent gravel filled section.

The smoothed tunnel zone is then filled with MX80 or other high quality bentonite blocks. The blocks used in this will need to be machined in order to have as small gap as possible between the blocks and rock.

After the installation of the blocks, artificial wetting is needed in order to quickly create a tight fit between the blocks and rock. The inflow water will then increase the swelling pressure by further hydrating the bentonite and therefore create an effective dam.

This type of dam is expected to remain mechanically intact and water tight for at least two weeks, but may need to be functional for up to a couple of months, but these estimates need to be confirmed. If so desired, supporting metal mesh can be anchored into rock to increase mechanical durability.

This concept is relatively complex, requiring trimming of the blocks as well as a tight fit of the block-filled volume within the tunnel. As a result, this option may be an expensive alternative for dam construction concepts. The trimming of the blocks and grinding of the rock can be done in advance, and hence it is not likely to significantly slow down the installation process once that stage of operations begins.

5.5.1 Level of knowledge

All the individual parts and methods are tested and well known. However, the functionality as system is not known. Also the friction between the blocks is not known and must be tested. The friction between the blocks is important in order to ensure the stability of the stacked blocks, since the initial water uptake of the blocks is uneven causing differences to the early behaviour of the blocks.

5.5.2 Long term safety

Since this method of tunnel damming contains very little deviation from the basic backfill concept, there are no new major long term safety concerns that have been identified. Metal mesh could potentially have a minor impact on the long term behaviour locally, but its use only an option and may not be needed.

5.6 Gravel 8/16 mm filled section

A Gravel (8/16 mm) filled section is designed to be used in conjunction with dam construction in order to create additional void space to store more water. The gavel filled section has around 45–50 % of void space, none of which will be occluded by swelling clay. The idea is that in case of high inflow rates, the water could be stored in the gravel filled section.

In some cases even in the tunnels where the inflow is within RSC (Rock Suitability Classification) acceptance criteria, small gravel filled sections may be required to avoid problems caused by water exiting the backfill. If gravel filled sections are needed to create more void space for the water storage, it may be beneficial to distribute those sections evenly along the whole tunnel (if this can be accepted in long term safety perspective).

For example in ideally distributed inflow of 0.1 l/min to every 6 m section throughout the 300 m tunnel, a need of 6 m long gravel filled section is required to have additional space for the inflow water. Other inflow cases may even require more gravel filled space to have enough space for the inflow water.

In order to minimize the effects on the EMDD, the backfill material near the gravel filled section should be MX80, or similar high quality bentonite, thereby providing a high-smectite material to ultimately swell into the voids between the aggregate particles.

The gravel-filled section would be isolated from the regular backfill by some kind of dam from both ends. Possible dam constructions are described in Sections 5.1-.5.5.

5.6.1 Level of knowledge

This method is designed only at the conceptual level. Similar gravel type to that proposed in this option will be used in the drainage layer in the tunnel end plug.

5.6.2 Long term safety aspect

A crucial question is "what is the minimum distance between the gravel filled section and nearest disposal hole?" Even with high quality bentonite to compensate the lower EMDD, the gravel filled section could be flow path as long as the high quality bentonite has not intruded into the gravel and a substantial hydraulic gradient exists to drive water movement. Over time it is anticipated that the bentonite will swell and intrude into the gravel but this would be long after the gravel's drainage function would have been completed. However, a composite materials with gravel and bentonite, there is a risk of separation of the material and remaining of permanent water conducting pathways into this section.

In addition to the minimum distance between the gravel filled section and nearest disposal hole, the maximum length possible for a gravel-filled section may play a significant role. Since the bentonite and gravel needs to homogenize with respect to water conductivity though bentonite intrusion into the pore spaces of the gravel before the backfill can be considered to be in a saturated, homogeneous condition, there is a maximum length to the gravel filled section. Even a one meter thick gravel section is likely to take at least hundreds of years for bentonite to swell into the voids between aggregate particles.

No new material types are included in this approach and so there should be no new issues with regards to long term safety from chemical point of view. However it would still be new system in the backfill, and alter the local water conductivity, so there may be undesired long term safety effects.

6   Drainage of the inflow water

Methods mentioned in this chapter require permanent structures or equipment to be left into the deposition tunnels.

6.1   Drainage hole to neighbouring tunnel

A technique assessed to have very high potential in order to handle high water inflows is to drill a borehole from one water bearing fracture zone in a deposition tunnel to an adjacent tunnel. In combination with a section filled with gravel (8/16 mm), see Section 5.5, which collects the water and leads it into the borehole, it would be possible to handle very high water inflows. After finishing the backfill installation in the tunnel and the building of a drift end plug, the borehole must however be sealed. The technique for sealing of boreholes has been developed within other SKB and Posiva projects for sealing instrumentation holes. The method works fine for short tests, but is not designed or demonstrated be a long term solution.

In Posiva's case, the minimum horizontal distance between deposition tunnels is 18 meters (Posiva 2013). Therefore, it is possible to have the same water bearing fracture zone cutting through more than one deposition tunnel. This may create more challenges to the drainage of the inflow to the neighbouring tunnel. It will also require further evaluation with regards to how artificial horizontal connections would influence the safety case (see Section 6.1.2). Also the assumption that water would be steered from the gravel filled zone into the borehole (and not along the pellet filled zone of the backfill) would need to be tested.

6.1.1   Level of knowledge

Techniques to plug boreholes have been developed in previous projects in ONKALO and Äspö HRL. It is well known method and should only require modest development to be applicable.

6.1.2   Long term safety aspect

The drainage hole would be a direct flow path between two tunnels. There are no-such drainage holes taken into account in the KBS-3 safety cases completed to date so it would require new calculations and modelling.

Depending on the materials used in sealing of the drainage hole, there may be a potential to add materials that will have some adverse long term safety effects. Concrete should be avoided in the sealing, since it will (is assumed to) degrade away with time. The best sealing material in long term safety aspect would be highly compacted bentonite such as MX80. Different techniques sealing of investigation bore holes have been developed and tested (Pusch and Ramqvist 2007).

A flow path between tunnels may provide a route for transport of possibly hazardous chemicals to both of the tunnels. Without the flow path provided by the drainage borehole, such chemicals would affect only single tunnel. The presence of a hazardous chemical in one tunnel is very unlikely, but is a possible case should there be a fluid spill from a vehicle.

Although there are many open questions concerning the drainage hole between two tunnels, there is no definitive reason why it could not be constructed without excessive adverse long term safety problems if the closure of the drainage holes can be accomplished. Therefore, a more detailed construction plan that includes these features should be developed so that the long term safety aspects can be studied.

6.2   Drainage tube along the tunnel

Installation of drainage tube along the tunnel wall was considered for use in conjunction with geotextiles. Tubes were planned to be installed into the backfill at the floor level. It was assumed

in this design that the tube material would corrode or break under the backfill swelling pressure. Materials to be used could be glass or thin steel pipe. Another alternative would be to use temporary piping that will be removed from the pellet fill along the backfilling front. This could be applied for a shorter tunnel section at a time, removal from the total length would be difficult.

6.2.1 Level of knowledge

Conceptual design using this approach has been done. There are also measurements of the breaking point of glass pipes. However, the development was stopped when potentially adverse long term effects were identified. The alternative with temporary piping would mean that the pipe material would need to be very robust enabling removal of the pipe.

6.2.2 Long term safety aspect

It was suggested that even with disintegrating/broken glass pipes there would be a potential preferential flow path remaining along the tunnel length as the result of the coarse grained glass fragments. Therefore the presence of a non-removable drainage tube along the tunnel is deemed to be too hazardous with regards to long term safety. Use of temporary pipes would not any significant long term effect, assuming that the pipe can be successfully removed from the backfill.

7 Methods used in civil engineering

7.1 Ground freezing

The method is commonly used in civil engineering when building in sand or other loose wet soils where there are strength or hydraulic issues. The soil is frozen around the construction site, so that the water is isolated from the site. In some cases the method has been applied in tunnels when it has been more practical. During construction, water sealing and wall fortification is done and when the soil water melts the tunnel or shaft is water tight.

It is unclear whether the method has been applied to crystalline rock, but if the method can be adapted to work in deposition tunnel conditions the potential is very high. The method could potentially stop the inflow for backfill installation time, without living any changes to the rock after the rock has melted. However, there are also risks to scatter the rock as well as for occupational safety. Due to the high potential the method is worth investigating more in order to rule out or confirm the risks.

This method has also been used to isolate radioactive contaminated sites such as in Oak Ridge USA. However, the Oak Ridge site is not comparable to the deposition tunnel since the freezing was done from surface. The active freezing was maintained for 6 ½ years. After around 2 years the frozen barrier was 7.6 m thick. After the refrigeration was discontinued, the complete thawing of the soil took months to complete (Phillips and Yarmak 2014).

7.1.1 Level of knowledge

The method is widely used and common knowledge in civil engineering. However, it is not known if it has been applied to crystalline rock or to deposition depths. It is also unclear what kind of limits would be caused by the confined tunnel space with limited ventilation.

In an application such as a repository where ground freezing would be done on a very localized basis (only where extreme inflow is occurring), it is also uncertain if it will prove to be particularly effective. Even if the feature were effectively frozen, the effects will be local and water could well find a new, previously un-utilized pathway into the tunnel (the same issue as often found with post-grouting).

A further potential issue would be the presence of an extensive array of boreholes needed to provide access to the fracture feature. On completion of the freezing process these would need to be remediated/sealed.

The cooling of the freezing agent should be done above ground and the cold fluid is then pumped down to the deposition level. This causes major challenges to construct and dismantle several kilometres of heat insulated pipes. The energy requirement to adequately freeze the amount of rock associated with a repository inflow feature is relatively low, assuming the heat leakage is not too high due to the long transfer pipes.

Depending on the freezing agent used, it may cause occupational hazards in case of leakage. However, the method has been successfully used in underground civil engineering, so the occupational hazards should not be overwhelming. It might also cause chemical effects on the materials it is in contact with should it escape. At room temperature and atmospheric pressure most of the commonly used freezing agents are volatile, displacing air in confined spaces such as tunnels.

7.1.2 Long term safety aspect

Long term safety effects are associated with coolant circulation holes and possible freezing induced rock cracking. The freezing-induced damage to the rock is unknown, however the effects would not extend more than few meters from the deposition tunnel. This could still mean that a much more extensive EDZ is generated at the location of the freezing, potentially affecting the subsequent mass transport characteristics at that location. This needs to be evaluated to determine if localized changes in EDZ (and transport characteristics) will alter the results of the safety case. In addition, the long term safety effect of the composition of the freezing agent should be evaluated as well as the effect of freezing for the engineered barriers in the vicinity of the frozen zone.

7.2 Shotcreting and tunnel drainage

In this approach, shotcrete or other kinds of water sealing material is applied to the rock surface. Between the water sealing compound and the rock wall there are small pipes to drain the water out of the wall into a collection drain (Maidl et al. 2014), although the pipes can be replaced by some other water conducting material such as geotextile. This method is commonly used in road and railway underground tunnel construction around the world. It has been used in the construction of LILW repositories in Finland. However, the containment systems needed in the LILW repositories are designed for much shorter time periods, and requirements are not as strict as in spent nuclear fuel (SNF) disposal.

Method is used to handle relatively high inflows for long periods of time, but due to the long term effects to the flow paths as well as cement-bentonite interactions no large scale usage of the method cannot be encouraged in the deposition tunnels.

7.2.1 Long term safety aspect

This method for water control will introduce materials that are potentially hazardous to the long term safety (cement, pipes). There will also be a flow path in the tunnel since the pipes cannot be removed. Depending on the quantity of cement involved and lack of ability to removal of the drainage pipes, this potential means of accomplishing tunnel drainage may not be viable.

7.3 Water sealing of the tunnel

Water sealing is also widely used method where the rock surface is coated with some water sealing material. Typically the material is plastic, bitumen or shotcrete (Maidl et al. 2014). However, the method is not suitable to high inflow pressures expected at the deposition level. It is also likely that the inflow would occur elsewhere, if the water sealing is done only selectively to high inflow areas.

7.3.1 Long term safety aspect

This approach has numerous issues with respect to its effect on long term safety. As noted above, most applications referenced in the literature or available use materials that are excluded from a repository, making it impractical to consider this approach. In long term, the method would also leave a permanent water conducting pathway in the location of the shotcreted section after dissolution of cement from the concrete. The aggregate material remaining would most likely have higher k-value that is accepted for backfilled tunnels, even though some mixing would take place between the swelling clay used for backfilling and the aggregate.

8 Conclusions

Methods described in this document can provide a technology toolbox to use in developing a means to handle the predicted inflow rates. The existing and demonstrated methods to distribute and store water in existing structures can handle fracture wise inflow up to 1.0 l/min under normal backfilling rates (installation time schedule is around 60 d per 300 m for SKB). However, the void space present in the backfill volume is not adequate if the inflow rate is in the order 5 l/min or operations are interrupted for an extended period. From a backfill installation point of view, it would be beneficial if it could be installed without interruption. However, this may be difficult or impossible if the buffer water protection is not adequate for installing all of the canisters and buffers before backfilling. In any case, it is good to have a water control system available during backfilling independent of the predicted water inflow behaviour. This is because, channel formation in the backfill may in any case lead to faster water outbreak than the backfill rate.

Only preliminary estimations can be given for defining the inflow rate limits associated with specific inflow handling methods. These estimations are provided in Table 8-1. Further testing and field trials are needed in order to get more accurate limits for applicability.

Table 8-1. Estimated inflow rate limits for different handling methods.

Method	Tunnel (300 m) inflow [l/min]	Tunnel section (6 m-long) inflow limits where approach is predicted to work [l/min]
Pellet optimization	All	All
Geotextile		$Q \geq 0.3$
Temporary drainage pipe		$1.0 \leq Q \leq 2.0$
Artificial wetting	All	All
Pregrouting		$Q \geq 0.1$
Post grouting	Can it be functional?	
Tunnel end plug	Only at the deposition tunnel end	
Tunnel end plug light	Unpractical	
Light concrete plug	$Q \geq 4.0$	$1.5 \leq Q \leq 4.0$
Fortified light concrete plug	$Q \geq 4.0$	$1.5 \leq Q \leq 4.0$
Shotcrete plug	$Q \geq 4.0$	$1.5 \leq Q \leq 4.0$
Copper dam	Unpractical	
Bentonite dam	$Q \geq 4.0$	$1.5 \leq Q \leq 4.0$
Gravel 8/16 mm filled section	$Q \geq 4.0$	$2.0 \leq Q \leq 4.0$
Drainage hole to neighboring tunnel		$Q \geq 4.0$

With inflow rates higher than 1.0 l/min for a 6-m section, some short of dam construction is needed in conjunction with a gravel filled section. Upper inflow limits that will trigger their construction have not yet been assessed, but should be directly proportional to the length of the gravel filled section. The gravel filled section may also be required if the available void space in the pellet front is not sufficient to store the inflow water. Also, the long term safety aspects of the gravel filled section needs to be carefully assessed.

In extreme inflow cases the installation of a drainage hole to a neighbouring tunnel may be the only viable solution if the tunnel in question is to be used. Such an approach has many potential issues with respect to long term safety, each of which needs to be assessed before this approach can be

accepted for use. It should be noted that the best option would be for such extreme inflow features to be detected before excavation through use of lookout holes, so the tunnel would not be excavated.

Temporary drainage pipes along the tunnel would be viable option from long term safety point of view, but would require further testing.

From conventional civil engineering tunnel construction methodologies, no suitable methods were identified that could be used in the repository due to adverse long term safety aspects. Also the ground freezing was deemed to be too complicated and have too high occupational and potential long term safety hazards to be a viable option in deposition tunnels.

9 Recommendations

Due to limited resources all potential approaches cannot be studied at once, therefore following list is organized in a suggested priority order. Priority is determined based on potential benefit against estimated cost. The estimation is done solely by the author.

1. Effect of the sieved bentonite pellets on water retention. A test can be made at laboratory scale as a "slot test". In 2012 geotextile tests (Koskinen and Sandén 2014), an effect of sieving out the small particles was noted. The limited number of tests undertaken in that study did not however allow the manner in which granularity effects water retention to be clearly quantified. More testing is needed.

2. Geotextile tests. One or two of the tests done in 2012 (Koskinen and Sandén 2014) in the steel tunnel need to be repeated. Additionally, one test with higher inflow rate (possibly 1.0 l/min) would be beneficial in order to identify the upper limit of water inflow that can be accommodated using this method.

3. In conjunction with the geotextile steel tunnel tests, the temporary drainage tube concept for short-term water management should be tested. Also, the removal process for the temporary drainage tube should be tested. Removal demonstration can be done after the completion of the water outflow management test, when it does not affect the results.

4. Design of the concept of using a gravel filled section as a water retention structure should be further developed, and long term safety limitations should be assessed. Possibly some intermediate- or large-scale (e.g. steel tunnel) laboratory tests can be done, if it is required to verify some of the assumptions made in developing this concept.

5. The use of a bentonite dam to restrain water movement out of the tunnel should be tested, at least at small scale. A testing rig could be built to withstand the higher pressures needed for the feed water, as well as for withstand the swelling pressure. Testing rig needs to be relatively small, so that it would be lighter while able to withstand the pressures. Such a test chamber could be in the order of $70 \times 70 \times 70$ cm with a separate gravel-filled section and water inlet pipe. An example of such a test setup picture can be seen in Figure 9-1.

6. The idea of installing a drainage borehole to a neighbouring tunnel should be investigated further, including a thorough long term safety assessment done on such a system. Design evaluation should include determining the minimum acceptable distance of such a drain from the deposition hole as well as developing a detailed plan of how to close the drainage hole.
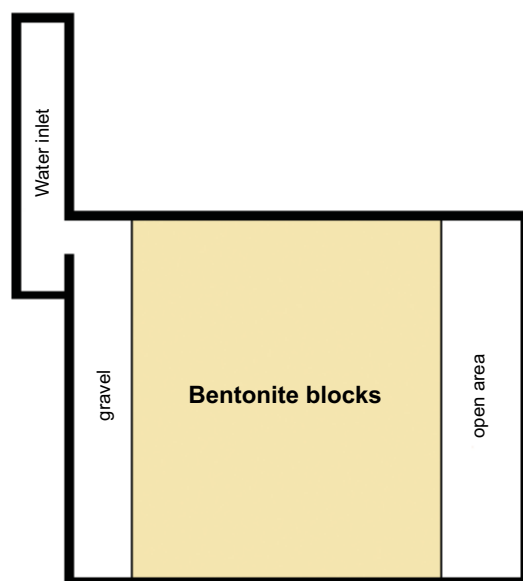

Figure 9-1. *Bentonite dam testing rig, schematic picture.*

7. More detailed design of the temporary concrete dam, as well as the shotcrete dam, should be completed. How these types of technical solutions can be designed to take a reasonable high load needs to be investigated.

8. Full scale test of an inflow management design in an actual tunnel should be undertaken. A system consisting of: blocks, pellets (Sieved), geotextile, gravel filled section and some type of dam construction should be installed. Since there are multiple components that would be investigated in such a construction, the minimum length of tunnel needed for such demonstration would be around 10 m.

The remaining techniques presented in this memo are either well known and demonstrated or are considered not readily viable and so not in need of further developmental effort at this time.

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SKB is tasked with managing Swedish nuclear and radioactive waste in a safe way.

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