The role of crystalline rock for disposal of high-level radioactive waste (HLW)

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Abstract

The stress conditions in the host rock of HLW repositories, altered by construction and heat generated by the waste, can cause large changes in groundwater flow, and failure of the repository. These shortcomings reduces the barrier role of the rock and the required effective isolation of HLW has to be provided by engineered barriers of which clay-embedded metal canisters are most important. The paper describes the performance of the nearfield rock of HLW packages according to two types of repositories: very long subhorizontal holes with large diameter (1.9 m) bored or blasted at 400-600 m depth, and bored 4 km deep holes with clay/concrete seals in the upper 2 km parts and HLW packages separated by clay/concrete seals in the lower 2 km parts. The most important difference between the two concepts is the much higher groundwater transmissivity of the rock in the shallow repository. A common feature is the need for rock support by filling the space between waste containers and rock with clay mud after installing the containers in shallow repositories, and by keeping the holes in deep repositories filled with clay mud from the start of boring the holes and throughout the installation of clay/concrete seals. Focus is on the role of rock discontinuities on the hydraulic and mechanical performances of the repository host rock at construction and under seismic and tectonic impact. Comparison of the two concepts shows that the one implying deep-disposal is superior but requires preparative steps in the waste placement phase.

Key words: hydraulic conductivity, rock, strain, stress, structure

1. Scope

The host rock of a repository for highly radioactive waste in the form of spent reactor fuel (HLW) is commonly considered as back-up barrier to migration of possibly released radionuclides from the major engineered barrier, the clay-embedded metal canisters with waste, to the biosphere. This barrier consists of very tight “buffer clay” placed around and between the canisters for providing them with an almost impermeable and ductile embedment. In recent years it has been shown that the change in rock stresses caused by the construction of a repository at a few hundred
meter depth and the superimposed thermal effect of the radioactive decay can fracture the most heated “near-field” rock, activate water-bearing fractures hydraulically and mechanically, and create new ones. This, together with alteration of the regional stress field caused by exogenic impact like glaciation, reduces the role of the rock to be just to provide mechanical protection of the “chemical apparatus” (Pusch, 2014). It is well put by the logo of this symposium: “For eons the earth has been bending, buckling, contracting and expanding. And this unique dance gives rise to sculptures which are wonders of nature”. The ever ongoing massage continues to make discontinuities propagate, and changes the stress conditions so that new structural features evolve.

2. Rock structure

2.1 Basis of quantifying groundwater flow and mechanical strain

Crystalline rock is block-structured on any scale with somewhat undulating orientation of the fractures and fracture zones (Fig.1). In practice, sets of such discontinuities of all sizes can have different orientation and varying spacing but it is usually possible to generalize the rock structure for estimation of stability and hydrological conditions as exemplified in the paper.

Fig.1.

Fig.1 refers to a simple practical classification scheme of rock discontinuities that gives typical ranges of geometrical measures, hydraulic conductivity and mechanical stability listed in Table 1. The respective features refer to core and field data and to what can be observed or measured.
Table 1. Categorization scheme for rock discontinuities (Pusch, 1995).

| Geometry | Characteristic properties |
|----------|---------------------------|
| Order    | Length m | Spacing m | Width m | Hydraulic conductivity | Gouge content | Shear strength |
| Low-order (conductivity and strength refer to the resp. discontinuity as a whole) |
| 1st      | >E4      | >E3       | >E2     | Very high to medium    | High          | Very low      |
| 2nd      | E3-E4    | E2-E3     | E1-E2   | High to medium         | High to medium| Low           |
| 3rd      | E2-E3    | E1-E2     | E0-E1   | Medium                 | Medium to low | Medium to high |
| High-order (conductivity and strength refer to bulk rock with no discontinuities of lower order) |
| 4th      | E1-E2    | E0-E1     | <E-2    | Low to medium          | Very low      | Medium to high |
| 5th      | E0-E1    | E-1 to E0 | <E-3    | Low                    | None          | High           |
| 6th      | E-1 to E0| E-2 to E-1| <E-4    | Very low               | None          | Very high      |
| 7th      | <E-1     | <E-2      | <E-5    | None                   | None          | Very high      |

This categorization scheme has a simpler form used by some of the organizations that are responsible for disposal of HLW in Spain, Sweden and Finland (ENRESA, SKB, POSIVA). It has the following four categories:

- **D1** Very large discontinuity (“fault”). It must not intersect a repository,
- **D2** Large discontinuity (“fracture zone”). It may intersect a ramp or shaft leading into a repository but must not intersect or interact with those parts of a repository where waste is located,
- **D3** Moderately large discontinuity (“minor fracture zone”). It may intersect those parts of a repository where waste is located but not the positions of waste containers (canisters),
- **D4** Discrete water-bearing and mechanically active discontinuity (“fracture”). It may intersect any part of the repository including positions where waste containers are located.

Fig.2 is a corresponding general rock structure model proposed by the Spanish equivalent of Sweden’s Nuclear Fuel and Waste Management Co (SKB).
2.2 Hydraulic conductivity related to rock structure

Large-scale hydrological investigations are necessary in planning and site selection of HLW repositories. Of special importance is that the average hydraulic conductivity drops with increasing depth. In Sweden and Finland the upper 100 m of the bedrock is commonly very permeable, especially where major fracture zones are present and where zones of 3rd order are frequent (Fig. 1).

2.3 Repository concepts

We will distinguish here between shallow and deep repositories exemplified by the Swedish/Finnish KBS-3H design (Autio, 2008; SKB, 2008) and the deep-hole concept, VDH (Pusch, 2014), (Fig.4). The role of the clay is to tightly embed HLW canisters in very dense smectite (expandable) clay in perforated supercontainers and to seal the space between these (Pusch et al, 2015). For the HLW waste considered in Sweden and Finland, i.e. spent reactor fuel, the temperature of the near-field rock for KBS-3H will be 70°C at maximum and drop to 25°C in about 1000 years (Svemar, 2005). For the upper 2 km of VDH with no waste the temperature of the rock will be 60-70°C, while for the lower 2-4 km part, holding waste, it will be
150°C at maximum but drop to about 100°C after 1000 years. At 4 km depth the temperature will be down to 70°C after 10,000 years.

Fig.4.

The KBS-3H tunnels have a diameter of about 2 m and will be oriented in the direction of the major principal stress and hence also in the direction of the main groundwater flow. The regional hydraulic gradient in the direction of the tunnels can be 0.01 m/m (meter water head difference per meter flow distance). For VDH holes regional hydraulic gradients have no impact on vertical and horizontal migration of water. While boring is the only construction alternative for VDH, very careful blasting can be considered for KBS-3H\(^{(1)}\), which is taken here to be synonymous with any concept with similar function.

2.4 **Quantification of macroscopic flow through repository rock**

Field experiments made at 360 m depth in granitic rock at Stripa some 200 km northwest of Stockholm as part of the international Stripa Project have given comprehensive information on the hydraulic performance of the near-field of blasted and bored holes (Pusch, 1994). They comprised small- and large-scale flow experiments in granitic rock and tests of the general performance and sealing ability of dense smectite clay. The hydraulic testing was made in the inner part of a 30 m long drift at 360 m depth using the arrangement in Fig.5. The 11 m long test site was sealed off by concrete walls, and 7 m long boreholes were drilled radially from the drift with an average spacing of 0.2-1.0 m. The holes could be pressurized and sampled over

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\(^{(1)}\) The designation refers to SKB’s concept of horizontal placement of waste packages in long subhorizontal holes
selected parts so that water flow and pressure at different distances from the drift could be recorded along the drift by pressurizing defined parts of the boreholes in the inner gallery and recording flow into the outer one.

Fig.5.

A finite element flow model was applied for evaluating the hydraulic conductivity using the continuously recorded pressure and flow data. The drop in pressure from the inner to the outer gallery was found to be linear as verified by piezometer readings at steady state conditions (Börgesson and Pusch, 1992).

Assuming straight horizontal flow paths from the inner to the outer borehole gallery, and applying Darcy flow theory, it was concluded that the most shallow “blast-disturbed” zone, extending 0.75 m from the drift, had an average hydraulic conductivity of 1.2E-8 m/s, i.e. 2-3 orders of magnitude higher than of the virgin rock for which it had been evaluated as 3E-11 to E-10 m/s (Börgesson and Pusch, 1992, Pusch, 2008). The rock from 0.75 to 3 m depth, which was concluded to represent the “stress-generated” zone of excavation disturbance (EDZ), had an axial average conductivity that was 10 times higher than of the virgin rock, and a radial average conductivity of about one fifth of the that of the virgin rock, hence indicating a “skin” zone caused by high hoop stresses. The Stripa BMT flow test is still the only experiment that has been performed on a sufficiently large scale to verify that tunnel excavation by blasting has a significant effect on the conductivity of the near-field rock. The results are in complete agreement with preceding field tests ("Macroporomeability Experiment") in the same area performed by Laurence Berkeley Laboratories using ventilation techniques (Gale and Roleau, 1986).
2.5 Impact of construction on the hydraulic conductivity of the near-field of tunnels and boreholes

Construction of an underground repository changes the typical property of undisturbed crystalline rock of transporting water through relatively few interconnected fractures with considerable spacing, often several meters, to create additional flow paths close to drifts and tunnels. Here, new fissures and fractures can be formed and natural ones can widen and propagate by shifting of stresses to become effective water conductors both close to and distant from the excavated space. In its close vicinity numerous blast-generated fractures with a potential to transport water are created. Within a distance of 1-1.5 m from a tunnel wall even careful blasting can increase the net bulk hydraulic conductivity by an order of magnitude and in the floor of a blasted tunnel it can rise by three orders of magnitude as demonstrated by the described field experiments. The main reasons for considering blasting of long tunnels instead of boring is that the high hoop stress at the perimeter of bored holes are relaxed and that the cost is lower than for boring. A further advantage can be that the fragmented rock, the muck, has a granulometric composition that is more suitable for constructing fills and embankments for roads and railways than the fine debris from boring.

Assuming the data from the Stripa field tests to be generally valid the overall hydraulic effect of the EDZ is illustrated by Table 2, which shows the axial flow across an assumed near-field of 25 m² of a blasted or TBM-bored KBS-3H tunnel, and across 25 m² of virgin rock. The outer boundary of this circular area corresponds approximately to the extension of the “stress-generated” EDZ for a blasted 2 m-diameter tunnel for which the extension of the “blast-generated” zone with a hydraulic conductivity of E-8 m/s is taken as 0.75 m. For a bored tunnel the “bore-generated” EDZ extends only to 0.05-0.10 m distance from the tunnel perimeter and has an average cross section area of about 0.04 m². Its hydraulic conductivity is taken to be E-10 m/s as evaluated from laboratory tests of core samples and from field tests in another granite-dominated underground site, i.e. SKB’s underground laboratory at Åspö (Pusch, 2008).

The cross section area of the “stress-generated” EDZ is the difference between the 25 m² near-field area and the sum of the excavation-disturbed cross section area and the cross section area of the tunnel (3 m²). It is ascribed a hydraulic conductivity of E-10 m/s for both construction methods, which is one hundred times
higher than for the virgin rock. In reality, the conductivity of the “stress-generated” EDZ for a bored tunnel is lower than for blasted rock because of less dynamic impact of the construction work but it is conservatively assumed here to be the same.

The evaluated total axial flow across the 25 m² near-field section of a tightly backfilled blasted KBS-3H tunnel is about 4 times higher than for a TBM-bored tunnel. Of particular importance is that the flow across a 25 m² section of virgin rock is only one percent of the flow in the near-field of a sealed TBM-bored tunnel, meaning that even very tightly backfilled bored tunnels significantly affect the overall groundwater flow pattern in the host rock. For blasted tunnels the difference is even stronger.

Table 2. Approximate axial water flow across the assumed 25 m² near-field for the hydraulic gradient i=0.01 m/m across the respective EDZ components. The backfilled 2 m diameter tunnel is assumed to be impermeable.

| Case                     | Approximate permeated cross section, m² | Hydraulic conductivity, m/s | Approximate water flow, m³/s |
|--------------------------|----------------------------------------|----------------------------|----------------------------|
| Virgin rock,             | 25                                     | E-11                       | 3E-12                      |
| Blasted tunnel           | 22                                     | E-11                       | 8E-10                      |
| * Blast-EDZ             | 6                                      | E-8                        | 6E-10                      |
| * Stress-EDZ            | 16                                     | E-9                        | 2E-10                      |
| TBM-bored tunnel        | 22                                     | E-10                       | 2E-10                      |
| * Boring-EDZ            | 0.04                                   | E-10                       | 4E-14                      |
| * Stress-EDZ            | 16                                     | E-9                        | 2E-10                      |

2.6 Rock stability

2.6.1 Stress/strain mechanisms

The mechanical performance of the various discontinuities and their importance for selection of suitable sites for HLW repositories is determined by future movements in the rock mass intended for hosting them. It is commonly assumed that HLW packages must not be located in fracture zones or in discontinuities along which shearing can take place and the problem is to predict where future movements will take place. The governing factors are the
stress/strain properties of discontinuities of different kinds and their location and orientation with respect to the prevailing stress fields.

All fracture zones in crystalline rock can be assumed to behave as Coulomb material with negligible cohesion and dominating internal friction, illustrated by the graph in Fig.6, in which experimental data have been collected (Hökmark and Pusch, 1992). The low friction angle of low-order discontinuities explains why movements under applied or changed deviatoric stress conditions primarily take place in long ones, while those of limited length, i.e. 4\textsuperscript{th} and higher order fractures, are activated secondarily or not at all. The higher normal stress on the discontinuities at depth means that the shear resistance is stronger deep down and that shearing by tectonically induced changes of large-scale stress fields primarily takes place along the relatively few long-extending discontinuities, i.e. those of 1\textsuperscript{st} and 2\textsuperscript{nd} orders.

Fig.6.

2.6.2 Strain phenomena

Planners of repositories for HLW search for fracture-poor local areas, which can be identified by combining topographic analysis, deep borings, cross-hole measurements, and geophysical investigations. Such areas are represented by small rock blocks confined by 1\textsuperscript{st} and 2\textsuperscript{nd} order discontinuities, along which shear strain has been large (Fig.7). The thereby generated stress conditions in the blocks caused breakage and formation of 2\textsuperscript{nd} order discontinuities etc (Pusch, 2008). These are weaker than the rock between them, implying that large-scale strain has had and still has the character of slip along discontinuities of which the largest ones are weaker than the next largest
etc. The deviatoric stresses may not be very high in fracture-poor blocks but the normal stresses can be high and make KBS-3H tunnels unstable until the supporting mud and dense clay have solidified enough to provide stability.

The risk of shearing of fractures intersecting holes and tunnels by the thermal impact of hot waste packages and by seismic and tectonic events, determines where the waste packages should be located. With access to representative core samples and applying the principle of distribution of slip in the rock structure model one is able to identify the presence of 3rd and 4th order discontinuities and to select suitable positions of the waste packages. This matter is further examined here.

Fig.7.

2.6.3 Rock pressure and stability
A problem with fracture-poor rock is that the high rock stresses let so little water into the tunnels and holes that the clays seals may not become water saturated until after hundreds of years, during which the clay desiccates and loses part of its waste-isolating capacity even after subsequent hydration. A first and prominent difficulty is, however, the risk of failure in the construction and waste placement phases. This is a problem in the planning of a repository to be constructed in the Forsmark area north of Stockholm, where the maximal horizontal stress can be higher than 45 MPa at the intended repository depth of 400-500 m (Fig.8). Following rock mechanical principles the risk of failure of KBS-3H tunnels is minimized if the tunnels are oriented with their axes parallel with the major horizontal rock pressure but it has to be realized that the orientation of this pressure can vary significantly (Pusch, 2008).
The magnitude of the stability-controlling hoop stress of bored KBS-3H-type tunnels in crystalline rock at 400-600 m depth is determined by the subhorizontal rock pressure as shown by Table 3, which gives stresses calculated by using conventional elastic theory neglecting impact of canister-generated heat. Including also heat effects the hoop stress has been estimated to increase by at least 50 % (Kristensson and Hökmark, 2007). The difficulty is that the compressive strength is scale-dependent as illustrated by Fig.9, meaning that rock blocks having lower strength than standard 50 mm test samples will be exposed to nearly 100 % of the hoop stress of bored holes. Using a strength scale factor of 0.8 for a compressive strength of 150-300 MPa obtained from testing ordinary 50 mm cores, the block strength would be of the same magnitude as the maximal hoop stress 120 MPa in Table 3. A certain fraction of bored KBS-3H tunnels will hence fail early after placement of the hot waste packages.
Table 3. Calculated maximal compressive hoop stress in MPa of tunnels and shafts of a KBS-3H repository in granite/gneiss. Assumed vertical rock pressure\(^1\)=12-15 MPa at 400-600 m depth (Munier et al, 2001).

| Horizontal rock pressure (max/min) | Hoop stress of 2 m diameter KBS-3H tunnel oriented: |
|-----------------------------------|-----------------------------------------------|
| 45/30                             | Perpendicular to \(\sigma_1=120\), Parallel to \(\sigma_1=75\) |
| 40/25                             | Perpendicular to \(\sigma_1=105\), Parallel to \(\sigma_1=45\) |
| 30/20                             | Perpendicular to \(\sigma_1=75\), Parallel to \(\sigma_1=45\) |
| 25/18                             | Perpendicular to \(\sigma_1=60\), Parallel to \(\sigma_1=43\) |

\(^1\) SKB’s “A-berg” (SR 97, main report-Vol.I),

For VDH the conditions are different. The horizontal rock pressure is expected to increase with depth from 45/30 MPa at 500-600 m depth and further to about 60/60 MPa at about 2000 m depth, according to Herget’s formula (Pusch, 1995), taking the ratio of max and min horizontal pressure to be unity according to Rummel’s classical rule. For 4000 m depth this formula gives 77/77 MPa pressure. The maximal theoretical hoop stress of a VDH at this depth would hence be 154 MPa for a water-filled hole, which is a little too high to avoid spalling. Recording of rock pressure in Germany and elsewhere has given higher values (Pusch et al, 2014) implying higher hoop stresses: 100 MPa for 2000 m depth and 190 MPa for 4000 m depth, causing local spalling or rock fall. By filling the hole with smectite mud weighing at least 1150 kg/m\(^3\) (in air) stability would be provided but a pervious metal net casing has still been recommended to provide extra support. The matter of selecting a suitable mud density has been frequently discussed but among recent recommendations one notices that mud densities as low as 1020 kg/m\(^3\) would be sufficient for stabilizing 4.5 km deep holes, while 1420 kg/m\(^3\) can be needed for a 5 km deep hole (Brady et al, 2009). The dense clay in the supercontainers interacts with the clay mud to give a homogeneous dense clay ultimately both in the upper 2 km sealed part of the holes and in the lower 2 km part that contains both canisters embedded in clay and pure clay seals (Pusch et al, 2015).

The criterion that no supercontainers shall be installed where the tunnels and holes intersect water-bearing fracture zones is motivated by the risk of erosion of the mud and the canister-embedding clay. Concrete seals cast here
provide significant tightness and mechanical support to adjacent supercontainers with or without canisters (Pusch and Ramqvist, 2007, Pusch et al, 2012). The risk of erosion depends on the rate of groundwater flow, which may be negligible in 3rd order fracture zones in the waste-bearing part of VDH (cf. Section 2.6.5).

2.6.4 Rock fall

Certain fractures of 4th order type can cause rock fall as illustrated in Fig.10. The risk naturally depends on the rock structure and falls can have very bad consequences in the placement of supercontainers even if fallen rock fragments are small. Problems with unstable rock wedges calls for bolting and grouting. The risk exists only for the KBS-3H concept since the mud in VDH prevents rock fall. The earlier mentioned pervious casing serves to prevent rock fall in the early construction phase, i.e. when the clay mud is being placed.

Fig.10.

2.6.5 Role of rock structure and stress conditions for shearing of waste packages

In shallow rock with high frequency of potentially active minor fracture zones, i.e. discontinuities of 3rd order, these are exposed to relatively low normal pressure. The risk of seismically or tectonically generated shearing of VDH holes or KBS-3H tunnels intersected by such zones is hence higher than for deep rock where they are under high pressure. Since their spacing ranges between a few tens to a hundred meters a considerable number of supercontainer positions in a shallow repository would have to be abandoned while the high pressure of VDH rock at depth would allow waste packages also where minor fracture zones are intersected. The upper 2 km of VDH holes will be intersected by potentially slipping 3rd order fracture zones as frequently as KBS-3H tunnels but less frequently deeper down where the higher normal pressure means that shearing is concentrated to 1st and 2nd order discontinuities. Here, it may be possible to place supercanisters where minor fracture zones intersect a VDH. The
high friction angle of discrete fractures of 4\textsuperscript{th} and higher orders makes it possible to accept them in all parts of a VDH.

3. **Role of rock structure and stress conditions for groundwater movement in the near-field**

3.1 **General philosophy**

The most important issue in forecasting migration of possibly released radionuclides from HLW packages via groundwater is the waterborn transport through repository rock. Focusing on the near-field of waste packages one must consider the impact of excavation disturbance by blasting and boring for KBS-3H repositories and the role of boring-disturbance for VDH. The finest discontinuities in the rock matrix are believed to be practically closed at depth and large-scale discontinuities with gouge retain their tightness because of the high normal pressure compared with the conditions in more shallow rock.

3.2 **Role of a blast-disturbed EDZ**

One finds from the description of how excavation disturbance affects the hydraulic performance of the nearfield of KBS-3H tunnels in Section 2.3.1 that blasting makes the near-field more permeable than boring but that the difference is not very significant. It may in fact be balanced by the positive effect of stress relaxation caused by the impact of dynamic forces that lead to an increased degree of fracturing and fissuring (Fig.11). The relaxation caused by the softening of the near-field of shallow repositories can make it possible to construct tunnels where the rock pressure is very high, which is exploited by the Swedish Nuclear Fuel and Waste Co (SKB) in their planning of a HLW repository at Forsmark north of Stockholm.

Boring of KBS-3H tunnels causes very limited disturbance. At 400-600 m depth water inflow takes place primarily where 4th order are intersected, causing problems with local strong “spray” inflow of groundwater that can erode the mud around the supercontainers and the clay within them. In blasted tunnels inflowing water is distributed among numerous fractures and fissures with less risk of erosion.
3.3 Role of a stress-generated EDZ

The increase in rock pressure with depth means that the hydraulic conductivity of the virgin rock where the waste-bearing part of VDH would be located, i.e. from 2 to 4 km depth, is at least one order of magnitude lower than at 500-600 m depth where a KBS-3H repository would be constructed, as indicated by the diagram in Fig.3.

Boring of VDH holes causes very limited disturbance. The rate of water saturation of clay seals in VDH is determined by the very strong suction potential of the clay rather than by the high water pressure. Casting of low-permeable concrete seals where the holes intersect significantly water-bearing fracture zone contributes to make the holes tight and mechanically stable (Pusch et al, 2012).

4. Discussion and conclusions

4.1 Preliminary assessment of the function, constructability and cost

The increase in rock pressure with depth means that the hydraulic conductivity of the rock where the waste-bearing part of VDH is located is significantly lower than for the shallow KBS-3H. This makes the VDH concept more attractive but other facts do also speak in favour of this solution. One is that retrieval of the supercontainers by overcoring is feasible while it would be much more expensive, risky and difficult for KBS-3H. Another fact of even greater importance is the high density of possibly contaminated groundwater deeper than about 1500 m: it makes the heavy water stay at depth and not move upwards and mix with or pollute shallow groundwater (Claesson, 1992; Åhäll, 2006).
A practical issue for the construction of the respective repository types is the prevailing and generated hydraulic gradient, which is of no problem for VDH but which can be very high for KBS-3H until it is fully water saturated, causing erosion of injected mud and of dense clay in the supercontainers. Practical difficulties are foreseen for the latter concept in the placement of the supercontainers that have a weight of more than 46 tonnes for KBS-3H compared to the about 80% lighter ones for VDH. The light supercontainers can be inserted without difficulties provided that the installation can be completed before the shear strength of the mud has become so high that they have to be forced down (Yang, 2015). The smaller amount of HLW in VDH (about 15-30%) requires a higher number of holes than for KBS-3H but rational planning and use of the host rock, i.a. by drilling several VDH holes with slightly different orientation from the same shallow underground chamber, can bring the cost down substantially.

4.2 Development and testing
No demonstration of the constructability and function of VDH or KBS-3H holes and tunnels, or parts of them, has yet been made. Two ways of studying large-scale hydrological and mechanical performances of the two concepts with special respect to the role of the groundwater salinity and of major changes in the stress field are recommended:

- Theoretical modelling of the evolution of temperature, hydraulic gradients, groundwater flow and strain in the rock and engineered barriers for KBS-3H and VDH assuming relevant rock structure models and changes in stress field in the first 100, 1000, and 10,000 years,
- Boring of VDH hole for field study of the performance of deployment mud, supercontainers with dense “buffer” and clay and electrical heaters simulating HLW canisters, recording the evolution of temperature, stress and strain in a 100 m deep hole in crystalline rock.

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**Figure captions**

Fig.1. Schematic model of mechanically and hydrologically important discontinuities in crystalline rock. Shallow and deep HLW repositories are dotted.

Fig.2. ENRESA’s general rock structure model with major fracture zones. *Fractura de primer orden* corresponds to 1st order discontinuities, *Fractura de Segundo orden* represent those of 2nd order. *Emplazamiento genérico* is space for locating HLW repository.

Fig.3. Example of recorded hydraulic properties of granite/gneiss rock using packed-off deep drillings (Gideå). The left boundary represents rock with only 4th and higher-order discontinuities. The boundary to the right includes minor fractures zones of 3rd order (SKB data).

Fig.4. Principles of storing HLW in long, bored holes with waste packages separated by clay seals.

Fig.5. Test set-up for determination of the hydraulic conductivity of the near-field of the BMT test drift (Börgesson and Pusch, 1992). A is water-filled rubber bladder and B bentonite slurry that prevented water in the rock to flow into the drift. K represents galleries of radially bored holes for pressurizing and collection of migrated water.

Fig.6. Literature survey of rock friction angles referring to discontinuities of different extension (Hökmark and Pusch, 1992). “1” and “2” represent separate studies.

Fig.7. Relative movements of large rock blocks caused by rotation of the regional stress field and resulting in shearing of large discontinuities and elastic strain of the rock matrix and integrated minor discontinuities in the blocks (After Stephansson).

Fig.8. Major principal stress below about 300 m depth recorded by SKB in the Forsmark area north of Stockholm (Generalized from SKB report TR-11-04).

Fig.9. Influence of size of sample with one type of defects on the compressive strength. The drop in strength at increased volume is explained by the increasing number of xenolites and 5th, 6th and 7th order discontinuities and the greater possibility of critical orientation and interaction of the defects in larger samples (Pusch, 2008).
Fig. 10. Unstable rock wedge that can slip off if the shear resistance of the fractures separating them from the main rock is too low.

Fig. 11. Creation of blast-generated EDZ. Left: 1a-zones have regular plane fractures extending from contour blast-holes; 1b-zones represent strongly fractured parts at the charged tips of the holes (Pusch, 1994). Right: Rock matrix with 5th order fissures integrated in 4th order system after blasting causing disintegration and high hydraulic conductivity according to 2D numerical calculation (After Hökmark).