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Rock quality designation of the hydraulic properties in the near field of a final repository for spent nuclear fuel

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ROCK QUALITY DESIGNATION OF THE HYDRAULIC PROPERTIES IN THE NEAR FIELD OF A FINAL REPOSITORY FOR SPENT NUCLEAR FUEL

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ABSTRACT

Quality assurance of a final repository for spent nuclear fuel requires detailed information on the characteristics of the rock, backfill, canisters and the waste itself. Furthermore, and of fundamental importance, is the knowledge on the behaviour of the integrated system of the waste and the different barriers. The in-situ characteristics of the rock must therefore be assessed and their influence on and interactions with the remaining barriers must be predicted and verified.

A rock quality designation process of the hydraulic properties in the near-field is out-lined both for the KBS-3 system as well as for the WP-Cave system. The process, once updated and approved, will be included in a Quality Assurance Program for the final repository for spent nuclear fuel.

Some of the available methods for the near-field designation process are presented as well as techniques that need further development or are not developed at all.

Finally, a presentation is given of a generic designation process of the KBS-3 and WP-Cave repository systems in the previously investigated area in Central Sweden where the final repository for reactor waste, SFR, is located. Geological and hydrogeological data are here at hand and it is therefore possible to carry out a simulation of how the designation process would be accomplished.

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Quality assurance of a final repository for spent nuclear fuel requires detailed information on the characteristics of the rock, backfill, canisters and the waste itself. Furthermore, and of fundamental importance, is the knowledge on the behaviour of the integrated system of the waste and the different barriers. The in-situ characteristics of the rock must therefore be assessed and their influence on and interactions with the remaining barriers must be predicted and verified.

A rock quality designation process of the hydraulic properties in the near-field is out-lined both for the KBS-3 system as well as for the WP-Cave system. The process, once updated and approved, will be included in a Quality Assurance Program for the final repository for spent nuclear fuel. However, the suggested strategy and investigations should be adopted with great flexibility depending on the site specific conditions.

The designation process of the hydraulic properties in the near-field is closely linked to the actual construction of the repository. In the construction sequence for a KBS-3 repository there are details not fully decided upon yet like the use of a shaft or a spiral ramp for access to the repository within or outside the identified rock block. However, once down at the repository level, the actual construction of the repository will start by excavation of a central or main tunnel from which a system of storage tunnels for deposition holes is constructed.

The construction phase of a KBS-3 repository will be divided into four main stages as:

1. Excavation of access shaft and access tunnels
2. Excavation of central (main) tunnel
3. Excavation of storage tunnels
4. Drilling of deposition holes

During each stage, a strategy is outlined regarding techniques possible to use for rock quality designation of the hydrogeological conditions and properties important for the long term safety of a final repository for spent fuel. Geological, geophysical, hydrogeological and hydrogeochemical investigations are performed for collection of data for conceptual and predictive models. The predicted data is compared with field investigations in an iterative manner for the different construction phases. In this way, the most favourable volumes of

rock can be designated for the final locations of the deposition holes. Adverse hydraulic features, intersecting the excavations or located in their vicinity, will be sealed by grout or bentonite injection, thus enhancing the quality of the rock by redirecting the groundwater flow away from the waste.

A repository designed according to the WP-Cave principle will be located at a depth determined from site specific data. The main principle when locating a WP-Cave is the same as for a KBS-3 repository, that is in a rock block surrounded by major vertical fracture zones.

The WP-Cave construction can generally be divided in five main stages comprising construction of:

1. Access shafts and tunnels
2. Annular tunnels
3. Hydraulic cage
4. Bentonite-sand barrier
5. Waste storage cavern

After the hydraulic cage is constructed, the resulting egg-shaped volume will be more or less drained by the numerous boreholes constituting the cage. The function of a WP-Cave is further based on the presence of the bentonite-sand barrier in combination with the hydraulic cage. As in the case of a KBS-3 repository, the adverse features in the bedrock will be transmissive fractures.

Outside the hydraulic cage, transmissive fractures would connect the cage hydraulically to the regional groundwater flow in major bounding fracture zones. In the WP-Cave study, the term near field is considered primarily to be the volume encompassed by the hydraulic cage. The rock quality designation for a WP-Cave repository thus primarily aims at describing the rock volume inside the cage. As in the case of the KBS-3 system, geological, geophysical, hydrogeological and hydrogeochemical investigations are performed for collection of data for conceptual and predictive models. The predicted data is compared with field investigations in an iterative manner for the different construction phases. However, in contrast to the KBS-3 concept, the flexibility of a WP-Cave is limited once the hydraulic cage is laid out and the annular tunnels constructed. To compensate for this, a thoroughly pre-investigation including a detailed conceptual model of the assigned rock block need to be conducted. Also a heavier use and confidence of engineering barriers is required compared to the KBS-3 concept.

1 INTRODUCTION

Quality Assurance of a final repository for spent nuclear fuel requires detailed information on the characteristics of the rock, backfill, canisters and the waste itself. Furthermore, the knowledge on the behaviour of the integrated system of the waste and the different barriers is of fundamental importance. The in-situ characteristics of the rock must therefore be assessed and their influence on and interactions with the remaining barriers must be predicted and verified.

The only feasible transport mechanism of leaching radionuclides from the repository to the biosphere is by dissolution and transport in flowing groundwater. Of uttermost importance is therefore the assessment and interpretation of the hydraulic characteristics and properties of the rock.

The hydraulic properties of the far-field have been investigated and the hydrogeological conditions have been presented for several potential repository sites in Sweden /SKB, 1988/. However, the rock quality designation process of the hydraulic properties of the near-field in a final repository for spent nuclear fuel has never been approached before.

The designation process of the hydraulic properties of the near-field is strongly linked to the far-field hydrogeological conditions as well as to the lay-out and construction of the repository. The overall purpose of the near-field designation process is to determine the hydraulic properties and conditions of the rock in the vicinity of the shafts, tunnels and deposition holes. However, the interconnectivity of hydraulic features in the near-field with major fracture zones or hydraulic conductive fractures in the far-field must be assessed. A flexibility in the construction phase of the repository is desireable so that hydraulic conductive features can be avoided in order to minimize the flow in the rock adjacent to the spent fuel. Where possible, depending on the repository lay-out, the near-field designation of the hydraulic properties will thus guide the final decision on the location of tunnels and deposition holes.

Sealing of adverse hydraulic features by tunnel plugging or bentonite and grout injection will greatly enhance the near-field rock quality. The sealing will redirect the groundwater flow away from the close vicinity of shafts, tunnels and deposition holes as well as delay diffusion of corroding agents and radionuclides in the near-field region.

This report out-lines the process for rock quality designation of the hydraulic properties in the near-field both for the KBS-3 system, c.f. Chapter 5, as well as for the WP-Cave system, c.f. Chapter 6. Chapter 7 presents some of the available methods for the near-field designation process as well as techniques that need further development or are not developed at all. Chapter 8 and 9 describes a generic designation process of the two repository systems in the previously investigated area in Central Sweden where the repository for reactor wastes, SFR, is located. Geological and hydrogeological data are here at hand and it is therefore possible to carry out a simulation of how the designation process would be accomplished. In Chapter 10, some preliminary groundwater modelling exercises are performed comprising simulation of an interference test carried out during construction of the Final Repository for Reactor Waste, SFR, at Forsmark.

2 ROCK QUALITY DESIGNATION

2.1 GENERAL

In general, a Quality Assurance (QA) program provides a framework for a planned and disciplined consideration of all factors that may influence the quality of tasks, performances, research results or services for the accomplishment of a specified operation. It describes the means of applying quality assurance principles to all of the activities carried out in the specific operation. The application of the QA principles will depend on the importance and complexity of the activity and on the need to apply controls to promote a safe, high quality, reliable and cost-effective operation.

The QA program for a repository for final storage of radioactive waste is a widely discussed issue. The program is quite complex and includes a number of standard QA terms. Some of the most commonly used expressions are listed below /AECL, 1988/. Figure 2-1 shows the schematic explanation of the terms and in what order they are accomplished.

Quality - In the technical sense, definable, or controllable or measurable or verifiable properties, features or characteristics of a study, investigation, design, material, process or product. Quality is frequently defined in the physical sense as the fitness of a product or service for intended use.

Quality Achievement - is by definition, /AECL, 1988/, the performance by line organizations of quality related activities, such as drilling, logging, testing, designing, constructing, operating and decommissioning, in accordance with written procedures whereby technical criteria are met. In this report, the term **Quality Designation** is used for these activities.

Quality Assurance (QA) - All the planned and systematic actions necessary to provide adequate confidence that products and services will satisfy specified requirements. When the product is a report of a significant study or investigation, quality assurance comprises those planned and systematic actions necessary to provide adequate confidence in the validity and integrity of reported data, methods, procedures, conclusions, interpretations and recommendations.

Quality Assurance Program - An Organization's total concept requirements and scope of effort for achieving and verifying quality described by written quality

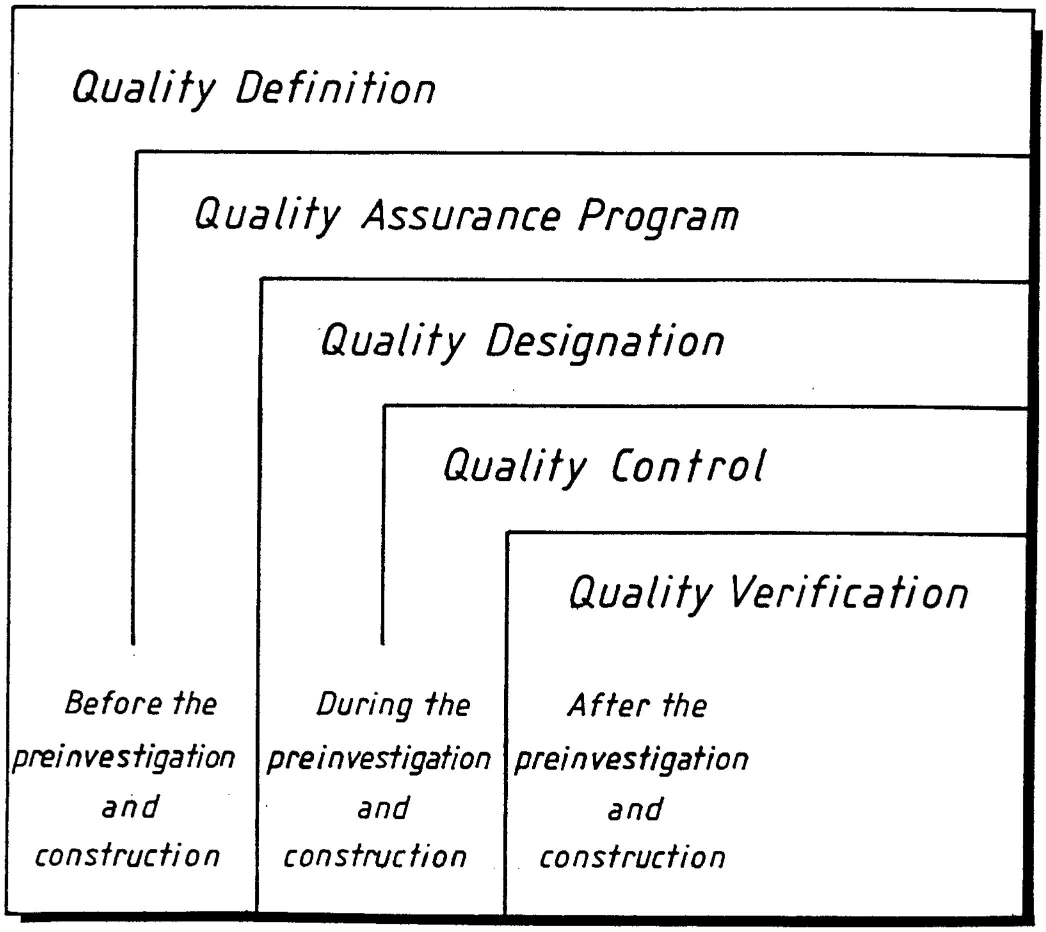


Figure 2-1. Schematic drawing of the major components regarding Quality Assurance of a final repository for spent nuclear fuel.

assurance manuals. The manuals set forth quality assurance policy, objectives, requirements, authority and responsibility, organization, methods and activities required to implement and assess the adequacy and effectiveness of the program.

Quality Control (QC) - Consists of a comprehensive process of identifying and specifying technical quality criteria and requirements, controlling work performance, measuring conformance to administrative and technical methods and procedures, applying statistical quality control and measurement methods, and preventing, migitating or correcting quality defiencies, as appropriate to the work-specific activity.

Quality Verification - Includes the activities of reviewing, inspecting, testing, checking, assessing, auditing, surveillance, monitoring or otherwise verifying that items, designs, processes, data, codes, documents or activities conform to established criteria. Independent quality verification is performed by individuals other than those who performed or supervised the activity, but who may, in some cases, be from the same organization.

3 REPOSITORY DESIGN SYSTEMS

Two systems of repository design for final storage of spent nuclear fuel are considered in this report - the KBS-3 system and the WP-Cave system. The well known KBS-3 design is solely developed by SKB whereas the WP-Cave design originally was developed by Boliden WP-Contech AB. Both systems are now part of the extensive research program carried out by SKB in terms of repository performance, safety and economy.

3.1 KBS-3 DESIGN

The KBS-3 report /1988/, describes a multi-barrier system for safe final disposal of spent Swedish nuclear fuel. The fuel is contained in copper canisters and is expected to resist corrosion for one million years. The canisters are surrounded by highly compacted bentonite clay and the entire package is placed in vertical boreholes 6 meters apart in the floor of a horizontal tunnel system, c.f. Figures 3-1 and 3-2.

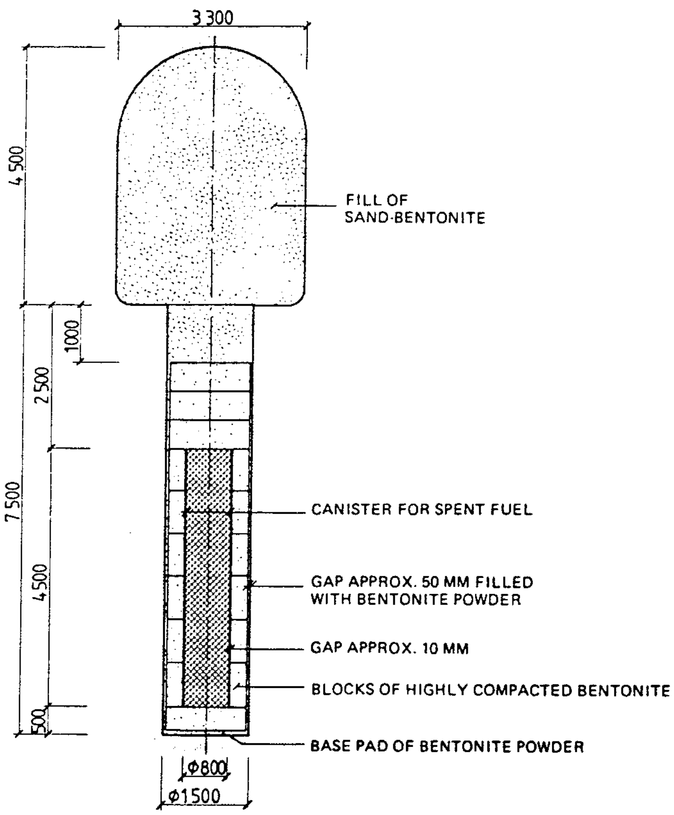


Figure 3-1. Deposition hole with canister, buffer material and backfill in a storage tunnel /KBS-3, 1983/.

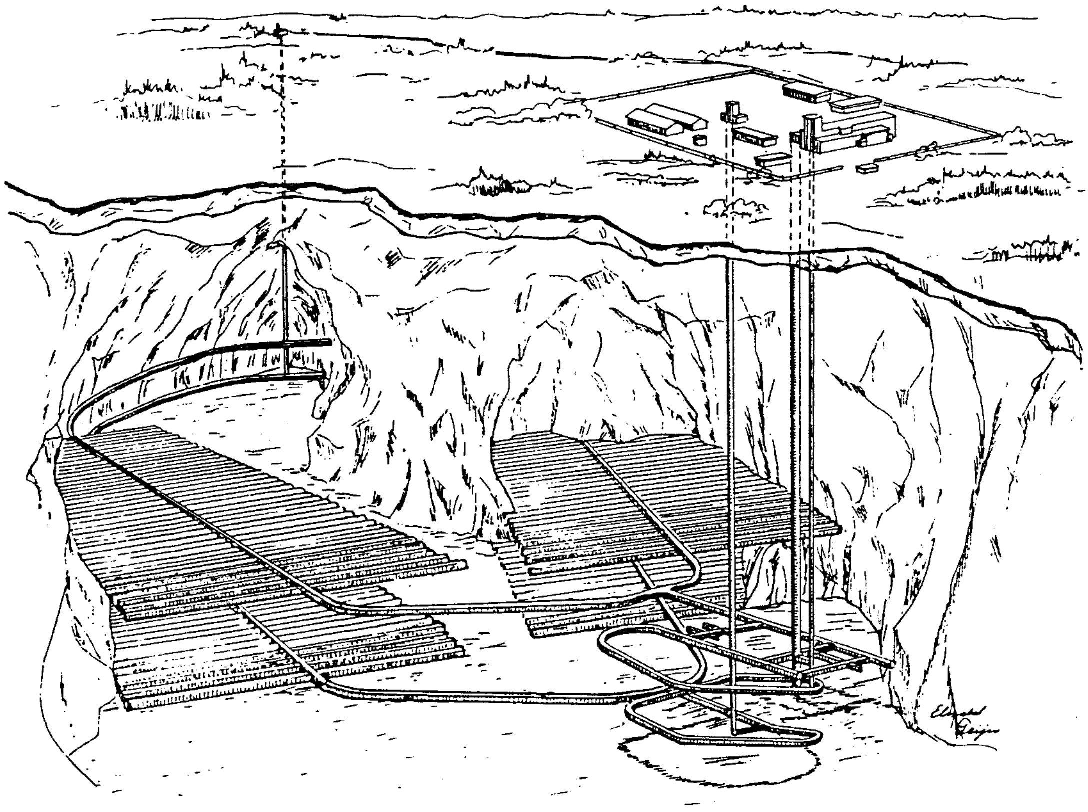


Figure 3-2. Artists view of tunnels and shafts in the final repository for spent fuel /KBS-3, 1983/.

The only probable transport mechanism of radionuclides from a penetrated waste canister to the biosphere is by flowing groundwater. The smectite-rich bentonite clay will absorb the water, swell and act as a sealant in fractures intersecting deposition holes and tunnels and thus limiting the transport of nuclides through the bentonite to the slow diffusion process, c.f. Section 4.3. After emplacement of the waste packages in the deposition holes, the entire tunnels and access shafts are filled with a mixture of sand and bentonite and the repository is left unattended for years to come.

The spacing between the storage tunnels and the depth below the surface to the repository level will be determined by the geological conditions at site. In addition, these distances are guided by the generated heat from the spent fuel. However, a minimum spacing of 25 meters between the storage tunnels is needed when a maximum temperature of 80^{o} C is desired for the hottest canister assuming 40 years old fuel. Furthermore, a single level repository is expected to be located at about 500 meters depth. A multi-level storage with an approximate distance of 100 meters between the levels may be

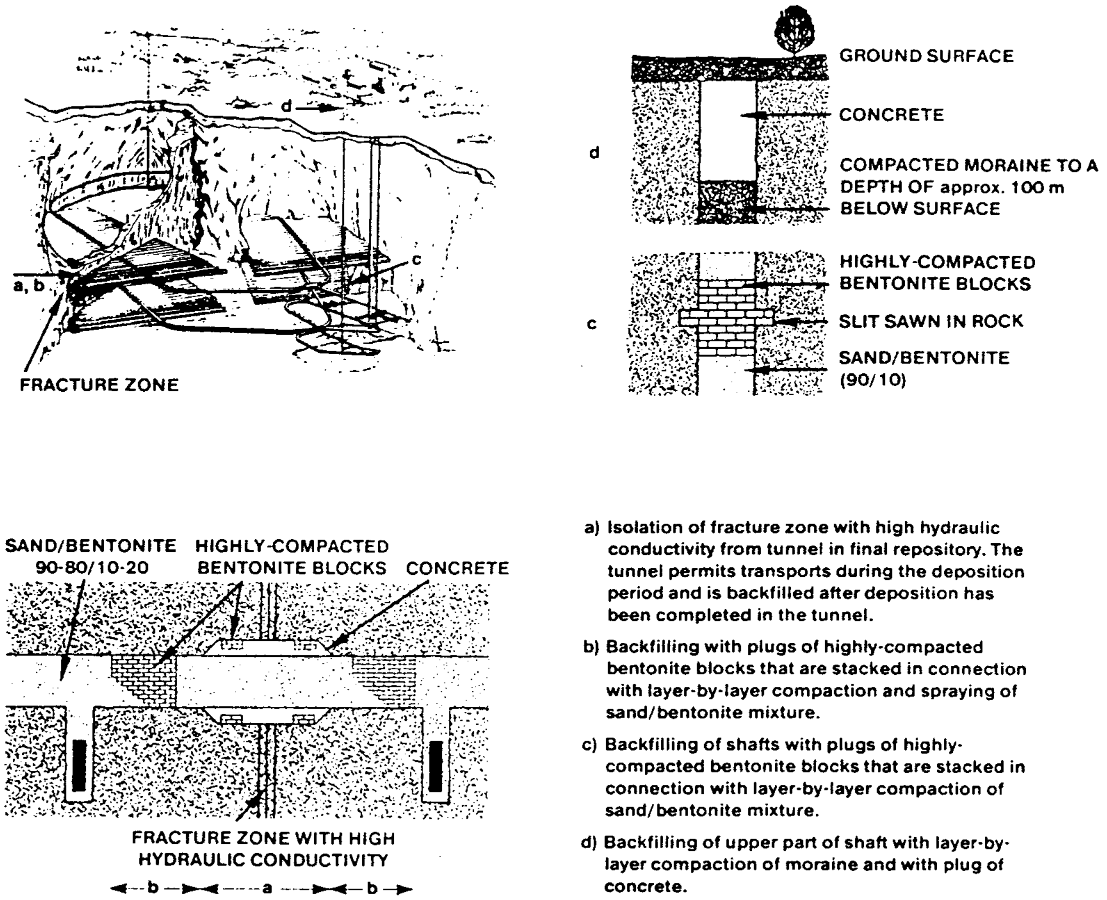


Figure 3-3. Plugs of highly compacted bentonite in tunnels and shafts in the final repository /KBS-3, 1983/.

considered if found appropriate due to the geological conditions.

The isolation properties of the repository is determined by the interaction of the canisters, the buffer material, the rock and the groundwater flow and chemistry. It is obvious that more hydraulic conductive zones should be avoided and less permeable rock should be preferred when siting the deposition holes. Additional measures for isolation of fracture zones, borehole and shaft sealing are taken by backfilling, plugging and sealing of fractures, c.f. Figure 3-3. The overall objective is thus to minimize the groundwater flow in the rock adjacent to the spent fuel. This can be obtained by using the design flexibility of the KBS-3 system to the prevailing rock conditions and the information obtained during the construction of

- the first shaft for the decision of the repository level;

- the first drifts for the decision of the most appropriate direction of the central and/or storage tunnels;

- the storage tunnels and the drilling of pilot boreholes for the decision of suitable sites for the deposition holes.

The quality assurance of the rock as a barrier is based on the capability of selecting favourable rock from the results of thoroughly performed investigations. These investigations include characterization of hydraulic active zones in the far field and their connectivity with the near field. It also includes studies of discontinuities that might alter the prevailing hydraulic conditions and/or the mechanical integrity of the canisters.

Theoretical modelling of groundwater movements due to relevant hydraulic pressure conditions after sealing is needed for the evaluation of the groundwater transport in the near field. There is also a need to model the mechanical behaviour of the rock which is influenced by temperature and/or tectonics.

3.2 WP-CAVE DESIGN

The WP-Cave repository is designed as an egg-shaped structure, c.f. Figure 3-4, and the dimensions are determined by the storage capacity. Different sizes of the structure are being studied but a WP-Cave with a storage capacity of up to 1600 tonnes of U will have a bentonite slot that is 300 meters in height and 130 meters in diameter. The diameter of the hydraulic cage is 230 meters /SKB 86-31, 1987/.

The Swedish nuclear back-end program is in totally expected to comprise 7600 tonnes of U up to year 2010 thus requiring a minimum of 5 WP-Caves. For practical and economical reasons all of these caves are expected to be located in one and the same area surrounding a central shaft at a radial distance of 250 - 300 meters. The required area is thus approximately 0.6 km^2 in good quality crystalline rock which is about the same as for a two-storey facility of the KBS-3 design. However, as in the KBS-3 design, the "respect distance" (defined in the KBS-3 report as the distance to the nearest fracture zone or section of rocks with high hydraulic conductivity) of 100 meters might increase the area needed.

The depth location of the WP-Caves can be chosen on the basis of site-specific data on rock structure, virgin stresses and hydrogeology but the hydraulic cage is expected to be constructed within the depth range of 100 - 600 meters. The geoscientific preinvestigations for the five WP-Caves are therefore similar to those

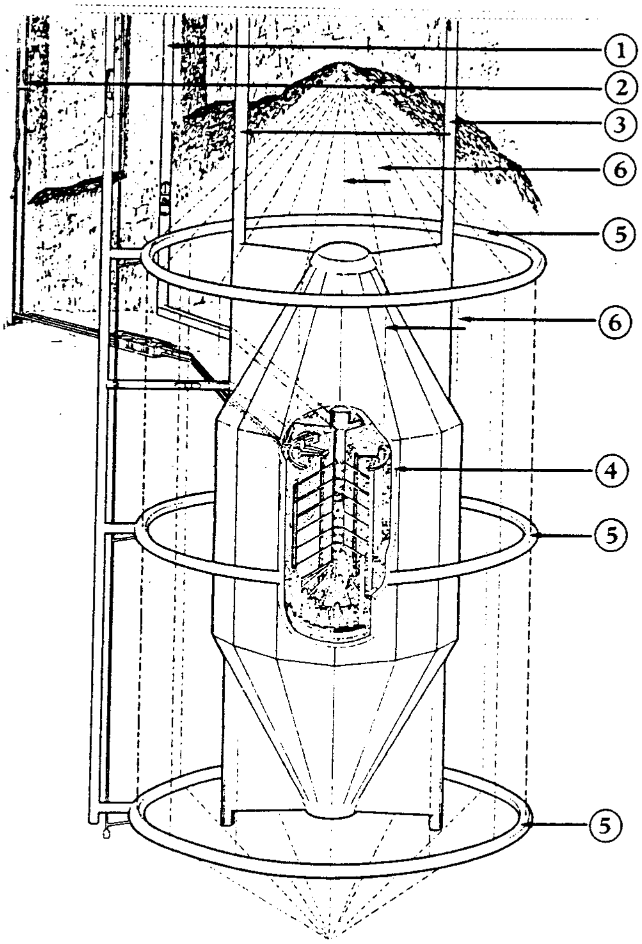


Figure 3-4. Artists view of a WP-Cave structure with a capacity of up to 1600 tonnes of U. The dimensions of the bentonite-sand barrier are 300 meters in height and 130 meters in width /SKB 86-31, 1987/.

performed for a repository of the KBS-3 design.

The deposition of spent fuel in a WP-Cave repository is followed by a cooling period of 100 years assuming disposal of 40 years old fuel. The waste canisters will be made of iron with an assumed service life of a few hundred years. The canisters are positioned centrally in radial sloping canister channels, c.f. Figure 3-4, with a gap for air circulation. The channels are connected to an inner and outer shaft and the cooling is maintained by circulating air heat exchanged with surface air directly or via an underground intermediate heat exchange plant.

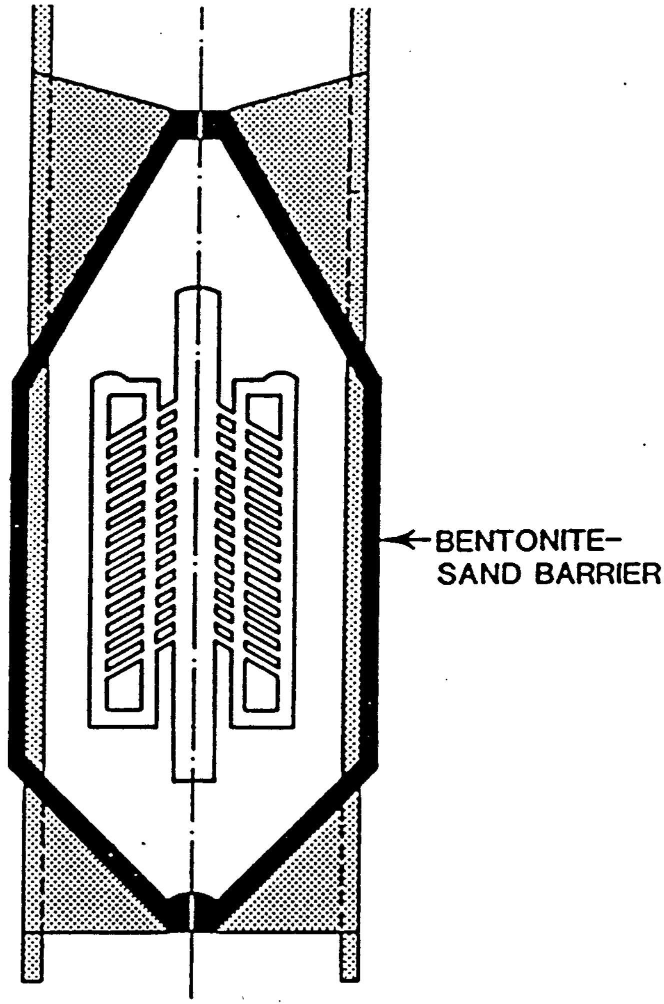


Figure 3-5. Schematic view of the design of the 5 meter wide bentonite slot surrounding the canister channels at a distance of 20 meters /SKB 86-31, 1987/.

The storage excavations are surrounded by a bentonite-sand barrier with a width of about 5 meters, c.f. Figure 3-5. Outside this barrier, a large number of boreholes are drilled to form a hydraulic cage, c.f. Figure 3-4, which is expected to conduct most of the groundwater past the storage at some distance from the bentonite-sand barrier. During the cooling period the hydraulic cage acts as a drainage system and the infiltrating water is pumped to the surface.

After the cooling period, the repository will be sealed off and waterfilled. The bentonite-sand slot will reduce the hydraulic conductivity and the hydraulic cage will reduce the hydraulic gradients.

It is claimed that the WP-Cave design is based on flexible, engineered barriers which can be adapted to various rock qualities. One main requirement which must be fulfilled, however, is that the strength of the rock and the in situ stress situation must allow for construction of the inner part (the storage) as well as of the bentonite-sand barrier. Another major requirement, as specified by the inventor Boliden WP-Contech, is that the ground water flow in the rock mass must be sufficiently low.

Several aspects of the safety of a WP-Cave repository functions are currently being studied. The evaluation of the release of radionuclides, the thermo-induced groundwater flow and the behaviour of the hydraulic cage are examples of further research that needs to be undertaken before a complete safety assessment of the WP-Cave design can be accomplished.

4 NEAR-FIELD BARRIERS

4.1 GROUNDWATER FLOW

The groundwater flow in a fractured crystalline bedrock takes place in fractures and fracture zones. The latter ones are usually the most prominent water conductors although single fractures of large extent can be of great importance for the groundwater flow.

In the Swedish site investigation program, the fracture zones are classified according to their length and width, that is, until additional information is available, a non-genetic classification. Two classes of fracture zones may be defined - regional and local fracture zones. However, and as in this report, the observed zones are generally referred to as major and minor fracture zones.

There is no strict definition of the concept of a major fracture zone. In fact, the most common concept considers a major fracture zone to be highly permeable, to have a large, usually traceable extension and a width of more than about ten meters. However, all of these criteria may not be fulfilled for every zone considered as a major fracture zone. On a regional scale over tens of kilometers, the major zones encountered are identified as regional fracture zones. Their location is usually based on aerial photographs, topographical maps and airborne geophysical measurements. The spacing between these zones varies as shown in Table 4-1. Since this type of major fracture zones usually is avoided in a site investigation study, only limited data on their properties and geometries are available. The width of these zones usually exceeds 50 meters and the zones delineate rock blocks of one to fifteen km^2 size.

Between the regional fracture zones, other fractures zones of different orders are usually encountered. The distribution and nature of these zones are very site dependent. These major fracture zones are generally termed local fracture zones. The width of these zones typically range from 1 to 25 meters and the length exceeds 1 km. The spacing of the local fracture zones varies between different sites, but is typically between 100 and 1000 meters, c.f. Table 4-1. They will thus most probably occur within or very near a repository.

Major fracture zones can be identified from the ground surface, providing they are inclined. However, horizontal or sub-horizontal zones are more problematic to

Table 4.1 Data on spacing of regional and local fracture zones encountered in the Swedish site investigation program.

Site	Spacing in kilometers between zones Regional zones	Local zones
Fjällveden	2 - 3	0.3 - 0.9
Gideå	3 - 5	0.4 - 0.8
Kamlunge	1 - 5	0.5 - 1.0
Svartboberget	1 - 3	0.1 - 0.6
Finnsjön	2 - 10	0.1 - 0.3

detect from ground-surface features. Core drilling and borehole testing might however reveal the existence of such zones at depth.

Horizontal major fracture zones having increased hydraulic conductivity are positive features when siting a repository providing these features are located above the repository level. Model calculations of the hydraulic conditions at the site Kamlunge have shown that the existence of a horizontal fracture zone having a hydraulic conductivity 300 times that of the surrounding rock will decrease the groundwater flow at the repository depth by a factor 0.4 /Carlsson et al, 1983/. It should be remembered however that the travel distance for leaking radionuclides in the vertical direction from the repository to the hydraulically defined biosphere will decrease when horizontal zones are present.

The major fracture zones delineate the bedrock into blocks, c.f. Figure 4-1. Such blocks are the target for the siting of a nuclear waste repository in the Swedish bedrock.

The distances to the nearest major fracture zone or section of rocks with high hydraulic conductivity - the "respect distance" - is in the current report set at 100 m, calculated from the outermost tunnel-walls and the upper ceiling where horizontal zones are present.

In a defined rock block assigned for a repository, other fracture zones not defined as major, will be present. The hydraulic properties of these minor zones might be of the same order as of the major fracture zones but their width and extent are usually less.

The rock block or blocks also contains single fractures of various extension, orientation and properties. Investigations on core-samples within the Swedish site investigation program has at several sites shown a

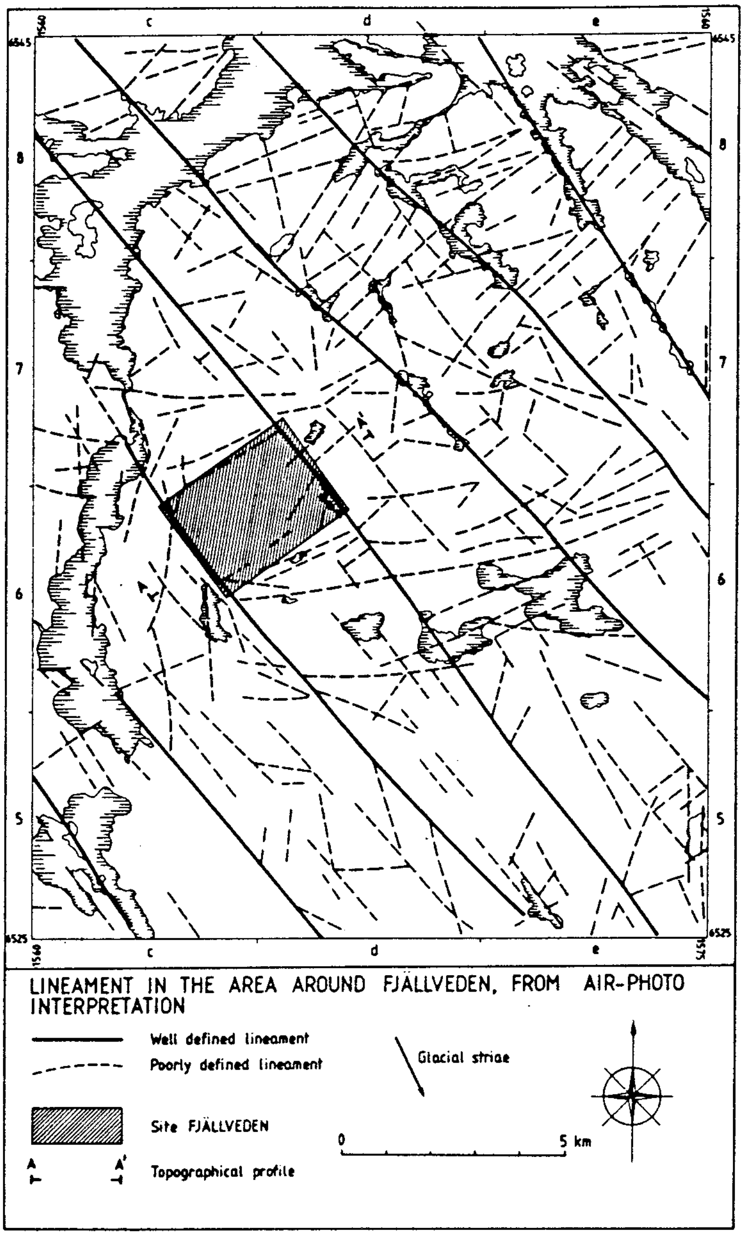


Figure 4-1. Distribution of regional and local Fracture Zones at the Fjällveden study site /Ahlbom et al, 1983/.

decrease in the total fracture frequency with depth. Not only does the total frequency decreases, also the amount of hydraulically conductive fracture frequency, CFF, decreases with depth /Carlsson, Winberg and Rosander, 1984/. Distances between these conductive fractures as determined by borehole investigations, vary from more than 50 m to less than a couple of meters.

In the very near-field of a canister, conductive fractures will constitute the path through which groundwater can be introduced into the buffermaterial surrounding the canisters. This process is taking place during the time when the repository is being open for storage and during the closure phase. After closure and when the natural conditions once more is prevailing, the conductive fractures will be the path-ways to the biosphere for water which may be in contact with the canisters.

One main question for the very near-field and near-field is how to identify conductive fractures so that they can be avoided or if possible sealed by different techniques. Methods to detect fractures having larger extension are developed and used with success in the Stripa Project (e.g. borehole radar).

Detailed investigations of individual fractures reveal that water will move in those parts where the fracture walls are not in contact with each other and which can be said to constitute a channel-pattern along the fracture plane. Precipitate of minerals will further reduce the possibilities for water to move within a fracture plane or a channel. According to investigations at Stripa the water transmitting parts of a fracture plane can be recognized as 15% /Rasmusson and Neretnieks, 1986/. This points to the limiting chances of penetrating the transmitting parts of a conductive fracture by a single borehole. Additional efforts to detect these water transmitting parts of such vital importance for the safety of a repository in fractured crystalline rock are therefore needed.

Natural fractures, conductive or not, have no infinite extent. In order to be able to transport water from the repository level up to the biosphere, a connective system of fractures is needed. In order to address the connectivity issue, the concept of percolation theory is usually applied /Robinson 1982, Wilke et al 1985/. In using such a concept, the fractures are assumed to be the only water conducting parts in the bedrock. The density of fractures having enough intersections to generate an interconnected lattice of flow paths is called the percolation threshold. Below the threshold, the fractures are on the average not connected. A finite set of fractures can be interconnected and creates what is known as a finite cluster. Such clusters are independent and water cannot flow from one to the next. Above the percolation threshold, one cluster becomes infinite and flow can take place throughout the medium. Local clusters may still exist independently of the infinite one. As the density of fractures continues to increase, the chance of finding clusters drops steadily to zero, and the medium becomes more and more pervious.

The threshold density can be related to density and length of fractures. In two-dimensions, Robinson /1982/ gives the following expression for the threshold:

$$[N\langle r^2\rangle]_c = 1.5 \tag{4-1}$$

where N is the density (number of fractures per unit area), r is the length of a fracture, $\langle\ \rangle$ stands for average and subscript c for "critical" meaning that $N\langle r^2\rangle$ must be close to 1.5 at the percolation threshold. The expression assumes that the orientation is purely random, otherwise the threshold value is increased. In three-dimensions, the percolation threshold is assumed to be proportional to the fracture area and the half perimeter /de Marsily, 1985/ according to:

$$[N\langle r^3\rangle]_c = 0.15 \text{ to } 0.3 \tag{4-2}$$

For a rock block containing conductive zones, as a repository block, and surrounded by major fracture zones, the connectivity of each class of water conductive zones has to be considered. For instance, the major fracture zones forming a permeable network can be found above its percolation threshold, while the network of the minor zones and fractures in the rock block is below the threshold. In such case, the flow will take place in the major network and the rock blocks will essentially be impervious. On the contrary, if the major network is below the percolation threshold and the network in the rock blocks above, flow will be governed by the properties of the blocks while the major network will act as local short-circuits.

The relationship between fracture density and mean fracture length given by Eq(4-1) is illustrated in Figure 4-2. The figure also shows data related to investigations at potential sites for a Hard Rock Laboratory in the eastern part of Sweden. When considering the regional scale (lineaments up to 600 m in length) in areas of 2-4 km^2, interconnectivity will be at hand according to Eq(4-1) under the assumption that all lineaments identified are conductive. On the other hand, when considering the scale of 50x50 m^2 the same equation indicates that connectivity is not prevailing in these blocks even when all fractures are assumed to be transmissive.

Thus, such a simplified exercise strongly stress the importance to recognize the conductive fractures within a block and to test these fractures interconnectivity in order to designate the quality of the rock in the near field.

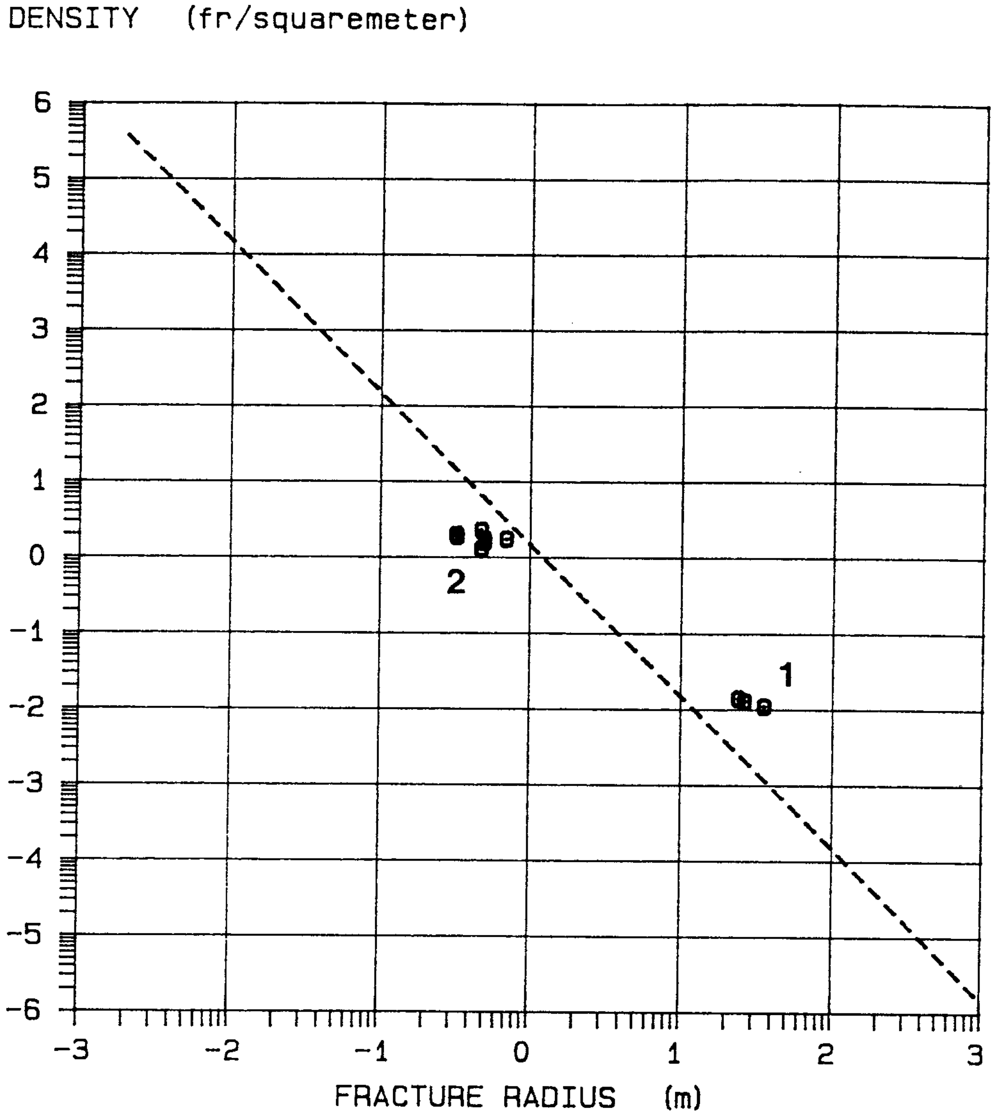


Figure 4-2. Relationship between fracture density and fracture mean length when the percolation threshold is achieved according to Eq(4-1). Data from investigations concerning the Swedish Hard Rock Laboratory are plotted in the diagram /Gustafson et al, 1988, Tirén and Beckholmen, 1988/.

4.2 TRANSPORT CAPACITY

4.2.1 Basic concept

Transport of water around a canister in a deposition hole in a KBS-3 repository will be governed by the backfill properties close to the canister. In the backfill, consisting of highly compacted bentonite, the transport process will be diffusive. Contact between

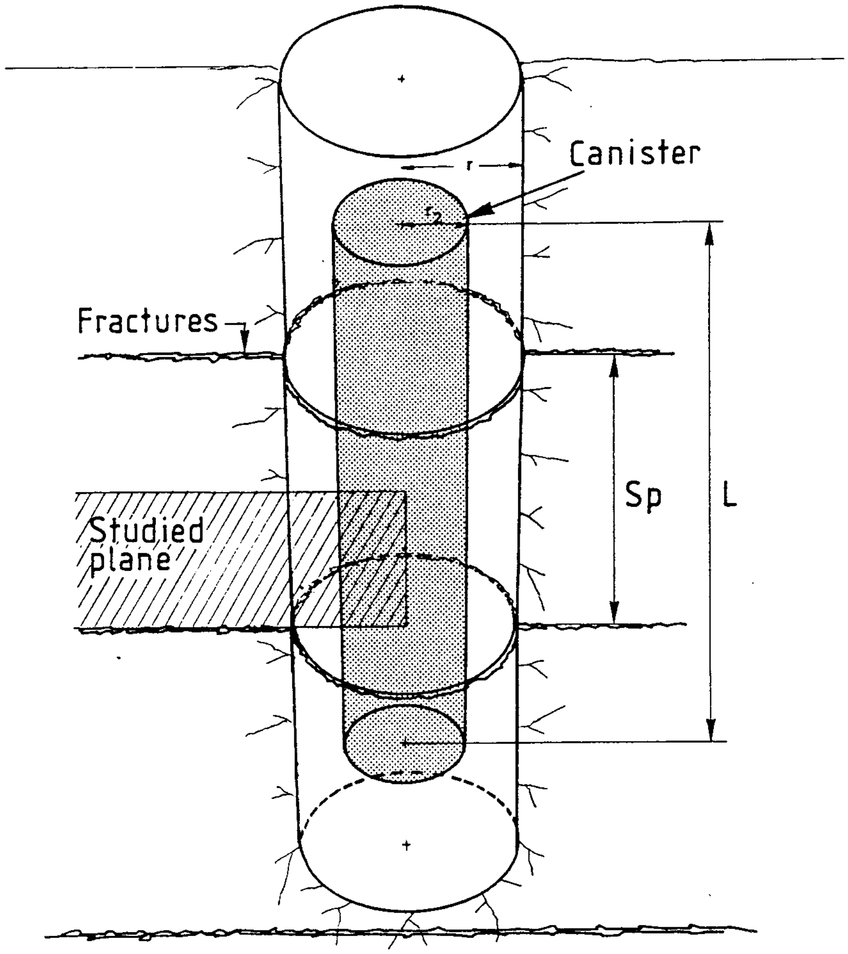


Figure 4-3 Deposition hole with canister. The shaded area is used in the model for reasons of symmetry (modified after Neretnieks, 1986).

the backfill and the bedrock is established by the water transmissive fractures. Here, the width, properties and number of fractures are to a certain degree decisive on the mass transport of species from the canister to the bedrock groundwater.

Neretnieks /1986/ has described the transport of dissolved species in the very near-field of a KBS-3 repository. On a canister surface, the concentration of a studied specie is C_o. The deposition hole is intersected by water transmissive fractures which for simplicity are assumed to have a regular spacing S_p, c.f. Figure 4-3.

A plane through the canister and the backfill is studied, where for the reason of symmetry, its boundaries are set at the centre of the fracture and half the spacing distance as illustrated in Figure 4-4. The mass flow from the canister out through the half width of the

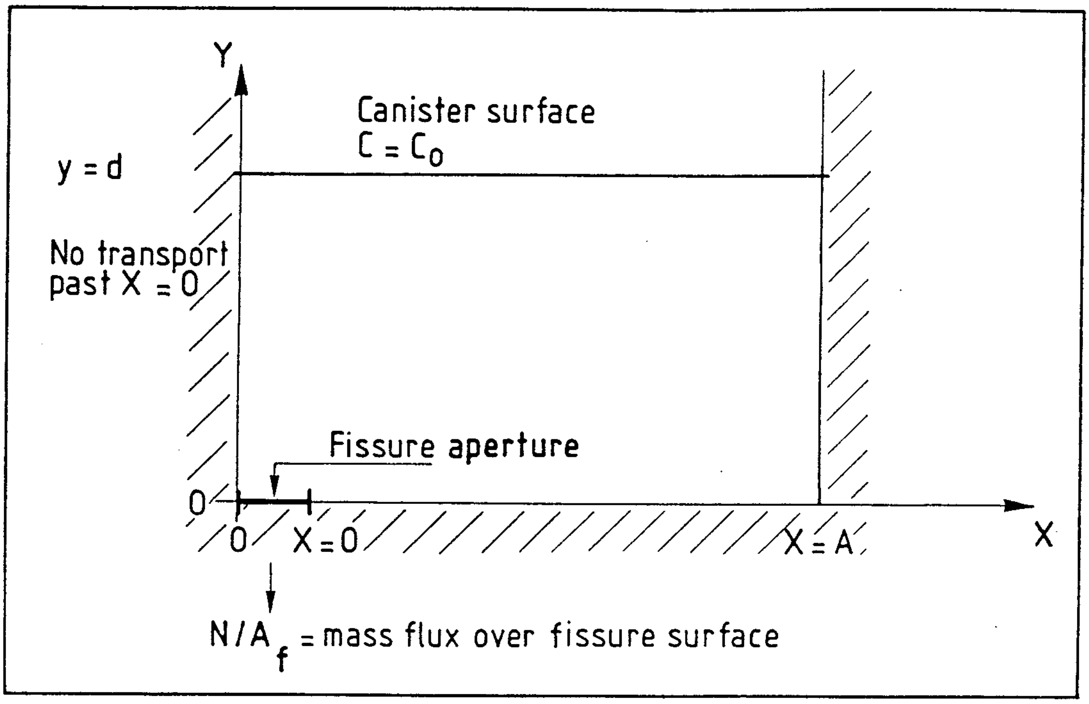


Figure 4-4 The cylindrical sector in Figure 4-3, approximated by a rectangle.

fracture is given by Neretnieks as:

$$N = A_f \frac{C_o - C_\infty}{\dfrac{F(x,0)}{D_e} + \dfrac{1}{K_v}} \qquad (4\text{-}3)$$

where

A_f = Area of fracture opening in contact with the backfill according to Figure 4-4 ($2b\pi r_2$)
b = Half-width of fracture
D_e = Effective diffusivity in backfill (set at 4×10^{-11} m^2/s)
K_v = Equivalent mass transfer coefficient given by:

$$K_v = \frac{2}{\pi} \frac{D_w\, u_f}{r_2} \qquad (4\text{-}4)$$

r_1 = Canister radius
r_2 = Backfill radius (radius of the deposition

borehole)
u_f = Water velocity in a fracture ($=K_f i$)
K_f = Fracture hydraulic conductivity
i = Hydraulic gradient
S_p = Fracture spacing
D_w = Diffusivity in water (set at $2\text{x}10^{-9}$ m^2/s)

F(x,0) is a function given by Neretnieks (1986) expressing the dimensionless concentration according to:

$$F(x,0) = b(1-1.35 \log(2b/S_p) + 1.6 \log((r_2-r_1)2/S_p) \qquad (4\text{-}5)$$

and valid for

$10^{-6}<2b/S_p<10^{-1}$ and $0.03<2(r_2-r_1)/S_p<1$.

By setting $C_\infty=0$ and $C_o=1$, the equivalent mass transport Q_e given as the amount of water emanating from the canister surface and discharging into the fractures intersecting a deposition hole can be calculated as:

$$Q_e = \frac{N\ 2\ L}{S_p} \qquad (4\text{-}6)$$

where L = Length of canister (6 meters)

The hydraulic conductivity of a single fracture having the width 2b can be expressed according to the parallel plate theory as:

$$K_f = \frac{(2b)^2}{12\mu} \qquad (4\text{-}7)$$

where μ = Dynamic viscosity

The equivalent hydraulic conductivity K_e of the bedrock can be expressed as:

$$K_e = \frac{K_f 2b}{S_p} \qquad (4\text{-}8)$$

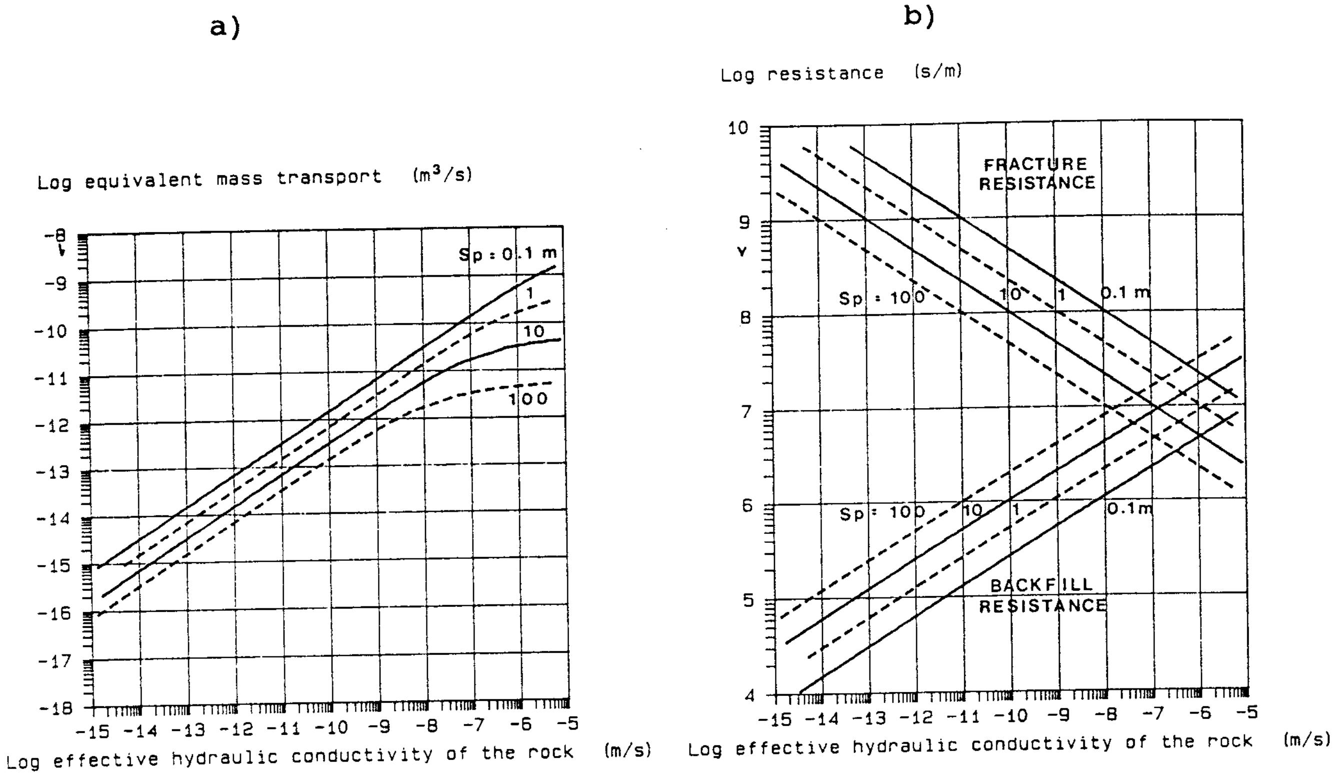


Figure 4-5 a) Equivalent mass transport versus rock mass hydraulic conductivity for different fracture spacing. Hydraulic gradient is 10^{-3} and canister length is 6 meters.

b) Fracture and backfill resistance versus hydraulic conductivity of the rock mass for different fracture spacing. The hydraulic gradient is 10^{-3} and the canister length is 6 meters.

Eq(4-8) assumes equal width of the fractures. Using Eq(4-3) through Eq(4-8), a relationship between the equivalent mass transport and effective hydraulic conductivity of the rock can be obtained. Figure 4-5 illustrates this relationship for various fracture spacing and with a hydraulic gradient of 10^{-3}. For an effective hydraulic conductivity of 10^{-8} m/s and less, a linear relationship is obtained where the resistance to mass transport is governed by the equivalent mass transfer factor into the groundwater passing around the deposition hole (fracture resistance). For higher values of the effective hydraulic conductivity, the backfill resistance expressed as $F(x,0)/D_e$ will be decisive on the equivalent mass transport, c.f. Figure 4-5.

For $K_e=10^{-8}$ m/s and less, an approximative expression can be deduced relating the equivalent mass transport in m^3/s to the effective hydraulic conductivity in m/s when the fracture spacing is 10 m as:

$$Q_e = 1.17x10^{-6} K_e^{0.654} \qquad (4-9)$$

4.2.2 Estimation of equivalent mass transport distribution

Detailed data, from the investigations carried out by SKB, on the hydraulic conductivity determined from testing of short borehole sections, are limited. However, data on the hydraulic conductivity determined by water injection in 2 or 3 meter borehole sections, are available from the investigations at the study site Sternö. Data below a depth of 300 meters from the boreholes Ka1 through Ka4 at Sternö, /Gidlund et al, 1979/, are in this report used to obtain a distribution of the hydraulic conductivity. In addition, hydraulic conductivity data from 3 meter sections below a depth of 210 meters in borehole KLJ1 at the research area Landsjärv, /Hansson, 1988/, is also taken into account.

It is usually assumed that a distribution of hydraulic conductivity data in crystalline rock follows that of a normal distribution when taking the logarithm of the data. Thus in Figure 4-6, data from the boreholes at Sternö (571 data) are plotted in a log-normal distribution diagram. In the same figure, data are plotted from Lansjärv. The straight-line adjusted to the data illustrates a log-normal distribution. Statistical characteristics given in Table 4-2, are used to generate data for further application on calculation of the equivalent mass transport distribution in the rock masses.

Table 4-2. Statistical characteristics of the log-normal distribution of hydraulic conductivity from Sternö and Lansjärv, shown in Figure 4-6. (log = ^{10}log)

Statistical character	Site Sternö	Lansjärv
log mean	-9.28	-9.62
mean (m/s)	$2.4x10^{-10}$	$5.2x10^{-10}$
standard dev.	0.55	1.32

The generated data on hydraulic conductivity for the two sites, following an assumed log-normal distribution are illustrated in Figure 4-7. The equivalent mass transport versus hydraulic conductivity is given in an approximate form in Eq(4-9) for 10 meter fracture

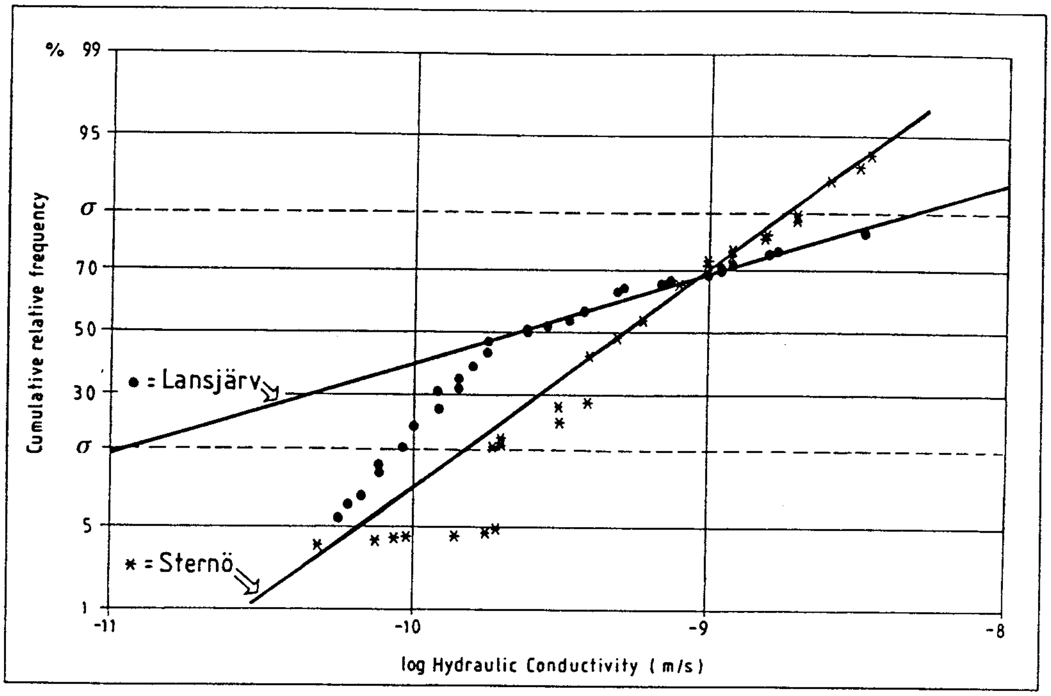


Figure 4-6. Hydraulic conductivity from 2 and 3 meters section in boreholes Ka1, Ka2, Ka3 and Ka4 at Sternö and borehole KLj1 at Lansjärv. Straight-line approximation of a log-normal distribution.

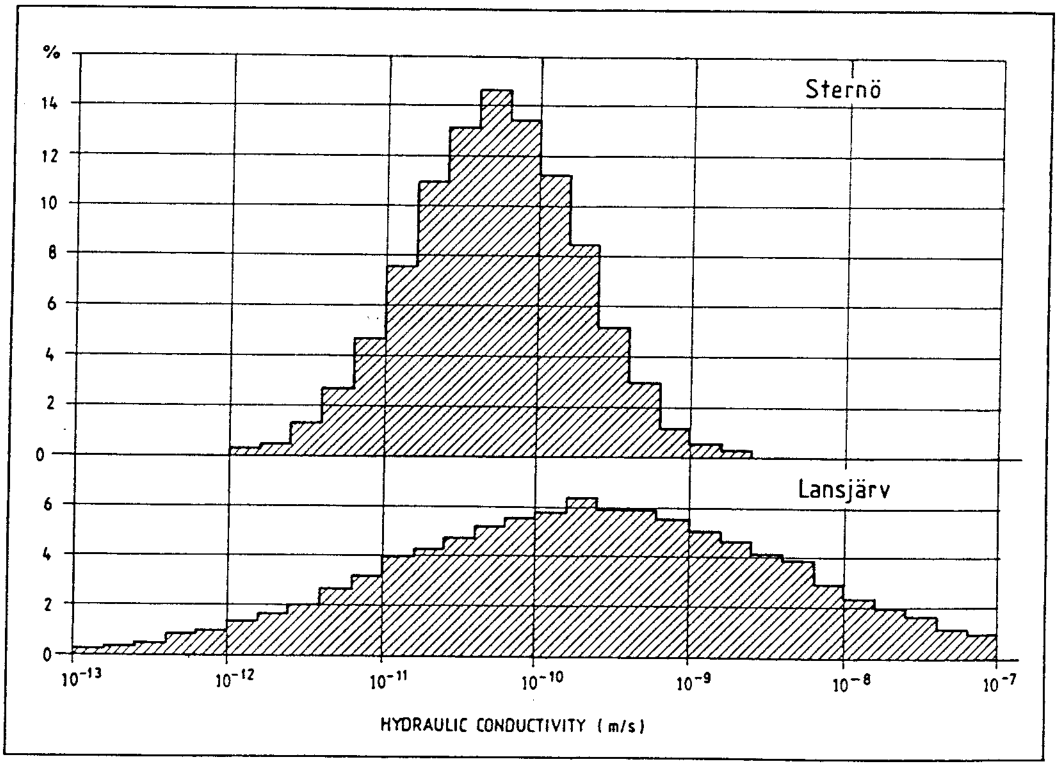


Figure 4-7. The distribution of generated hydraulic conductivity data from the sites Sternö and Lansjärv constructed from the straight-line approximation in Figure 4-6.

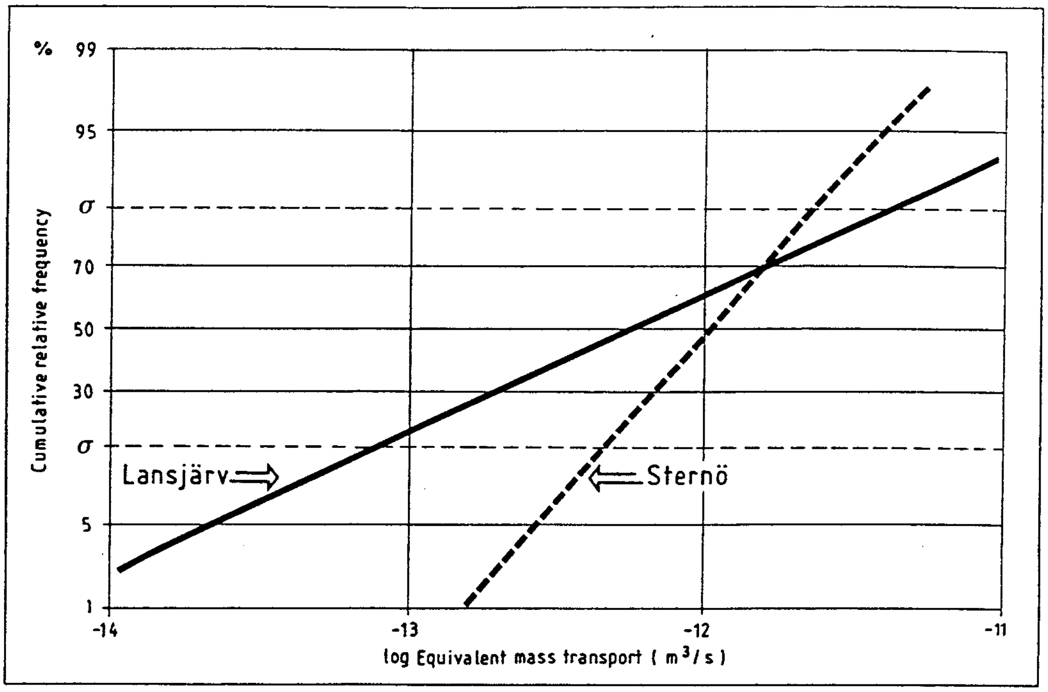


Figure 4-8. Relative cumulative percentage of equivalent mass transport calculated from Eq(4-9) and the generated data presented in Figure 4-7.

spacing. Using this equation on the two distributions of the hydraulic conductivities will give a distribution of the equivalent mass transport in the two rock masses considered, c.f. Figure 4-8.

The very small amount of rock having high equivalent mass transport will contribute with a large part of the total equivalent mass transport. To illustrate this a distribution of the total equivalent mass transport versus hydraulic conductivity is calculated and illustrated in Figure 4-9. The total equivalent mass transport is higher in the Lansjärv site than in the Sternö site based on the generated data and the approach presented in Section 4.2.1. Figure 4-10 illustrates the sum of the equivalent mass transport versus hydraulic conductivity calculated for a deposition hole with a 6 meter canister and the surrounding backfill. For instance, by discarding hydraulic values higher than 10^{-7} m/s, that is 2.4 % of the rock mass at Lansjärv according to Figure 4-6, the total equivalent mass transport will be reduced by 32%. Discarding values down to 10^{-8} m/s (11% of the rock mass) will reduce the total equivalent mass transport by 67% at Lansjärv. The same discarding at Sternö, comprising 1% of the bedrock, will hardly have any effect on the total equivalent mass transport. It should be remembered that

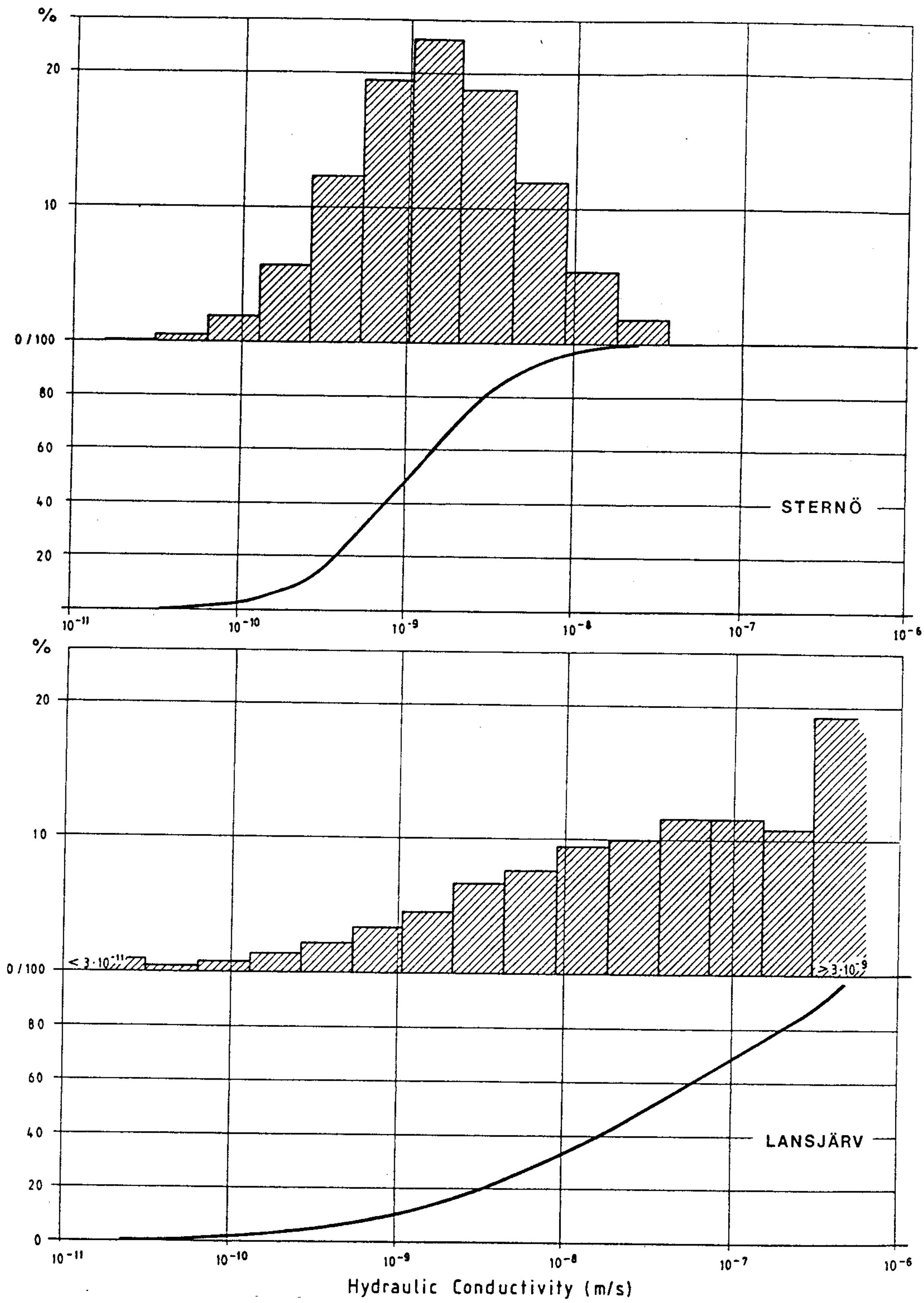


Figure 4-9. Percentage of total equivalent mass transport versus generated hydraulic conductivity data for Sternö and Lansjärv.

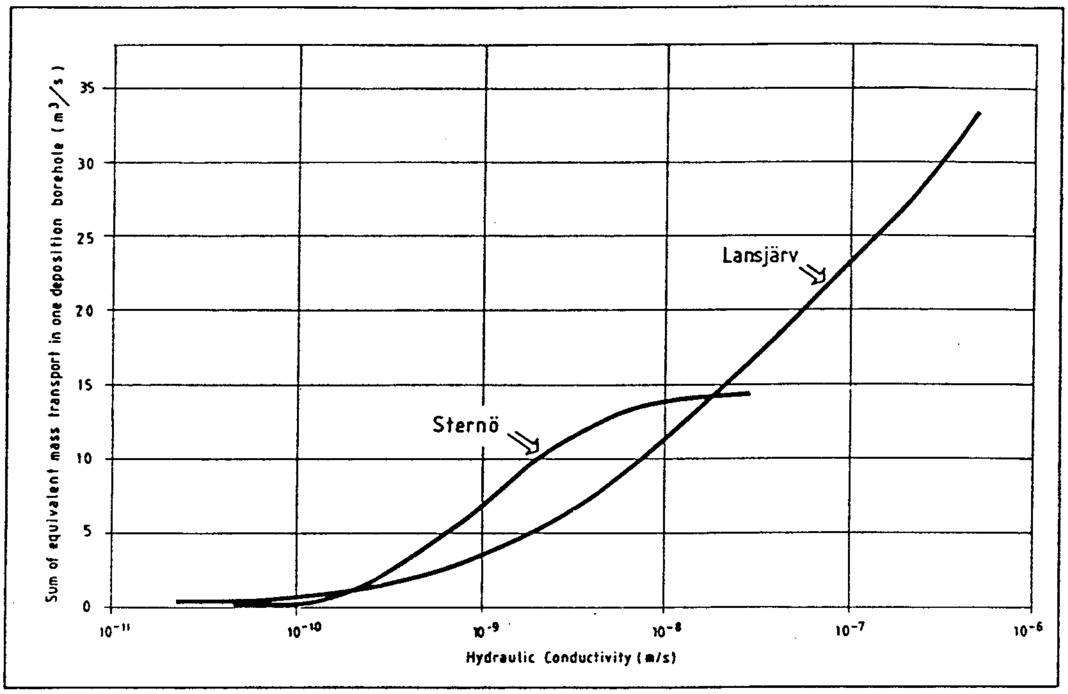


Figure 4-10. Total equivalent mass transport versus generated hydraulic conductivity data for Sternö and Lansjärv.

the values presented are generated and that there are assumptions behind the approaches that require the bedrock to behave in a 'proper' manner, e.g. the fracture spacing is assumed to be constant regardless of the hydraulic conductivity.

4.3 ENGINEERED BARRIERS

The isolating function of clay-based backfills or buffers in repositories is very much dependent on the temperature and the composition of the groundwater, which is in turn at least partly controlled by the mineral composition, particularly the content of carbonates and potassiumbearing constituents. The temperature is of major importance for the chemical stability of smectite, which is the major component of bentonite clay, while the calcium and magnesium content of the water affects its physico/chemical and reological properties. Access to potassium in conjunction with or after exposure to temperatures exceeding about 100^{o}C tends to convert the smectite to illite.

The physical state of the rock and its response to stress changes is of equal or even greater importance as demonstrated by the influence of the fracture apertures on the physical stability of the barriers, i.e. loss of clay from the buffers and erosion of smectite-poor backfills caused by rapid water flow in wide rock fractures at the rock/backfill interface. Clearly, the hydraulic gradients that are set up in the course of the excavation and that prevail after closing the repository, are determinants of the physical stability of such backfills.

The most critical conditions are expected where the stress fields and the rock structure are anisotropic since excavation of tunnels, holes etc, strongly affects the degree of fracturing of such rock. Construction of repositories according to different concepts affect the rock in different ways, which suggests that the structural constitution of the rock should govern the final choice of repository concept.

It is concluded that both the mineral constitution, the structural state, and the sensitivity to stress alteration caused by excavation, can be considered as measures of the rock quality designation.

The major types of engineered barriers discussed here are geometrically well defined, rather small volumes of tightly fitting blocks of highly compacted Na bentonite, and large volumes of mixtures of Na bentonite powder and suitably graded silt/sand/ gravel ballast material, applied and compacted on site.

The sealing function of engineered barriers depends on their chemical and physico/chemical integrity as determined by the interaction with the surrounding rock/water system. Also, the physical interaction between clay barriers and rock is of great significance, the establishment of a tight, interfingered contact being required. It is particulary strong when the swelling potential of the barrier is strong. This potential also determines to what extent fractures deeper into the rock can be closed, what the risk is of loosing expanding clay into fractures, and whether internal erosion can take place in backfills.

4.3.1 Chemical interaction

The longevity of the clay barriers primarily depends on the temperature and the pH. Temperatures higher than 100-150^{o}C cause dissolution of silica and aluminum and partly irreversible microstructural changes. On cooling, i.e. at the end of the heating cycle, precipitation of dissolved material takes place and this may cause slight cementation. The heat-induced release of silica yields lattice charge changes, which causes preferential uptake of potassium and conversion to illite. At such temperatures, the potassium content in the porewater is thus of significance, since it determines how much of the smectite that will be converted to illite form and how rapidly this alteration is. Usually, the potassium

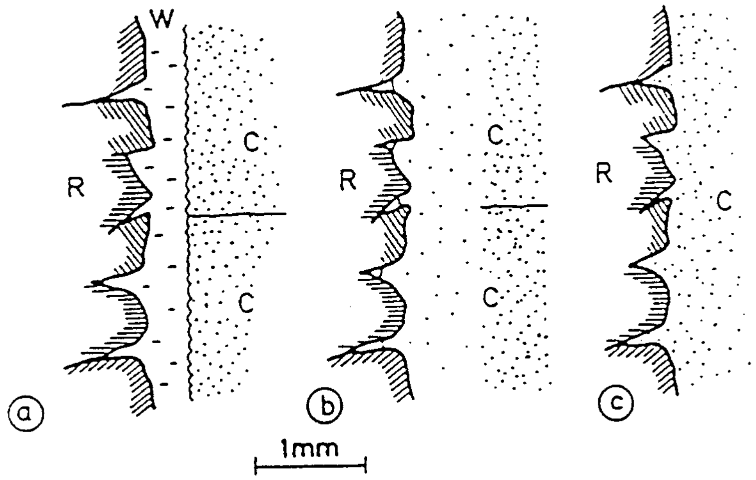


Figure 4-11 Schematic picture of clay penetration into the rock matrix and formation of a tight clay/rock contact. a) Clay blocks in position, starting point. b) Early stage of expansion. c) Final stage with integrated clay (C) and crystal matrix (R). W represents water, C bentonite, and R rock.

concentration is low and the illitization can be minimized if the surrounding rock is poor in K-bearing minerals, such as microcline, muscovite and biotite. For bentonite/ballast backfills the latter component should be quarts.

For dry bulk densities lower than about 1,5 t/m^3 the salt content (Na, Ca and Mg) in the porewater controls the microstructure of the clay and thereby its hydraulic conductivity and swelling potential.

4.3.2 Swelling potential

The swelling potential of the buffers and backfills has two practically importants effects on the rock: Firstly, the higher the density and the smectite content, the stronger are the expandability and self-healing power, and therefore the fitting between the buffer and the rock boundary, c.f. Figure 4-11. Secondly, the swelling pressure exerted on the rock is known to affect the aperture of fractures that are subparallel to the rock/buffer contact. This effect is insignificant for smectite-poor backfills but very substantial for pure Na bentonite of high density as manifested by the high swelling pressures at high densities (Table 4-2). Thus, buffers of the latter type do not have to be applied and fitted to the rock with much care because they take up water, expand and adjust themselves to become

Table 4-2 Swelling pressures p_s of typical buffers and backfills (Fresh-water conditions)

Material	Dry density t/m^3	Bulk density t/m^3	p_s MPa
Highly compacted Na bentonite	1.9	2.20	40.00
	1.8	2.13	20.00
	1.7	2.07	10.00
	1.5	1.95	3.50
	1.3	1.82	0.70
30 % Na bentonite + 70 % graded ballast	1.8	2.13	1.00
	1.7	2.07	0.50
	1.6	2.01	0.25
10 % Na bentonite + 90 % graded ballast	1.9	2.20	0.40
	1.8	2.13	0.20
	1.7	2.07	0.10
	1.6	2.01	0.05

conformous and integrated with the rock boundary.

It is essential to realize that blasting and stress relief that are associated with the excavation of tunnels and shafts increase the rock "porosity" through expansion and propagation of critically oriented fractures. This is particularly important in rock with strongly anisotropic primary stress fields and in shaly and slaty rock. Such widening is not recoverable but it has been demonstrated that highly compacted Na bentonite can at least partly restore the rock porosity, provided that the excavation-induced fracturing and fracture-widening is small. This is the case for boreholes and fullface-drilled tunnels and shafts, while the "repair ability" is expected to be less important for blasted excavations. Also, it is a scale effect, meaning that the rock is disturbed at a considerable distance from large excavations.

4.3.3 Self-penetration

Theoretical deductions as well as practical experience indicate that self-penetration of pure Na-bentonite from water-saturated dense blocks into fractures takes place at a rate that is controlled by the clay density and the fracture apertures. Figure 4-12 illustrates the phenomenon while Table 4-3 gives calculated values of the penetration depth. Bentonite/ballast mixtures do hardly have any penetration capacity at all, except for the case of fractures with apertures exceeding 0.5 mm.

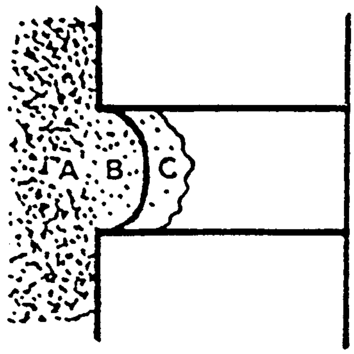


Figure 4-12 Schematic picture of physical state of clay penetrated into wider slots. (A) very stiff bentonite, (B) fairly stiff region, (C) very soft region.

It is obvious from Table 4-3 that the penetration is negligible in fractures which are 0.1-0.2 mm wide but that it is considerable when the fracture aperture is more than one millimetre. The consequence of this, with respect to the hydraulic conductivity of the rock and to the risk of loosing significant amounts of bentonite, will be discussed later in this report.

Table 4-3 Predicted penetration depth in meters of 100 % Na bentonite. Average dry density 1.2 t/m^3.

Joint aperture mm	10y	10^2y	10^3y	10^4y	10^5y	10^6y
0.1	$5.2x10^{-4}$	10^{-3}	$1.5x10^{-3}$	$2.0x10^{-3}$	$2.5x10^{-3}$	$3.0x10^{-3}$
0.2	$2.0x10^{-4}$	$4.0x10^{-3}$	$6.0x10^{-3}$	$8.0x10^{-3}$	10^{-2}	$1.2x10^{-2}$
0.3	$4.7x10^{-3}$	$9.0x10^{-3}$	$1.4x10^{-2}$	$1.8x10^{-2}$	$2.3x10^{-2}$	$2.7x10^{-2}$
0.4	$8.3x10^{-3}$	$1.6x10^{-2}$	$2.4x10^{-2}$	$3.2x10^{-2}$	$4.0x10^{-2}$	$4.8x10^{-2}$
0.5	$1.3x10^{-2}$	$2.5x10^{-2}$	$3.8x10^{-2}$	$5.0x10^{-2}$	$6.3x10^{-2}$	$7.5x10^{-2}$
1.0	$5.2x10^{-2}$	10^{-1}	$1.5x10^{-1}$	$2.0x10^{-1}$	$2.5x10^{-1}$	$3.0x10^{-1}$

4.3.4 Sealing of fractures in rock

Sealing of rock fractures by grouting can be made to redirect groundwater flow from the close vicinity of deposition holes and repository tunnels and shafts, as well as to delay diffusion of corroding agents and radionuclides in the near-field region. The efficiency of such grouting is demonstrated by the fact that if the gross permeability of one cubic meter of rock is determined by a few 100 micron wide fractures, yielding a hydraulic conductivity of 10^{-7} to 10^{-8} m/s, the net rock permeability can be reduced to less than 10^{-11} m/s if the grout has a hydraulic conductivity of 10^{-8} m/s. This condition is fulfilled by montmorillonite-based clay grouts and by cement grouts, provided that they can be injected to a depth of one meter into the fractures. As to the resistance to diffusion, estimates have shown that if a fracture extending from a deposition hole can be sealed with bentonite clay of moderate density to a distance of 10 cm, the diffusion resistance of the near-field region will be increased by ten times. One concludes from this that also moderately effective grouting can be of significant value in the retardation of canister corrosion and radionuclide migration.

The sealing power of grouts can be considered in safety analyses only on the following conditions:

- The groutability, i.e. the penetrability of grouts into fractures, must be sufficient and the penetration depth needs to be quantified.

- The grouts must be sufficiently resistant to erosion and piping as well as to chemical alteration.

The first point, concerning the fluidity, has to do with the water content and the applied technique for grouting. A general rule is that the water content of the grout must be 1.3 to 1.7 times the Atterberg limit, which can be suitable altered by using salt water. As to the grouting technique it is essential to know that ordinary injection using a constant static pressure only brings in very soft grouts into fractures with an aperture of 30-100 microns, while the superposition of oscillatory vibrations with a suitable frequency may reduce the fluid resistance significantly and allow for penetration of rather dense grouts several decimeters into such narrow fractures. The short pressure spikes greatly improves the penetrability for certain types of clay and cement grouts.

A sufficient sealing ability and required resistivity to erosion and piping calls for a relatively high density. For clay-based grouts this can be achieved by using sodium chloride solutions (0.5-2 %) in the preparation of the grouts, while organic superplasticizers must be used in the preparation of cement grouts. Superplasticizers serve to act as lubricants and to delay the hydration process sufficiently much to allow for bringing the grout into the fractures before the gel strength and flow resistance increases due to the chemical processes.

As concerns the chemical stability, the presently favored

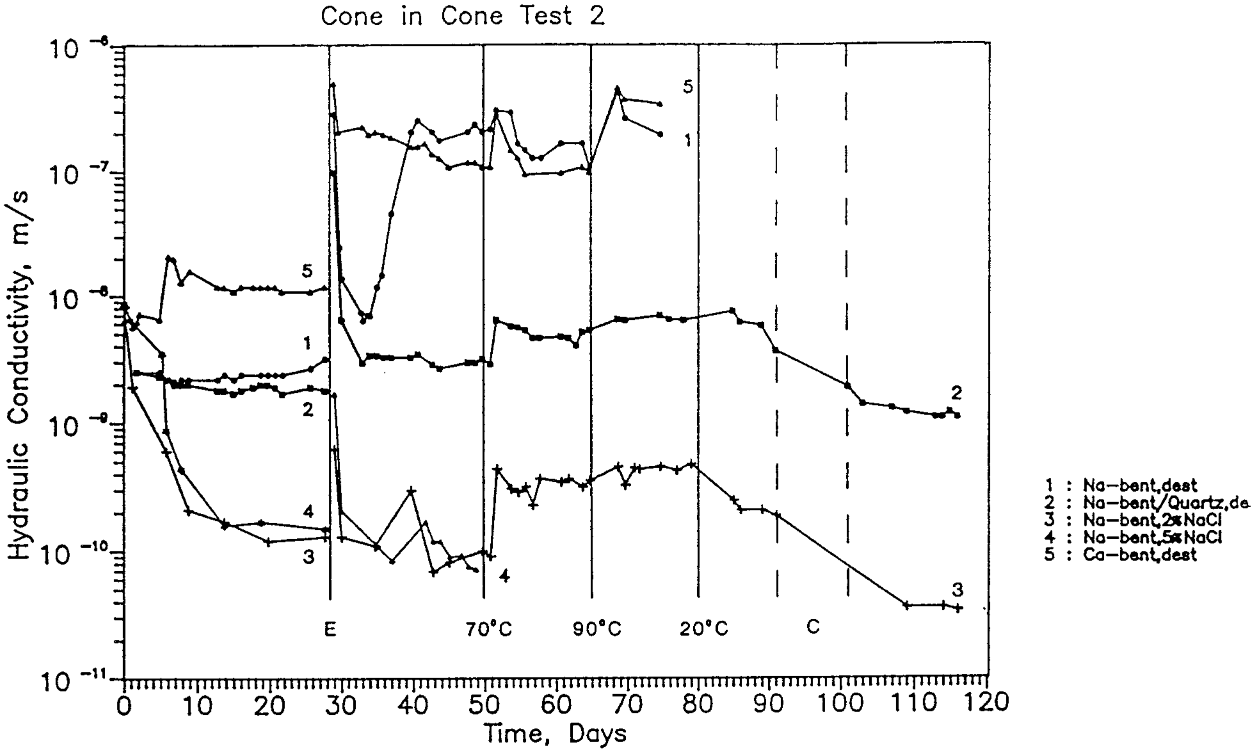


Figure 4-13 Hydraulic conductivity of clay grouts as influenced by 30 % fracture expansion (E), heating to 70 and 90°C, and finally compression to the initial fracture aperture. This cycle represents the different stages that a clay grout in a fracture may undergo in the near-field region.

idea is that smectitic clay gels only undergo partially recoverable microstructural changes at temperatures below about 90°C, while chemical alteration takes place at higher temperatures. Heating to 130-150°C initiates partial dissolution and crystal lattice changes leading to cementation and time-dependent alteration to non-expanding clay minerals. These changes tend to reduce but not eliminate the sealing ability of clay grouts. The longevity of cement is less well known but it is felt that the sealing function may well be preserved over many thousands of years, provided that the density remains constant and that the solubility is reduced by adding silica fume to the cement material.

The use of grouts for increasing the near-field resistance to flow and diffusion needs to be planned such that expected changes in aperture due to thermo-mechanical processes, i.e. heat-induced compression and subsequent expansion of grouted fractures, are considered. Current research indicates that the flexibility of smectitic clay grouts allows for 30% compression/expansion with simultaneous increase in temperature to 90°C without any loss in hydraulic conductivity, c.f. Figure 4-13. Although cement can also be considered for the present purpose the major disadvantage in using this type of material is the expected lack in long-term chemical stability and the brittle, i.e. non-flexible behaviour. Still, the use of superplasticizers offers a possibility of using cement with w/c about 0.35 which has potential to expand. This is because the water content is then lower than the stochiometrically required amount of water required for complete hydration. Hence, widening of the aperture of cement-grouted fractures, particularly in conjunction with shear, will reactivate the cement and cause self-healing through expansion.

Grouting of natural fracture zones and the "disturbed" zones off the near-field region, can be made by use of either clayey or cement-based materials. The choice should be based on the required flexibility and longevity and no definite criteria or rules have been set in this country or elsewhere for proper selection of grout composition or grouting strategy. In principle, however, cements should be preferred for this type of grouting since the fracture fillings will temporarily be exposed to high piezometric pressures and hydraulic gradients, for which cement is assumed to be more resistant than clay gels although the difference may not be substantial.

5 STRATEGY IN ROCK QUALITY DESIGNATION OF A KBS-3 REPOSITORY

5.1 SITING STRATEGY

A repository designed according to the KBS-3 principle will be located at a depth around 500 meters below the ground surface. The repository will be located in a block of rock which is surrounded by major fracture zones identified by ground surface and borehole investigations. The bedrock within the defined rock block should be favourable for repository construction and should also serve as a geological barrier for nuclide migration from the repository to the biosphere. In the safety analyses, no credit is taken for retardation of nuclides along the major fracture zones.

The area needed for a repository according to the KBS-3 system is about 1 km^2. Including a respect distance of 100 meter to bounding major fracture zones, an area of about 1.4 km^2 is needed. Such an area should be contained in one rock block if possible. If this is not possible, additional rock blocks are required or a repository in multi-storeys has to be considered. When additional blocks are considered, buffer zones of good quality rock have to be assigned when passing from one block to the next through a major fracture zone.

The geometric and hydraulic properties of the major fracture zones are investigated and quality designated during the site investigation phase and will not be considered in the current report. It should be remembered that the investigations in the site investigation program are performed in a limited number of surface drilled boreholes each covering a minor part of the entire site.

5.2 CONSTRUCTION SEQUENCE

In the construction sequence for a KBS-3 repository there are details not fully decided upon yet like the use of shaft or spiral ramp for access to the repository within or outside the identified rock block. However, once down at the repository level, the actual construction of the repository will start by excavation of a central or main tunnel from which a system of storage tunnels for deposition holes is constructed. The distance between shafts and tunnels to the nearest major fracture zones should be greater than 100 meters (respect distance).

The storage tunnels will be excavated from the central tunnel. Their spacing is set at 30 m for a single-storey repository. In a final sequence of construction, and probably parallel to the construction of the storage tunnels, the deposition boreholes will be drilled.

In the following strategy descriptions, the construction phase will be divided into four main stages as:

- Excavation of access shaft and access tunnels
- Excavation of central (main) tunnel
- Excavation of storage tunnels
- Drilling of deposition holes

During each stage, the strategy is outlined regarding techniques possible to use for rock quality designation of the hydrogeological conditions and properties important for the long term safety of a final repository for spent fuel.

The concepts of far-field, near-field and very near-field conditions are commonly used when discussing safety properties of a repository. Various definitions of the concepts exist and Figure 5-1 illustrates the use of the three terms. The far-field concept is not used in the current report since designation of rock quality in the scale of far-field is considered in the site investigation program.

The term near-field is described in the KBS-3 system as the area around a canister where the repository and its components directly influence the dispersion of nuclides after the canister has been penetrated. This influence can be of chemical, hydrological or mechanical nature. The extent of the near-field varies in time and cannot be defined exactly, but it can in practice be regarded as extending up to a distance of around 10 m from the canister /KBS-3, 1983/. In the current report this description is valid for the term near-field, but extended to include all rock between the deposition holes and the storage tunnels.

The term very near-field is assigned for the deposition holes with their close surrounding fractures. It also includes the buffer material introduced into the deposition holes (compacted bentonite etc).

5.3 ROCK QUALITY DESIGNATION DURING CONSTRUCTION OF THE ACCESS SHAFT

A first layout of the access shaft to a repository is made on the results from the site investigations. The construction of the shaft itself can be regarded as a final validation of the conceptual model set up from

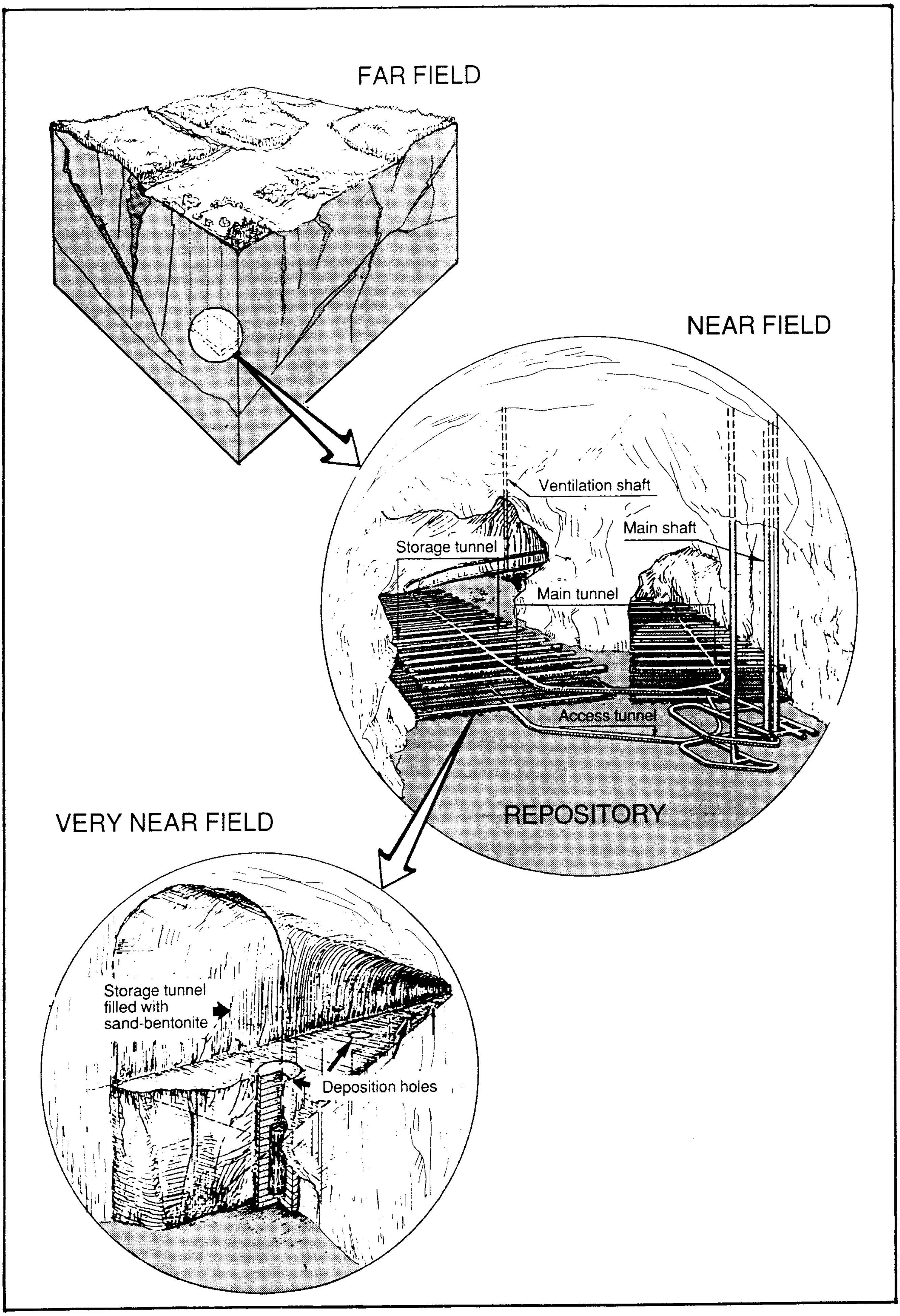


Figure 5 - 1. The terms far-field, near-field and very near-field used in the current report.

the result obtained from the site investigation program. After shaft sinking, lay-out options on tunnel-system design in the repository exist.

The shaft sinking at the underground laboratory, URL, in Canada can partly be regarded as a validation exercise and the results and techniques used are valuable also as a designation of the rock quality during the shaft sinking period /Simmons, 1988/.

During construction of an access shaft, measures to increase the rock quality by e.g. grouting, will probably be carried out. The access shaft is the prime connection between the repository to the biosphere and any adverse condition caused by the construction of the shaft has to be considered. Primarily, any zone of increased hydraulic conductivity outside the shaft has to be counteracted since they can cause short-circuit with water transmitting zones within the bedrock. Strategically sited curtains or slots of sealing material have to be applied to prevent such water connection between the biosphere and the repository.

The conceptual model set up from the site investigation period will comprise the prediction of groundwater inflow to the shaft and head changes around the shaft during construction. The shaft is in comparison to a single borehole of such a large diameter that fracture zones penetrated by the shaft have a long interception line. Such interception will probably be long enough to overcome effects of channeling along the fracture plane. Thus, if the fracture is conductive this would turn up at the interception. However stress redistribution might close fractures also where these are conductive, close to the shaft wall.

The access shaft to the repository can be located according to two different concepts:

1. So that the actual repository rock block can be hydraulically characterized.

2. So that minimum impact on the groundwater situation can be encountered.

According to the first concept, the shaft should be sited in a central hydraulic position with reference to the repository and should influence the rock block for the actual repository. Following this concept, the most suitable level for a repository should be selected after the completion of the shaft excavation.

In the second concept, the shaft is located in a separate rock block far away from the block assigned for the actual repository. The impact from the shaft on the hydraulic conditions in the repository block is then considerably reduced. Access tunnels of longer distances are needed than in the first concept to penetrate into the actual repository block.

Before starting the shaft sinking, vertical pilot boreholes should be drilled at the shaft site. Cross-hole measurements (geophysical and hydraulic) will disclose where fractures and fracture zones are to be found during the later shaft sinking. The boreholes should be sited one to two meters in the surrounding rock from the periphery of the shaft. One borehole should also be sited along the centre line of the shaft and in total approximately five boreholes will be needed. The centre line borehole should permit water to be withdrawn while observing head changes in the surrounding boreholes equipped with a multi-packer system. The boreholes should be investigated regarding occurrence of transmissive fractures or zones. The locations should later be compared to the results of inflow mapping and measurements during construction.

The first campaign of testing will result in an improved knowledge of the rock quality in the immediate surrounding of the shaft. Based on the conceptual model set up for this part of the bedrock, additional boreholes at increasing distances from the planned shaft might be drilled, tested and equipped. Such a field of boreholes makes it possible to follow head changes and locate transmissive parts in the bedrock during the actual shaft construction.

During shaft sinking, water inflow to the shaft will be monitored as a function of time and depth of the shaft. These measurements could have a varying degree of ambition. By detailed monitoring of inflow and inflow points, data for statistical treatment and for quality designation of the close surrounding of the shaft will be obtained. In combination with head measurements in surrounding boreholes, designation will also be possible over longer distances.

The shaft itself will have an impact on the close surrounding bedrock due to blasting damages and rock stress redistribution. The skin-zone developed around the shaft will affect the water inflow and the head distribution. Special investigations have to be assigned for the description of the skin-zone. These investigations include hydraulic testing in boreholes within the skin-zone before and after construction of the shaft, head profile measurements from the shaft into the bedrock, geophysical measurements, etc.

During and after shaft sinking, a groundwater situation affected by the shaft will be obtained. The configuration of the head around the shaft is a function of among other things, the hydraulic properties of the rock. A close examination of the head situation will thus reveal in which direction the fractures are more conductive and an anisotropic condition might be established. The effect will be used in an improved and final lay-out of tunnels for the repository.

5.4 ROCK QUALITY DESIGNATION OF ACCESS TUNNEL AND CENTRAL TUNNEL

5.4.1 Access tunnel and decision on central tunnel location

It is usually said that the concept of a KBS-3 repository is flexible regarding the detailed siting and lay-out of the tunnels and deposition holes. The sensus of this is that a variety of possibilities and improvements are recognized after the first lay-out of the repository. The flexibility is significant at two main occasions during the repository construction. The first occasion is before the construction of the central (main) tunnel. The second main occasion is when siting the individual deposition boreholes. At both these occasions, the layout of elements in the repository is possible to change, neglect or improve. The criteria needed for decision on such steps are however not completely defined. However, techniques to designate rock quality are available and should be fully used in order to obtain background data for further decisions if needed.

The adverse features in the host rock mass are not mapped in detail before start of the construction of the central tunnel. The designation performed during the construction of the access shaft might also be of limited use when the access shaft is located outside the rock block assigned for the repository. Only in the case where the access is in the same block as the repository would valuable information on the hydraulic quality of the host rock be obtained.

From the shaft to the area of the repository, access tunnels are needed. These tunnels will be designed and constructed in such a way that their impact on the repository would be as little as possible and at the same time as much information as possible should be obtained on the groundwater conditions in the host rock.

In cases where no final decision is taken on which rock block to utilize for a repository, pilot boreholes, or access tunnels to these blocks might be required before the final selection is done. It is however expected that the block selection is done based on results from the site investigations performed from the ground surface before the shaft siting.

From the access tunnels, a central tunnel will start.

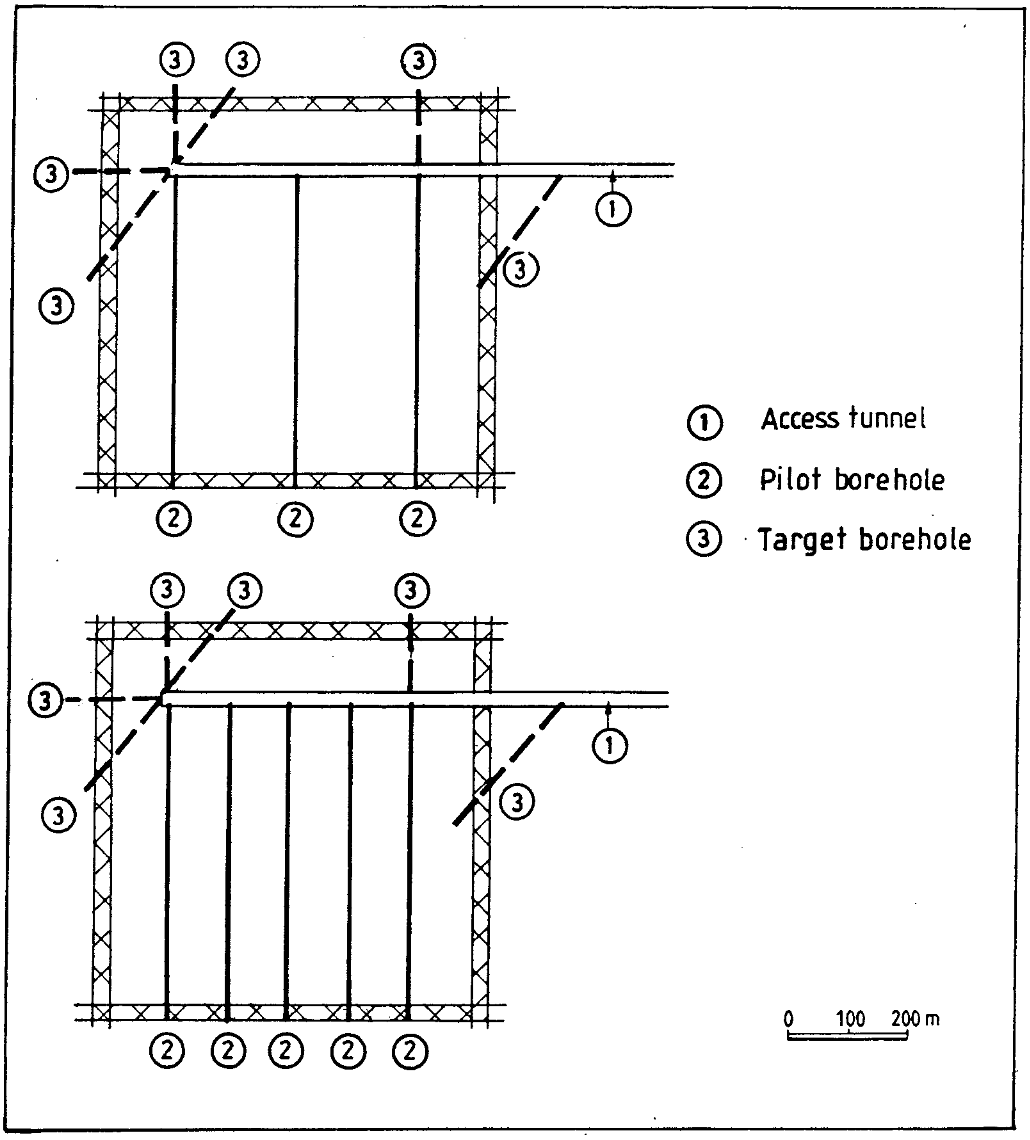


Figure 5-2. Pilot boreholes for the central tunnel laid-out from an access tunnel.

This tunnel can be perpendicular to the access tunnel thus utilizing the latter tunnel as a prime tunnel from which parallel boreholes (pilot boreholes) can be drilled to investigate the area designed for the repository, c.f. Figure 5-2.

The horizontal pilot boreholes will be drilled parallel to the designed direction of the central tunnel. The distance between these boreholes should if possible not be shorter than the length of the storage tunnels. These tunnels, oriented perpendicular to the central tunnel should only be intersected by pilot boreholes at the very end of the tunnels. The distance between the pilot boreholes might be larger than 200 m which is to long for application of crosshole measurements. A pilot borehole spacing in the order of 100 m will on the other hand allow crosshole measurements but will introduce artificial channels between the future storage tunnels in the repository, c.f. Figure 5-2. Techniques to seal these boreholes exist and by using a closer pilot borehole spacing, more detailed information are possible to obtain prior to the lay-out and construction of the central and the storage tunnels.

In the basic concept for a repository lay-out, 0.5-1 km spacing between the regional or major fracture zones are assumed. When using a whole block for a repository with one central tunnel, the length of the storage tunnels will be up to 150-400 m leaving 100 m respect distance as a control zone to the bounding major fracture zones.

Drilling of the pilot boreholes followed by crosshole measurements and hydraulic testing will give data for a first conceptual model of transmissive zones in the area. From the access tunnels, boreholes targeted for the major bounding fracture zones will be drilled and tested. In sections in these boreholes where the major bounding fracture zones are penetrated, non-sorptive tracers are injected. These tracers are specific for each borehole section. During the continuous constructions, occurrences of these tracers in pilot boreholes or on tunnel walls will be recorded and analyzed. The results and conceptual model set up, will be the background for a decision on the location of the central tunnel. The tunnel would preferably be located along one of the pilot boreholes.

5.4.2 Central tunnel

The rock quality designation before and during the construction of the central tunnel aims primarily to describe the hydraulic properties and conditions of the host rock and to locate and describe adverse hydraulic features. Secondary, the designation aims to describe improvements made to reinforce the bedrock by e.g. grouting, injections etc. Adverse hydraulic features to consider are fracture zones which are water conducting and single transmissive fractures with large extensions. Individual fractures which are slightly water yielding should not be neglected, and their extent and interconnectivity should, where possible, be followed up.

During the construction of the central tunnel, different techniques in rock quality designation are possible to use. Horizontal pilot boreholes from the tunnel front can be used and the cores taken could rapidly give a first illustration of the rock quality ahead of the tunnel. The pilot borehole prior to the layout of the central tunnel, drilled along the centre

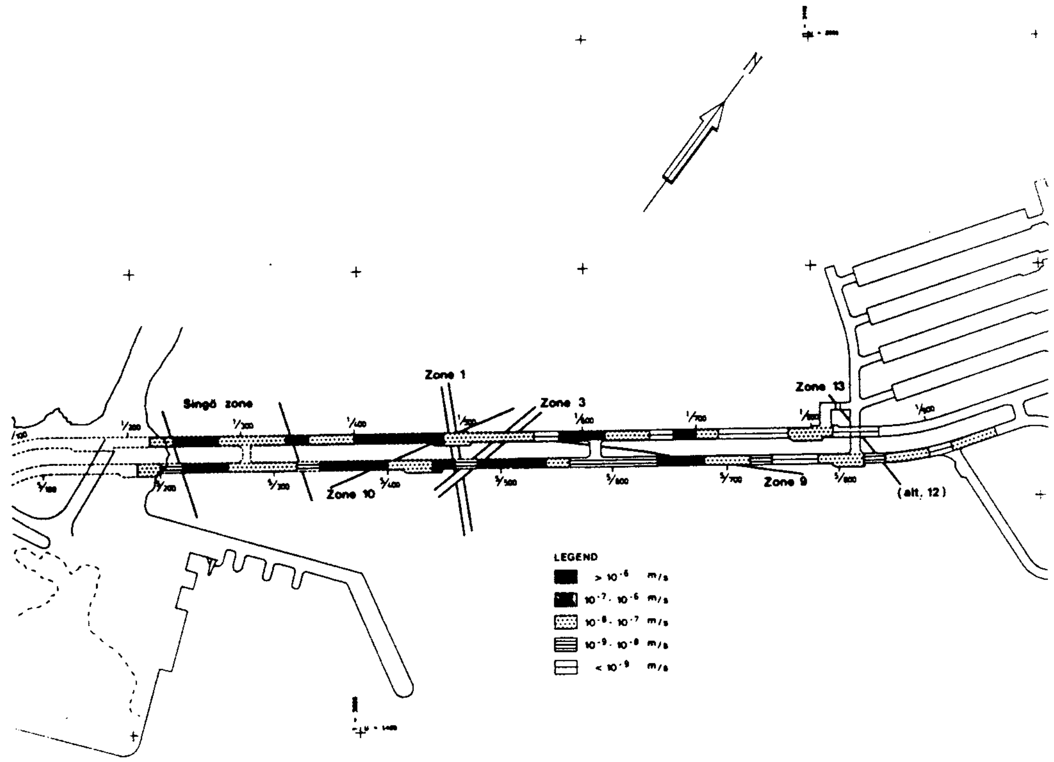


Figure 5-3. Gross hydraulic conductivity of the bedrock at the access tunnels to SFR calculated from inflow measurements in 20 m long horizontal pilot boreholes during the tunnel construction /Carlsson et al, 1985/.

line of the intended tunnel as described above, is sealed by grouting or plugging before the tunnel excavation starts. The outflow of water in the new pilot boreholes will be monitored and sampled after which the groundwater head will be measured in the boreholes. The testing procedure could be time consuming when the level of ambition is highly set. In general, the head will be monitored as an average head for the actual pilot borehole. In Stripa, it has been possible to drill horizontal pilot boreholes 300 meter in length which then were equipped and tested during long time /Carlsson and Olsson, 1985/. At SFR, a lower level of ambition was used which included only inflow measurements from, in average, three boreholes at the tunnel front. The length of these boreholes were about 20 meters and no head measurements were performed. However, assuming a full groundwater head in the bedrock, a preliminary hydraulic conductivity value from the entire borehole was possible to calculate and a preliminary map of the gross hydraulic conductivity was obtained, as illustrated in Figure 5-3.

Pilot boreholes during the construction of the central tunnel should have a length of 50-100 meters. Where high inflow are monitored, reliable head measurements should be carried out together with borehole radar measurements. The measurements together with the core logging will describe the features to be encountered in the continuing construction and the hydraulic conditions and properties of these features. Decision on restriction of the excavation or the need of reinforcement can then be made before the feature is actually penetrated by the tunnel.

In the long pilote boreholes drilled parallel to the central tunnel, a groundwater head monitoring system will be installed before the excavation of the central tunnel starts. A multiple packer system will seal off various sections in the boreholes, which are selected from hydraulic measurements. In the sealed off sections of the pilot boreholes, non-sorbing tracers are injected, each tracer being significant for each section. During the construction of the central tunnel, impacts monitored as head changes will be used to identify hydraulic connections along and between identified fractures within the repository rock block.

During the construction of the central tunnel, a thoroughly mapping of the tunnel regarding rock type, fractures, fracture fillings, water inflow, tracer occurrence, etc will be carried out. The mapping will be a validation of the results from the pilot boreholes used to predict the rock quality.

Where features of great importance for the safety of the repository are encountered in the pilot boreholes, measures can be taken to stabilize and improve the conditions by e.g. grouting or sealing. This can be done before the tunnel front reaches the actual feature. However such features should be designated before and after the reinforcement which can be done by testing in the pilot boreholes and testing after the tunnel has been constructed.

From the central tunnel, boreholes will be drilled vertically upwards and downwards to control the bedrock and the distance to major subhorizontal fractures. These boreholes, having a spacing of about 100 m, will be equipped with a multi-packer system after logging and testing.

5.5 ROCK QUALITY DESIGNATION OF STORAGE TUNNELS

The storage tunnels should preferably be oriented perpendicular from the central tunnel. They reach out in the rock mass, possibly crossing previously drilled horizontal long pilot boreholes from the access tunnel and end approximately 100 m in front of the bounding major fracture zones.

A large stock of data comprised in an improved conceptual model of the rock within the block designed for the repository is available before the layout and construction of the storage tunnels starts. In this model the main features of water transmitting zones will be outlined and the quality of the bulk rock known. The distances between the bounding major fracture zones and the central tunnel is about 150-400 m depending on the size of the allocated rock block for the repository. Interpolations of geological data, as fracture zones, infilled breaks etc over these distances will hardly be accurate and several geological features might still be hided in the rock between the central tunnel and the bounding major fracture zones.

The hydraulic conditions in the bedrock will be impacted by the central tunnel. Prior to the start of storage tunnel lay-out and construction, a general knowledge of the groundwater head in the rock mass is available from measurements performed in the pilot boreholes drilled from the access tunnels. In combination with mapping and measurements of inflow, also the properties of the main water transmitting features in the rock should be known. The central tunnel further acts as a new boundary for the groundwater conditions.

The strategy in the designation of the rock quality of the virgin rock for the storage tunnels includes three main campaigns:

1. Use of information obtained from previous stages

2. Use of information from pilot boreholes and measurements

3. Use of information during construction of storage tunnels

The use of the information obtained from mapping of the central tunnel will delineate areas with adverse features in the bedrock around the central tunnel.

In the second campaign, the virgin rock quality will be designated by the use of boreholes, borehole testing and measurements. Pilot boreholes along the projected line of the storage tunnels will be drilled and cores taken. Firstly, pilot boreholes will be drilled at each third site for storage tunnels, that is with a spacing of about 100 m. These boreholes will be drilled at such length that they penetrate the surrounding bounding major fracture zones, c.f. Figure 5-4. Geophysical measurements both single and cross-hole, mainly radar-

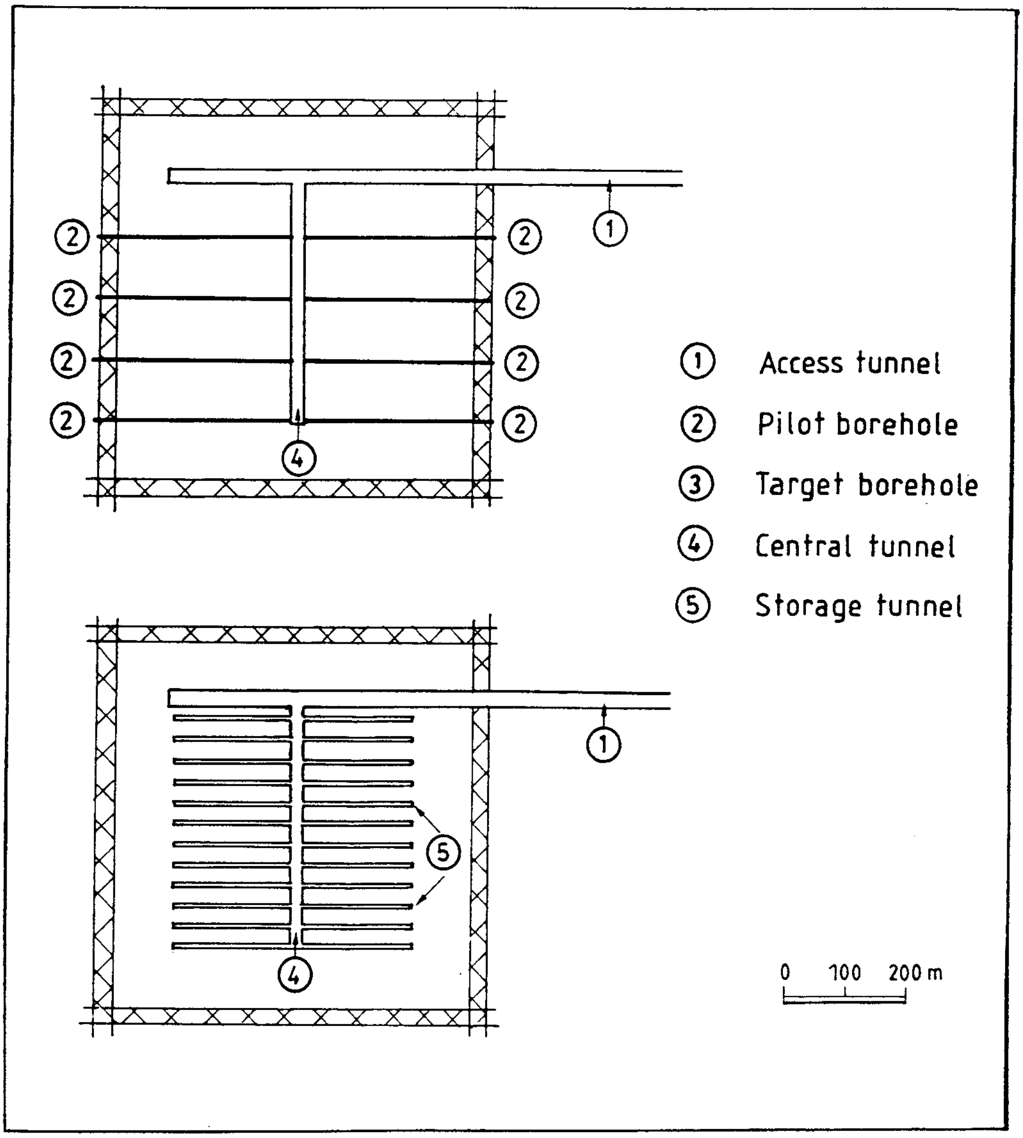


Figure 5-4. Pilot boreholes for the storage tunnels laid-out from the central tunnel. Every third storage tunnel is piloted by a horizontal borehole which also penetrate the bounding major fracture zones.

measurements, will provide a picture of the fracture network system. Hydraulic testing by i.a. spinner measurements will give additional information on location of transmissive fractures and zones in the boreholes. This picture will improve the first conceptual model set up from previous investigations.

The pilot boreholes will be equipped with a multi-packer system where the packers are placed to separate and monitor the identified main features. Each feature will be hydraulically tested by allowing the sealed-off sections to be free flowing and then monitor the changes in water head during this period and the preceding period of pressure build-up. Additional boreholes outside the future line of the storage tunnels should be avoided in the repository.

The second campaign of drilling and testing will probably be time-consuming. It is, however, of importance that the virgin rock quality should be designated during conditions where all disturbances are controlled. A careful planning of the testing campaign with due consideration to the actual hydrogeological structures identified would minimize the time needed. It may also be possible to perform the testing in one subarea after which the construction can start and the testing be moved to a new subarea within the future repository.

Based on the results from the pilot borehole drilling and testing, an improved conceptual model of the transmissive fracture geometry and properties is obtained. This model then serves as background for a discarding process of sites for future storage tunnels or restrictions in length of the tunnels. Before the storage tunnels are constructed, tracers are injected in the sealed off sections also in the new pilot boreholes. Each section should be injected with a unique non-sorbing tracer.

The storage tunnels will be thoroughly mapped in the same way as the central tunnel. An interpreted map of rock types, fractures and other structures in the bedrock will be compiled as a result of all mapping carried out. This map is then the basic map of the repository regarding the rock quality. On the map, quality data as hydraulic conductivity, location of transmissive fractures and their length will be added. Rock quality classification with regard to the long term safety of the repository can be then done from this map.

The bedrock surrounding the storage tunnels should be as little disturbed as possible by the applied construction technique (effects of blasting). Full-face drilling may be considered, but in the main concept, very careful and smooth blasting is assumed. Impact on the rock quality due to rock stress redistribution can hardly be avoided. Designation of the quality of the disturbed zone around the tunnel will involve the same testing procedure as described for the central tunnel. These tests, if done, should be performed in two campaigns, before and after construction of the tunnels. However, the testing procedure should not include methods which will affect the surrounding bedrock. Boreholes drilled from the storage tunnel into the surrounding rock should be avoided and the designation technique will be limited to methods as inflow measurements, tunnel geophysics and geological documentation.

5.6 ROCK QUALITY DESIGNATION OF DEPOSITION HOLES

The deposition holes in the KBS-3 concept are vertically drilled from the floor in the storage tunnels. The spacing between the boreholes is set at 6 meters. Each borehole will have a total length of 7.5 meters and a diameter of 1.5 meters. The boreholes will be carefully located using mapped data from the storage tunnels and considering characteristic structural features of the local rock mass. Thus, the individual boreholes can be placed such that few or none adverse features like zones of fractures, infilled breaks or transmissive fractures, will be intercepted by the holes. In order to further improve the siting technique and designate the quality of the rock close to the deposition holes, pilot boreholes will be used prior to the actual drilling of the holes.

As a first step in the siting process, pilot boreholes will be used at each fifth deposition hole. These boreholes should reach a depth 15 m below the tunnel floor, c.f. Figure 5-5. In this sense, the term pilot borehole is used for a core-drilled borehole in which cross-hole measurements and hydraulic testing will be performed. Radar measurements should also be conducted from the tunnel floor (tunnel radar) to get information on adverse features in a profile along the tunnel. A better resolution of this profile will then be achieved by various recordings in the pilot boreholes and by borehole radar measurements.

In such a way, all major fractures in the deposition hole profile along the tunnel line will be identified. Quality figures in terms of transmissivity and extension of water yielding fractures etc can then be inserted as results from hydraulic testing of the pilot boreholes. Interconnectivity between fractures and tunnels will be obtained by the use of the head monitoring system installed and the tracers injected during the previous stages in the repository construction. All of these quality data will be scrutinized before making the final decision on the location of the deposition holes.

At this stage, decisions will be taken on which borehole sites should be developed. Before drilling of a final deposition hole at a selected site, a pilot borehole (7.5 m in length) is drilled and tested and a new selection process is performed before the drilling of the actual deposition hole.

After drilling of each deposition hole, careful mapping of the holes will be carried out. A full hydraulic testing of the borehole by use of specially designed equipment will be made. The data obtained is then the final designated rock quality data for each deposition

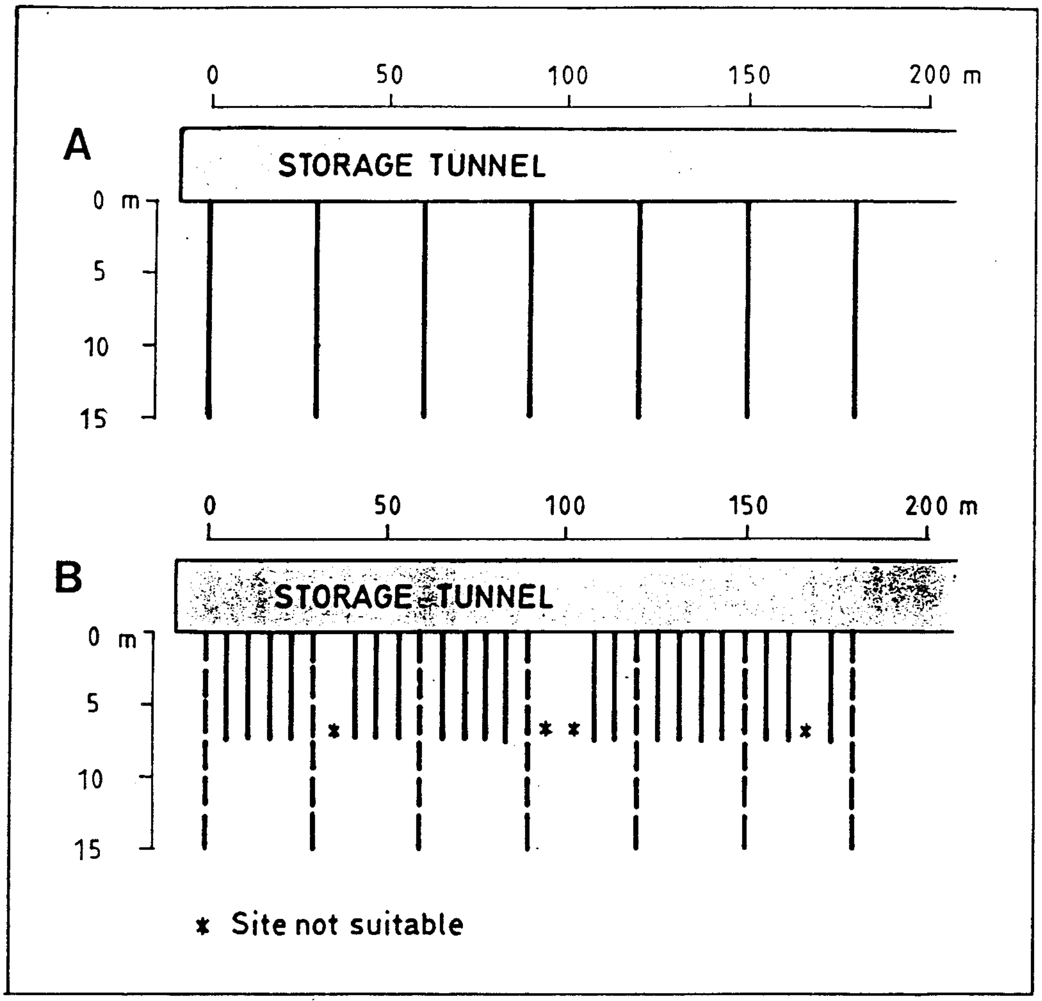


Figure 5-5. Pilot boreholes for the deposition holes in a storage tunnel. - A. At each fifth site for cross-hole measurement. B. At each site approved for testing before final drilling of deposition holes.

hole. A further selection is now possible if too adverse data are obtained.

Around the deposition holes, a disturbed zone will be created due to the rock stress redistribution. Depending on the orientation of the individual fractures versus main rock stress directions, the redistribution will cause increased or decreased hydraulic conductivity.

No boreholes will be allowed in the close surrounding of the deposition boreholes and thus no direct designation of the rock quality in the skin-zone will be possible. Instead comparison between results from testing of pilot boreholes and final deposition holes will give numbers of the quality of the disturbed zone.

5.7 BARRIER FUNCTION

5.7.1 Sealing effect of the canister envelopes

The blocks of highly compacted Na bentonite surrounding the canisters will absorb water, swell and ultimately become homogeneous with a dry density that ranges between about 1.4 and 1.75 t/m^3. The hydraulic conductivity at normal ambient temperature will be in the interval 10^{-14} to 10^{-13} m/s, i.e. lower than that of the crystal matrix of the rock, and about one order of magnitude higher at about 70°C. The swelling pressure that is exerted on the rock will be between about 3 and 15 MPa, c.f. Table 4-2, which means that the initial rock stress condition is at least partly reestablished and certain fractures, close to or intersecting the holes become partly or totally closed. The huge swelling potential of such dense smectite clays means that the contact between the rock and the clay is very intimate and tight, and that there are no flow passages along the interface.

The potential of dense bentonite to penetrate fractures is very beneficial and as concluded from Table 4-3, it is obvious that richly fractured rock with fracture apertures of 1-2 mm will be sealed to a distance of several decimeters from the holes in a relatively short time. This offers a possibility of accepting rather fractured, "low quality" rock in the near-field, provided that the fractures are not too wide and long. Also, it is required that thermally or tectonically induced shear does not expand existing fractures too much. Consideration of a very conservative case i.e. one in which rock mass displacement along steeply oriented joints induces frequent fractures through one or several deposition holes, c.f. Figure 5-6, has given a good picture of how safe the concept actually is. The shear displacement was assumed to alter the initial joint assemblage to a system of fractures with 5 times the number of initial joints and with 5 times the initial aperture of these joints, and this was found to yield a fracture volume of almost 3 m^3, assuming the fractures to be disc-shaped and extend 3 m from the holes. The filling of this space by bentonite originating from the deposition holes would seal the fractures with a clay gel with an average density of 1.5 t/m^3, at the same time reducing the bulk density of the remaining highly compacted clay core from about 2.05 t/m^3 to about 1.9 t/m^3. This yields a drop in swelling pressure from 10-15 MPa to 3-7 MPa, and an increase in permeability from the interval $2 \cdot 10^{-14}$ - $8 \cdot 10^{-14}$ m/s to about $1.5 \cdot 10^{-13}$ m/s, which does not jeopardize the barrier effect of the bentonite in the deposition holes. In fact, it demonstrates that significantly fractured rock

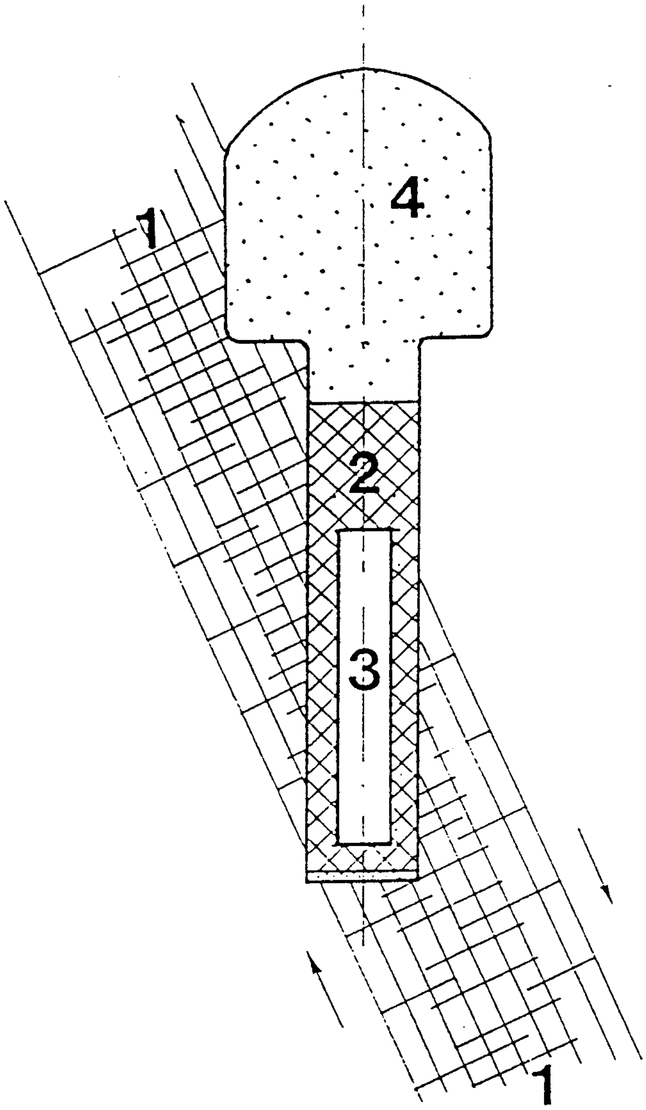


Figure 5-6. Clay-embedded canister in fracture-rich "near-field" 1) Shear zone, 2) Highly compacted bentonite, 3) Canister, 4) Sand/bentonite backfill.

may well be considered for hosting canisters and also that considerable fracturing in the operative lifetime of repositories can be acceptable. It is clear, however, that if such fracturing is associated with the percolation of hydrothermal solutions from larger depth, the chemical state and isolating properties of the bentonite may be significantly altered.

5.7.2 Sealing effect of the tunnel backfill

While the WP cave slot allows for very effective compaction of the backfill except for a very small volume at the top, the KBS-3 tunnels can be backfilled with very dense bentonite/sand (10/90) material only to about 2/3 of the height with the presently intended technique. The low density of the uppermost part that follows from application of such mixtures by blowing, can be partly counteracted by increasing the bentonite content. Still, the compressibility of the tunnel backfill will be rather high, and a more effective compaction technique should therefore be applied. It is anticipated that a very soft condition of the upper part of the backfill makes it vulnerable to erosion and there is also a risk that the exposure to heat may yield a complete loss in swelling ability. At any rate, the rock at the crown will not be supported effectively by backfills of the suggested type.

5.7.3 Disturbance caused by excavation

Shear failure has been observed in shafts with 1.5 m diameter, demonstrating that large diameter boreholes may be unsuitable where the primary stresses are very high. Under normal conditions at 500-1000 m depth, failure will not take place although stress-induced changes in aperture - both expansion and closure - and in interconnectivity of steeply oriented fractures adjacent to boreholes with 1-2 m diameter are certainly expected.

Due to the removal of solid mass when excavating tunnels and shafts, the virgin rock stress situation will change in the vicinity of the tunnels. Despite the fact that excavation projects at depth have been very common in civil and mining engineering for a long time, only limited knowledge has accumulated as to the stress redistribution effect on the hydraulic properties of the remaining adjacent rock. This is valid especially for deep-sited tunnels characteristic of KBS-3 repositories and for long shafts (slots) as in the WP-cave case.

In the current understanding, it is realized that a very shallow zone (0.5-1 m) close to the tunnel will experience an increase in total "rock porosity" defined as the fracture space, due to damage caused by blasting and stress relief. Further off in the surrounding rock, the redistribution of rock stresses will probably cause a decrease in hydraulic conductivity in radial direction and an increase in tangential and axial direction, at least in certain parts of a cross section. It is beyond doubt that the disturbed zone is characterized by a much higher axial conductivity than that of the virgin rock. Since the size of KBS-3 tunnels is very moderate - their cross section can be approximated to be circular with 5 m diameter - the extension of the zone with enhanced hydraulic conductivity in the axial direction is also moderate. Assuming, as for the WP cave, that the aperture of tangentially oriented fractures are altered within a distance corresponding to one diameter, their interconnectivity will not be significantly increased to more than a few meters distance. Adding to this the partly relaxed shallow 0.5-1 m zone of rock that is damaged by blasting, it is realistic to believe that an approximately 2 m wide, continuous disturbed zone, serving as "superconductor" is formed along the entire system of interconnected tunnels and shafts.

Calculation of the secondary stress situation around the silo at SFR shows that the tangential stress will be two times the maximum horizontal virgin stress /Stille et al, 1985/. Other calculations of the stress situation around the tunnels in the rock cavern area at SFR show up to four times higher secondary stress than the virgin horizontal rock stress. This maximum stress is obtained at a distance of 2-3 m from the periphery of the tunnels, which is slightly less than half the diameter of the tunnels /Winberg and Carlsson, 1987/.

It is concluded from this and from the previously mentioned fact that the upper part of the tunnel backfill does not effectively support the rock at the top, that the disturbed zone is the most important hydrological component in the entire repository area. Especially the rock at the top of the tunnels and at their base, where the blasting effects are particularly strong, represent "superconductors". Naturally, significant water flow at the rock/backfill interface at the top of the tunnels may cause erosion of the backfill, which would lead to local regions of high hydraulic conductivity.

In summary this means that initially conductive fractures with a particular orientation, i.e. more or less parallel to the long axis of tunnels and shafts, become increasingly conductive. Although this is usually not recognized since the skin-effect around the tunnel reduces the hydraulic conductivity in the radial direction. The location and intensity of water inflow into a tunnel through the skin-zone is probably very much dependent on the characteristics of the shallow rock affected by blasting.

5.8 MEANS OF MAXIMIZING THE ISOLATING CAPACITY IN THE CONSTRUCTION PHASE

5.8.1 Preferable types of rock

The common character of granite, being very impermeable in relatively large volumes but having local, strongly conductive passages of long extension, is attractive because local sealing can make large volumes almost

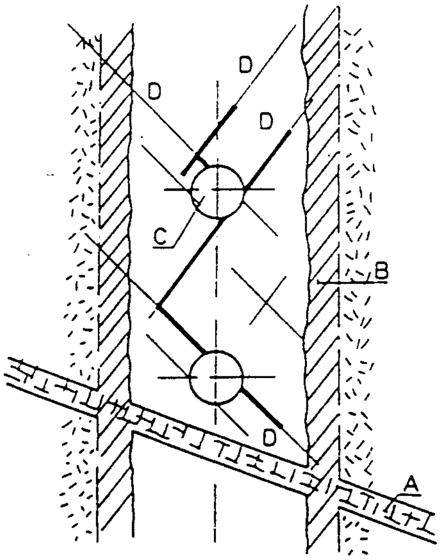


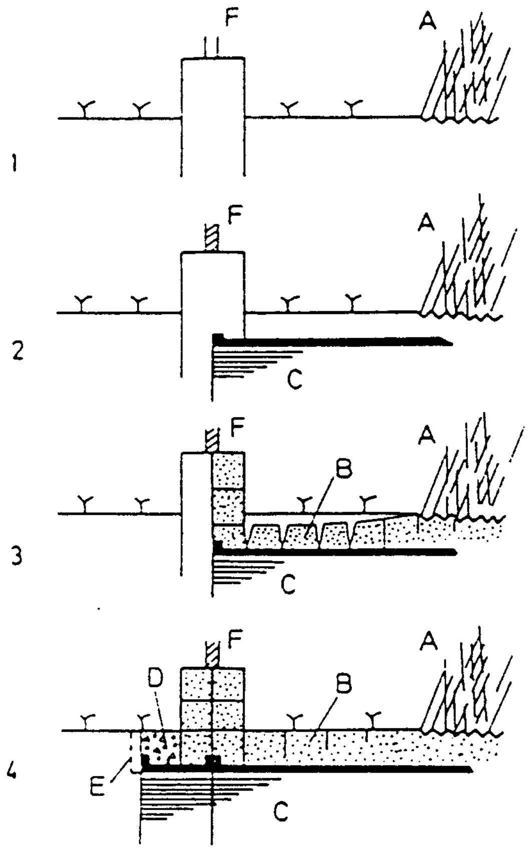
Figure 5-7. Principle of shunting off fractures (D) serving as short circuits; thick lines indicating sealed parts. (A) is major hydraulically active rock zone, (B) disturbed zone of increased permeability ("superconductor"), (C) is canister hole.

impermeable. Also the risk of splitting and delamination at points where the principal stress state is locally very anisotropic, which is usually expected at the tunnel roof, is certainly less than in gneiss or shales, which all speaks in favour of granite as host rock.

The most important task is to orient the tunnels and shafts so that the structure, i.e. fracture systems and dikes, are favourable oriented with respect to the stress state. This is a common matter in locating and orienting caverns for gas storage and presents no major problem. A particular advantage of the KBS 3 concept is that deviation from originally chosen orientations can be made in the course of the excavation operations.

5.8.2 Plugging and sealing

While sealing activities in order to redirect or stop water flow in the surrounding rock would be very expensive and not particularly practical for the WP cave, the KBS-3 concept offers possibilities of controlling water flow effectively in the near-field as well as at larger distances at moderate costs. As to the near-field, grouting by use of clay-based materials as well as cement slurries can be made of fractures with apertures as narrow as 10-20 μm. Hereby, rock volumes of about 250 m^3 per deposition hole can probably be sealed so that no groundwater movement takes place within it, c.f. Figure 5-7. A most important way



Procedure:

1 Slot is cut and holes drilled for grouting

2 Grouting is made and casing installed

3 Bentonite blocks, fragments and powder are applied between casing and rock (rapid swelling at the water-bearing zone)

4 Extension of casing, completion of slot-filling and casting of concrete support

Figure 5-8. Axial section of tunnel illustrating possible arrangements and procedures in plugging tunnels so that passage-through is allowed for. (A) Pervious rock-zone, (B) Bentonite blocks, (C) Steel casing, (D) Concrete, (E) Temporary steel form, (F) Boreholes for grouting.

of cutting off water flow in the disturbed zones can be made by systematic grouting of the shallow rock over shorter or longer distances. Very effective isolation at strategic sites can be made by cutting slots for application of blocks of highly compacted Na bentonite and combining this with grouting. As demonstrated by Figure 5-8, this can be arranged so as to allow for passage through the plug in the construction and deposition periods. At the stage of final sealing of the repository, the casing is removed and replaced by block fillings of highly compacted Na bentonite. By this, stagnant water conditions can be created around large portions of the tunnel system.

The sealing philosophy on the very large scale is indicated by Figure 5-9, which shows how strategically placed clay plugs and grouting screens can help to

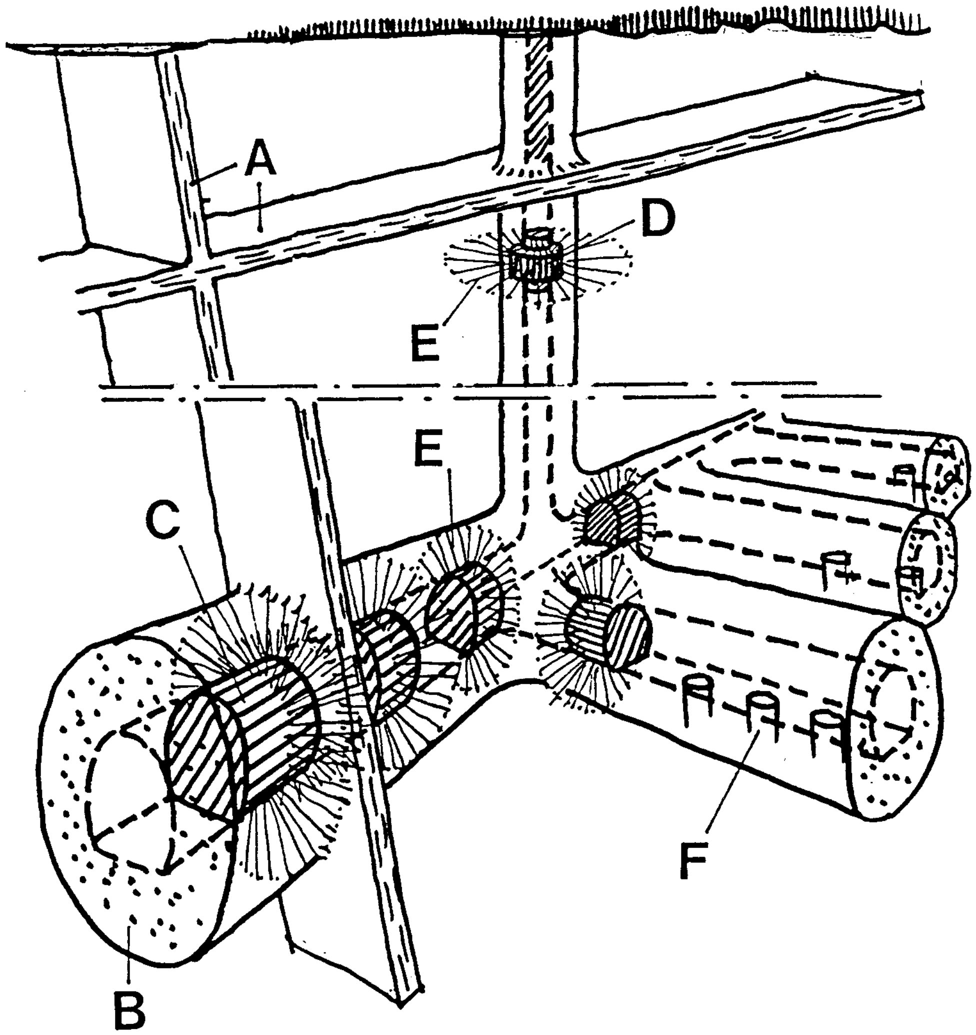


Figure 5.9. Application of local sealings to create large volumes of stagnant water in a KBS-repository. A) Natural fracture zone, B) Disturbed zone with increased axial hydraulic conductivity, C) Clay plug in a transport tunnel, D) Clay plug in a vertical shaft, E) Grouting screen to shut-off of disturbed zones.

create large volumes of stagnant groundwater.

5.8.3 Choice of quality level

The KBS-3 concept allows for the choice of quite different quality levels of which the following three criteria can be suggested for the near-field conditions:

1. Natural statistical variation of the hydraulic conductivity is accepted, applying a standard

distance between the holes of 6 m

2. Impermeable near-field conditions within 3 m distance from each hole is required, for which grouting is applied. Certain but very few hole sites have to be given up

3. Impermeable near-field conditions within 6 m distance from the holes is required, which is attained by systematic grouting of the floor. This would allow for applying a 6 m distance criterion, except where the hole sites intersect long-extending conductive zones

5.9 INVESTIGATION SCHEDULE IN ROCK QUALITY DESIGNATION OF A KBS-3 REPOSITORY

Mentioned above in Section 5.3 through 5.6 is a number of investigations performed during various stages of a repository construction. In order to summarize the efforts and their successive performance in the whole construction program, an overall schedule should be set up. The schedule describes what should be done and when and to what extent the investigations are performed. The outcome or expected results of the investigations are given in terms related to rock quality designation.

The schedule comprises five stages as:

Stage 1: Site characterization (1a) and establishing of monitoring system (1b).

Stage 2: Access shaft (2a) and access tunnel (2b).

Stage 3: Pilot boreholes for siting (3a) and construction (3b) of central tunnel.

Stage 4: Pilot boreholes for lay-out (4a) and construction (4b) of storage tunnels.

Stage 5: Pilot boreholes for aly-out (5a) and construction (5b) of deposition holes.

In the following discussion, stage 1 is excluded since it is described and discussed elsewhere, e.g. Ahlbom et al /1984/.

The objectives for the different stages are various and manyfold. The current report is focused on the objectives of the rock quality designation during stages defined. These objectives can briefly be described as:

Stage 2a: Identify adverse features, especially horizontal zones at depth. Obtain bulk properties of the host rock.

Stage 2b: Obtain properties and geometries of adverse features bounding the repository host rock.

Stage 3a: Identify main adverse features within the rock block assigned for the repository. Best lay-out of a central tunnel.

Stage 3b: Obtain geometries and properties of main adverse features within the rock block assigned for the repository.

Stage 4a: Obtain increased knowledge of main adverse features within the rock block assigned for the repository. Best lay-out of storage tunnels.

Stage 4b: Obtain increased knowledge of geometries and properties of adverse features within the rock block assigned for the repository in the scale of 5-50 m.

Stage 5a: Obtain detailed knowledge of transmissive fractures in the very near field of the deposition holes (scale less than 5 m). Decision on siting of individual deposition holes.

Stage 5b: Obtain increased and final knowledge of transmissive fractures at each deposition hole, their geometry, and interconnection with other fractures.

During the different stages, various investigation methods would be applied. These methods and boreholes needed are summarized in Table 5-1 regarding their application in the different stages.

In the course of the repository siting and construction, conceptual models of the host bedrock for the repository will be developed and the understanding of the groundwater occurrence and flow will increase. Based on the site characterization results, a first base conceptual model of the hydrogeology of the repository area is set up. After access shaft and tunnels have been excavated - including target boreholes - this model is improved and the geometry and properties of the bounding major fracture zones will be achieved. At this stage, the conceptual model would give a thoroughly understanding how water is entering and leaving the rock block assigned for the repository, that is how the bounding major transmissive fracture zones are interconnected, their properties and fluxes. Also a first estimate on the connection between these zones and the groundwater within the rock block would be at hand.

Table 5-1. Different methods to be used in rock quality designation of a KBS-3 repository and their application in different stages.

Method	Stage 2a	2b	3a	3b	4a	4b	5a	5b
Pilot boreholes, core mapping	X	X	X	X	X		X	
Target boreholes, - " -		X	X					
Observation boreholes	X							
Geological mapping	X	X		X		X		X
Inflow measurements	X	X		X		X	X	X
Hydraulic testing	X		X		X		X	
Head measurements	X	X	X	X	X	X	X	X
Tracer injection		X	X		X			
Tracer observation				X		X	X	X
Borehole geophysics	X		X		X		X	
Crosshole geophysics	X		X		X		X	

During stage 3 (layout and construction of central tunnel) the understanding of the water flow within the rock block is improved. Numbers of transmissive fractures, their extent and interconnections should offer a possibility of forming a layout in both a stochastic and a deterministic way such that an improved conceptual model of the repository rock block volume can be achieved. The importance of transmissive fractures or zones varies in the block and at this stage, the main fractures would be identified. During the next stage when the storage tunnels are laid-out and excavated, a detailed knowledge of these fractures in the scale of 5-50 m should be obtainable. The degree of interconnections between the individual fractures and the major fracture system should be considered. The interconnections are more emphasized in stage 5 when the deposition holes are sited, constructed and tested.

In stage 5, the scale in rock quality designation is more detailed. Fractures intersecting the deposition holes should be considered with reference to their interconnection to other transmissive fractures and also to the major fracture zones. Such a conceptual model based on deterministic data should be set up for each individual deposition hole in the end of the repository construction stage and before the actual emplacement of the waste package.

A certain percentage of the deposition holes probably has to be rejected due to the presence of adverse features like transmissive fractures in or close to the holes. These are observed both before siting of the individual hole, in which case a new site will be selected, or after the pilot borehole drilling and crosshole measurements. At this stage, the pilot borehole will not be transformed into a deposition hole. Also after final construction of a hole, it might turn out as not appropriate due to adverse features present. Thus the rejecting of deposition holes or site for deposition holes can be performed at least three different occasions during stage 5.

It is clear, however, that a practical strategy in the selection of suitable locations of deposition holes needs to be set, and that it has to based not only on the presently observed fracture patterns but also on future changes due to heat generation and cooling, as well as on possible tectonics. In this conjunction, rather great comfort is offered by the self-sealing capacity of the surrounding rock by the highly compacted Na bentonite in the deposition holes. This capacity, as well as the possibility of grouting major fractures in the near-field with clay-based material may actually allow for very economic selection principles as concerns the location of the deposition holes.

6 STRATEGY IN ROCK QUALITY DESIGNATION OF A WP-CAVE REPOSITORY

6.1 SITING STRATEGY

A repository designed according to the WP-Cave principle will be located at depth chosen on site specific data. In the general description, the WP-Cave repository is an egg-shaped structure, about 450 m in height and 230 m in outer diameter, c.f. Chapter 3. The uppermost point, c.f. Figure 6-1, is located about 135 m below the ground surface. The main principle when locating a WP-Cave is the same as for a KBS-3 repository, that is in a rock block surrounded by major vertical fracture zones. The respect distance to these zones is set at 100 m.

The horizontal area of a rock block required for a WP-Cave is about 450x450 m^2, that is less than for a KBS-3 repository in one storey within one block, c.f. Chapter 3.

The WP-Cave construction can generally be divided in five main stages comprising construction of:

1. Access shafts and tunnels
2. Annular tunnels
3. Hydraulic cage
4. Bentonite-sand barrier
5. Waste storage cavern

After the hydraulic cage is constructed, the egg-shaped volume, about 14 Mm^3, will during the construction and operation stage be more or less drained by the numerous boreholes constituting the cage. The function of a WP-Cave is further based on the presence of the bentonite-sand barrier in combination with the hydraulic cage. As in the case of a KBS-3 repository, the adverse features in the bedrock will be transmissive fractures.

Outside the hydraulic cage, transmissive fractures would connect the cage hydraulically to the regional groundwater flow in major bounding fracture zones. In the WP-Cave study, the term near field is considered primarily to be the volume encompassed by the hydraulic cage. The rock quality designation for a WP-Cave repository thus primarily aims at describing the rock volume inside the cage.

In contrast to the KBS-3 concept, the flexibility of a WP-Cave is very limited once the hydraulic cage is laid out and the annular tunnels constructed. To compensate for this, a thoroughly pre-investigation including a

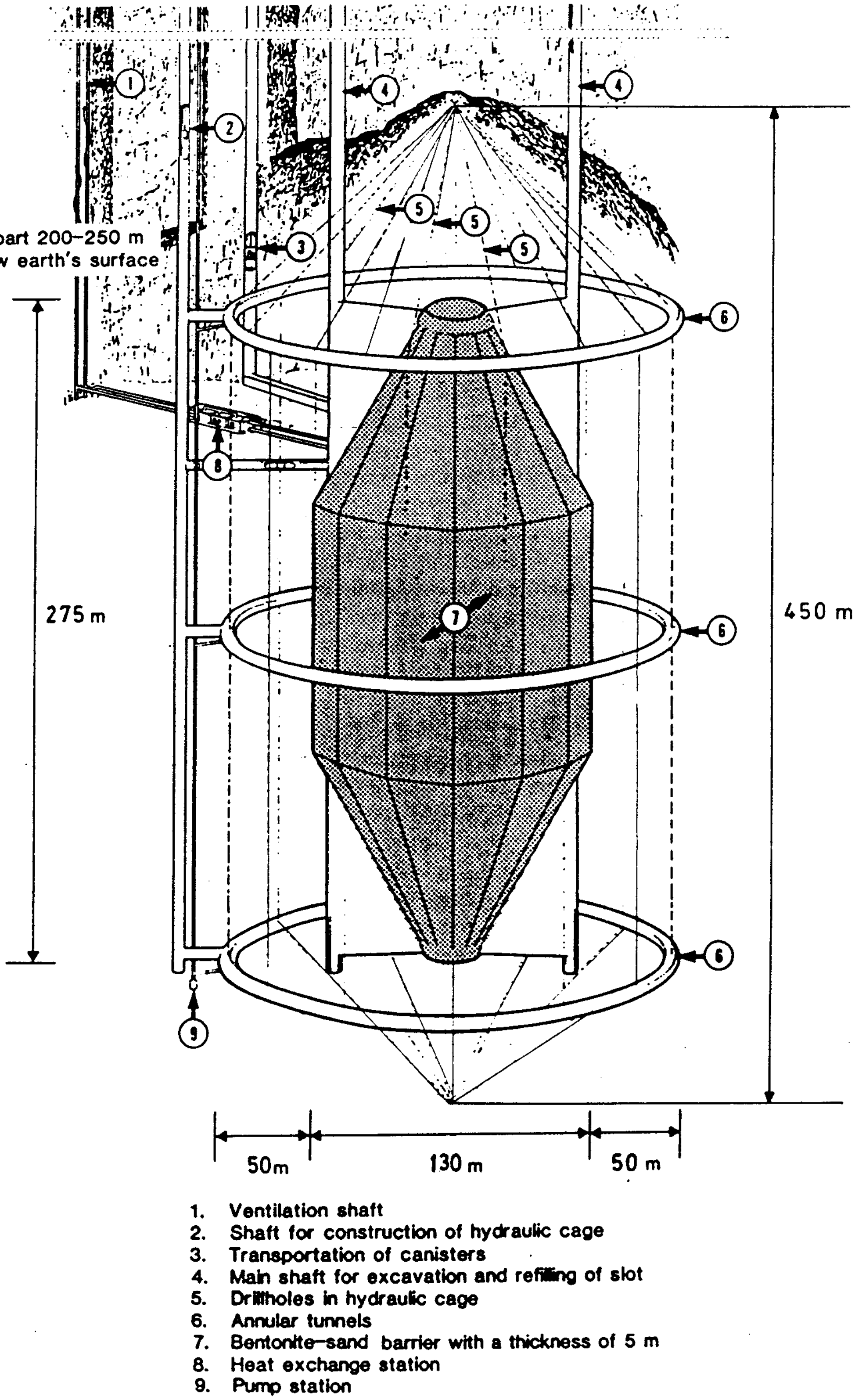


Figure 6-1. Design of a WP-Cave repository (modified after SKN /1985/).

detailed conceptual model of the assigned rock block need to be conducted. Also a heavier use and confidence of engineering barriers is required compared to the KBS-3 concept.

6.2 ROCK QUALITY DESIGNATION DURING CONSTRUCTION OF ACCESS SHAFTS AND TUNNELS

The first access shaft in the WP-Cave construction will be for the hydraulic cage and will be located aside the cave. The shaft can be sited in the same rock block as the repository or in a neighboring rock block. The shaft will be about 500 m deep.

The rock quality designation before and after the shaft sinking will mainly be the same as described for the KBS-3 case, c.f. Section 5.3. At the levels of 200, 345 and 490 m below the ground surface, access tunnels will be constructed towards the location for the annular tunnels of the hydraulic cage. The levels are given in the lay-out of the WP-Cave before construction of the shaft. However, the occurrences of transmissive major horizontal fracture zones at the prescribed levels will change the level of the access tunnels and the annular tunnels. Their mutually distances will be kept.

In order to achieve optimal information on rock quality designation during excavation of the access tunnels and the continuing annular tunnels, these should be constructed in sequences, starting with the upper access and annular tunnel. The excavation of all three access tunnels should have been completed before construction starts with the upper annular tunnel.

Long pilot boreholes will be used for the access tunnels before and during the construction. The measurements to be performed in these holes are the same as in the KBS-3 case and include core mapping, hydraulic testing and geophysical measurements.

6.3 ROCK QUALITY DESIGNATION DURING CONSTRUCTION OF ANNULAR TUNNELS FOR THE HYDRAULIC CAGE

The three annular tunnels all circumscribe the rock volume assigned for the bentonite-sand barrier and the storage cavern. The radius of the hydraulic cage is 115 m and each of the annular tunnels would thus be about 720 m in length. It is suggested that the construction will start with the upper annular tunnel, even before the final depth is reached in the access shaft.

The annular tunnels are 6 m in width, allowing horizontal pilot boreholes of 50-60 m in length to be drilled along the tunnel line. The pilot borehole investigations comprise core mapping, hydraulic testing and geophysical measurements. The boreholes are mainly aimed at forecasting the rock quality for construction purposes, but information on the hydrogeological conditions is achieved simultaneously.

The tunnel should be mapped and the inflow measured covering various sections of the tunnel. From the tunnel, a curtain of boreholes will be drilled both vertically downward and inclined upward as illustrated in Figure 6-2. In total 240 drainage boreholes with a spacing of 3 m will be drilled upwards and downwards. The diameter of these boreholes are designed at 150 mm. Before the start of the construction of the middle annular tunnel, about 16-32 cored pilot boreholes (Ø 56-76 mm) should be drilled along sites for future drainage boreholes. These boreholes should be 140 m in depth (vertical downwards) and 100 m in length (inclined upwards). No curtain boreholes will be drilled at this stage. Crosshole measurements, hydraulic testing and geophysical logging will be performed in the holes. The boreholes should further be equipped by multiple-packer systems for head measurements. The sections to be sealed off, will be selected primarily from the results of the hydraulic testing.

During the excavation of the middle annular tunnel, impact on the groundwater head will be monitored in the pilot boreholes and analyzed in order to identify and later quantify transmissive fractures of extent covered by the lay-out of the boreholes and the tunnel. A thoroughly monitoring of changes in inflow in the upper annular tunnel will also contribute information on transmissive parts in the bedrock. The actual excavation and combined rock quality designation will be carried out in the same way as for the upper annular tunnel. Also from this tunnel, pilot vertical boreholes will be drilled 140 m downwards. The number of holes would be 8-16 and the same investigations and monitoring as for the pilot boreholes from the upper annular tunnel will be performed.

The construction and the rock quality designation of the lower annular tunnel is done in the same way as for the other tunnels.

6.4 ROCK QUALITY DESIGNATION DURING DRILLING OF THE BOREHOLE CURTAIN IN THE HYDRAULIC CAGE

After the three annular tunnels are constructed, a first conceptual model is at hand describing where transmissive fractures are present in the periphery of the WP-Cave rock volume. This model can probably only delineate the main features within the circumscribed rock volume interpolated along and between the tunnels. An improved knowledge of the existing features should be able to obtain during drilling of the curtain of drainage boreholes in the hydraulic cage.

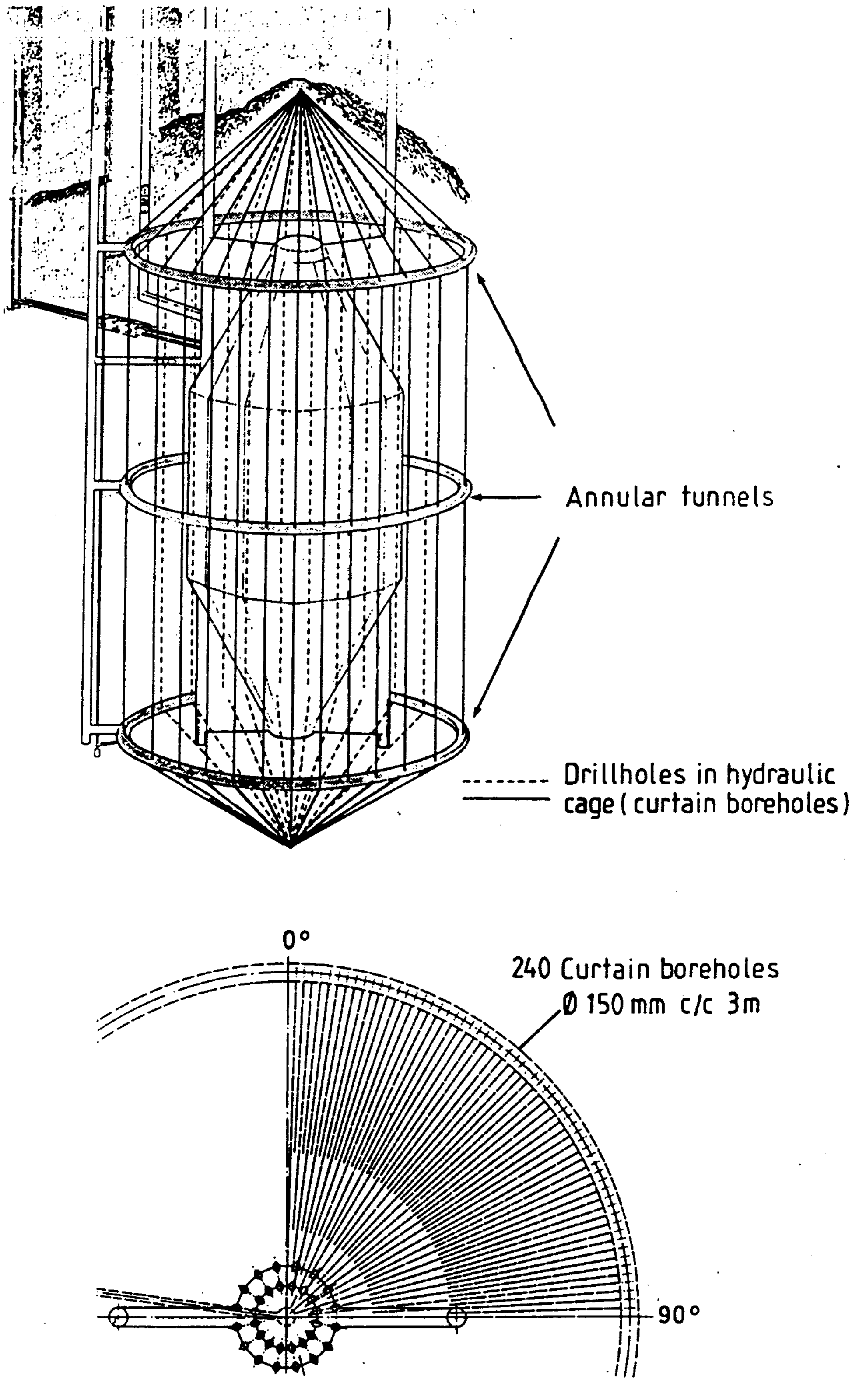


Figure 6-2. A. Lay-out of the hydraulic cage. Not all drainage boreholes are indicated. B. A quarter of the lower annular tunnel with boreholes in the drainage curtain.

Before the drilling, some adjustments of the installed monitoring system, now a total of about 50 boreholes with probably up to 250 monitoring sections, might be needed based on the results obtained during construction of the annular tunnels. Additional boreholes for groundwater head monitoring may be needed to cover the bottom cone of the hydraulic cage.

The actual drilling of the drainage boreholes (no cored boreholes) can start from the top or from the bottom of the hydraulic cage. It is also possible to start at several places simultaneously, but it is important that the drilling will be performed in such a way that optimal information could be obtained on the occurrence and geometry of transmissive fractures. This can for instance be done by installing a multiple-packer system for head monitoring in a borehole immediately after drilling and logging but before drilling of the neighboring boreholes. All responses of drilling recorded in adjacent boreholes can then be used to identify transmissive fracture intersections. After drilling, crosshole measurements (e.g. borehole radar) will improve the conceptual model set up on the occurrence and geometry of the transmissive fractures.

It should be mentioned that after completion of the curtain of drainage boreholes, an excellent arrangement is at hand for i.e. geophysical crosshole investigations and evaluation of a fractured crystalline bedrock. The function of this curtain is however to drain the WP-Cave during the construction and operation period. After closing and sealing of the repository it will short-circuit the groundwater flow once the WP-Cave has been filled up with groundwater, thereby substantially reducing the groundwater flow through the repository /SKN, 1985/. The first mentioned function reduces the possibilities of hydraulic testing of the WP-cave rock mass using the natural groundwater in the boreholes once the curtain is completed. Thus, other means of testing have to be utilized in order to characterize transmissive fractures identified by crosshole measurements after completion of the borehole curtain.

6.5 ROCK QUALITY DESIGNATION OF THE BENTONITE-SAND BARRIER

The bentonite-sand barrier designed as an eggshaped structure inside the hydraulic cage will encompass a rock volume of 2 Mm^3. The barrier is built as a 5 m wide slot starting at the bottom level, 95 m above the bottom of the hydraulic cage. The construction includes blasting, transport of excavated rock through two specially constructed shafts, and filling and compaction of pre-mixed bentonite-sand material.

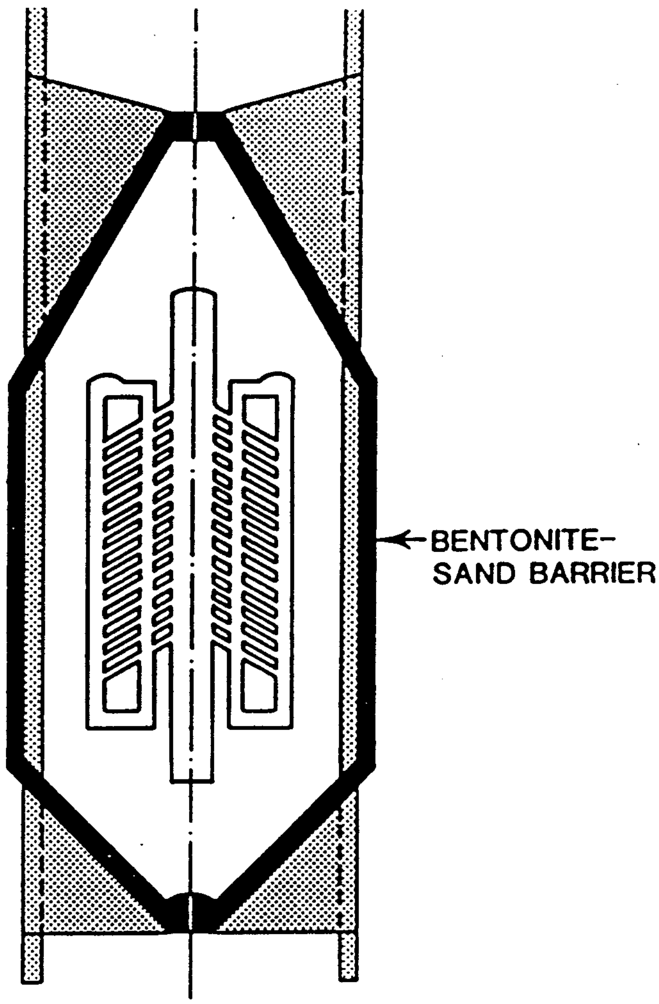


Figure 6-3. Section through the bentonite-sand barrier and the storage cavern in a WP-Cave repository concept (modified after SKN , 1985).

The two access shafts to the bentonite-sand barrier are not connected to the access shafts of the hydraulic cage. At the shaft construction, the same rock quality designation procedure as for the other shafts should be applied.

Between the two access shafts, two circumscribing "slot-tunnels" will be constructed. As illustrated in Figure 6-3, the rock volume between these tunnels and the future slot will host the storage cavern for the waste. In order to designate the rock in the slot before excavation, boreholes should be drilled vertically from the upper slot tunnel at a spacing distance of 25 m (16 boreholes). Core mapping, borehole logging, crosshole measurements and hydraulic testing should be carried out using also the possibility of response recording in neighboring drainage boreholes in the hydraulic cage.

Between the bentonite-sand barrier and the hydraulic cage there is a 50 m zone of intact rock. During the excavation this rock will be drained by the cage. The presence of water transmissive zones in this rock volume would thus be hard to identify from observations of water inflow during excavation. However, the conceptual model developed during the construction of the hydraulic cage and the results from the borehole investigations mentioned above should make it possible to predict the location of major transmissive fractures. Detailed geological mapping should therefore be performed of the slot walls, including indications of water flow or water occurrence, and should be compared with the conceptual model obtained from the borehole investigations. Techniques for measurements between the slot wall and the cage boreholes, e.g. radar or seismics should be developed and used to identify transmissive fractures in the rock mass. There is also a need to characterize these fractures regarding geometry and properties in order to quantify their transport capacity in a future safety assessment. The properties may be quantified from hydraulic testing in the cage boreholes. However, at the slot wall, methods are currently not available for such exercises, but should be developed.

Where large fracture zones are known to be penetrated by the slot, reinforcement might be needed as e.g. grouting or bentonite injection. The rock quality after such reinforcement should be designated.

6.6 ROCK QUALITY DESIGNATION OF THE STORAGE CAVERN

The storage cavern in the WP-Cave repository will be reached by specially designed access shafts which should have no communications with other shafts or with the hydraulic cage or the bentonite-sand barrier. The hydraulic cage and the bentonite-sand barrier are therefore intersected under special conditions where water tight constructions will be used.

The storage cavern consists of a central vertical shaft, 14 m in diameter, c.f. Figure 6-4. Outside this shaft are 12 vertical shafts of 2 m in diameter (inner shafts) sited along the periphery of a circle of 26 m diameter. Further out are another 12 shafts, 3 m in diameter (outer shafts), sited along the periphery of a circle of 61 m diameter. The central shaft and the inner and outer shafts are connected by radial tunnels (canister tunnels) at 14 levels. The tunnels are inclining from the centre towards the periphery as illustrated in Figure 6-4. The height of the centre shaft is 155 m and of the other shafts 110 m.

The rock volume within the 100 high cylinder with 122 m in diameter will thus be penetrated by tunnels and a

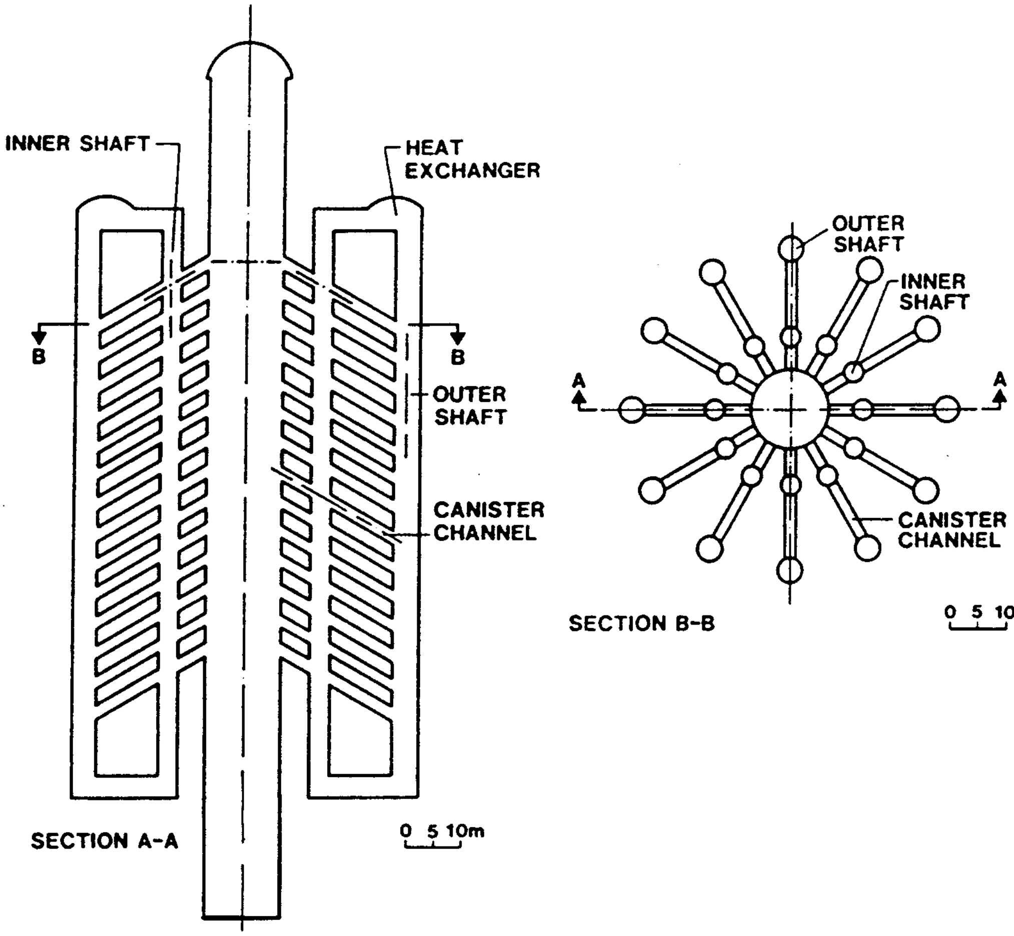


Figure 6-4. Section through the storage cavern in a WP-Cave repository concept (modified after SKN, 1985).

shaft and constitute the storage facility. The temperature within this volume will during operation be 40-60^{o}C and after closing up to 100-140^{o}C. Such high temperature will effect the rock quality and designation for these effects should be set up in a special program.

Compared to the KBS-3 repository concept, no flexibility in the emplacement of the waste exists in the WP-Cave case. The different shafts and connecting tunnels can not be excluded like in the KBS-3 case. Thus, any adverse features like highly transmissive fractures within the storage cavern area has to be excepted but, if possible, reinforced. However, the WP-Cave concept implies that the natural flow should strongly be reduced through the storage volume due to the hydraulic cage and the bentonite-sand barrier.

In the drained storage cavern area, the rock quality designation will be limited to geological mapping and geophysical single and cross hole measurements. The boreholes will in this case be pilot boreholes for the inner and outer shafts. It is assumed that when the storage cavern area is being constructed, the construction of the hydraulic cage and the bentonite-sand barrier are already completed. Thus no response in groundwater head in the monitoring sections in the cage would be possible to achieve. Neither would any change in inflow to the bentonite-sand barrier be possible to recognize as an impact of the construction of the storage cavern.

Before the actual construction of the storage cavern starts, a comprehensive conceptual model of the adverse features within the rock volume should be at hand based on the results from the construction and rock quality designation of the hydraulic cage and the bentonite-sand barrier. The results from geological mapping etc of the storage cavern construction could thus be considered as a large validation exercise of the models set up.

6.7 BARRIER FUNCTION

6.7.1 Sealing effect of the backfill

Bentonite-sand mixtures can be granulometrically composed so that the density becomes very high on effective field compaction ($\tau \approx 2.2$ t/m^3). Using low smectite contents ($\approx$10%), which is required in order to minimize the settlement of the inner rock core when it comes to rest on the lower bed backfill, it is possible to arrive at hydraulic conductivities that are as low as 10^{-10} m/s. For the backfill higher up in the slot, the bentonite content is planned to be higher, i.e. about 20 % at mid-height and up to 50 % at the top, since the settlement of the inner rock core will require a higher swelling potential to fill up the expanded space.

As indicated by Table 4-2 the swelling pressures for bentonite-poor mixtures are insignificant even at high bulk densities but if the backfill in the vertical and upper parts of the slot can be very effectively compacted, it is probable that the swelling pressure can be raised to a couple of megapascals at maximum.

The hydraulic conductivity is significantly increased if the backfill is exposed to vapour or higher temperatures than about 90°C, while the swelling pressure will be reduced by 50 % or more.

The matter is complexed by the fact that the heat evolution causes an appreciable expansion of the inner rock core, which produces a compaction of the slot backfill and a temporary, strong increase in contact pressure between the backfill and the rock. On cooling, the opposite series of events takes place, implying that the inner core, which is probably severely fractured by then, shrinks and that the backfill must expand in order to maintain a tight contact with the surrounding rock. The contact pressure will decrease and since the heat exposure and preceding compaction will reduce the expandability, it is most probable that the surrounding rock will have little support from the backfill after a couple of hundred years. Thus, the structural changes that were induced by the excavation, i.e. the change in fracture geometry by stress relief and the fracturing caused by blasting, will be largely preserved. Accordingly, the key question as to the design-related effect on the hydraulic conductivity of the integrated rock/backfill system is the excavation response with special respect to the intrinsic structural rock features.

6.7.2 Disturbance caused by excavation

While blasting influences the frequency and aperture of fractures and therefore the hydraulic conductivity to less than about 1 m from excavations, the stress relief has a very significant effect as illustrated by simple theoretical estimates and measurements in deeply located caverns and drifts.

Thus, approximating the rock to be an ideally elastic, fracture-poor medium with a modulus of elasticity of $5 \cdot 10^4$ MPa, which is a representative value of large volumes of fracture-poor rock, and applying basic elastic continuum theory one finds that the outer boundary of the large slot will be displaced inwards by about 1-3 cm if the primary, horizontal rock stresses are 10-20 MPa and Poisson's ratio ranges between 0 and 0.3. An E-modulus of $2 \cdot 10^4$ MPa would represent a more realistic value for normally to relatively richly fractured rock and this would give a lateral displacement of 2.5-7.5 cm. Measurements made in the course of excavation of deeply situated long-extending cavities verify this. Hence for the rock cavern shown in Figure 6-5, the displacements were about 4 cm of the 40 m high rock wall, which would suggest an approximately double figure for WP-cave wall. This is supported by systematic measurements of the lateral convergence of narrow stopes at the deep Näsliden mine in Sweden, where continuous cut-and-fill mining has been applied, was about 5 cm at 40 m stope height and 8 cm at 80 m height. The average effective modulus of elasticity was estimated at about $5 \cdot 10^4$ MPa.

It is reasonable to assume, on this basis, that the

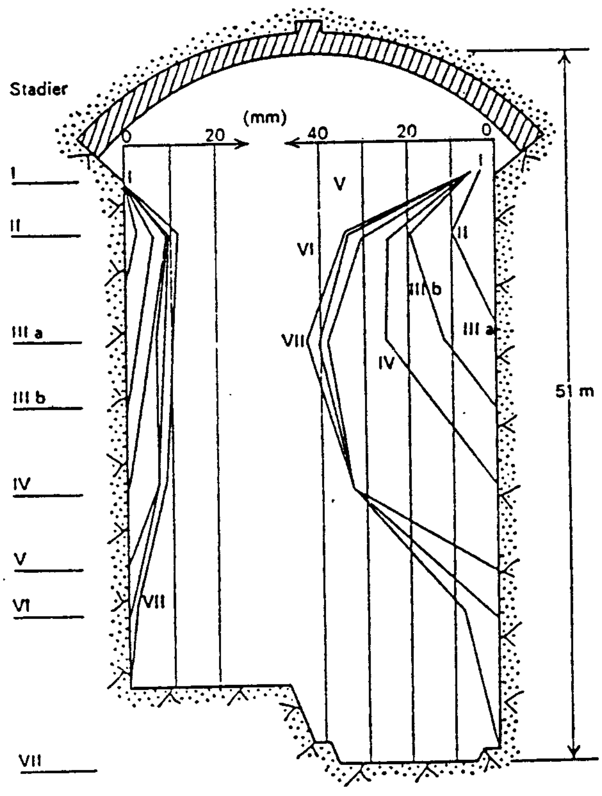


Figure 6-5. Lateral displacements recorded at different stages of the excavation of a large cavern at 250 m in shale (after Yoshida and Yoshimura, 1970).

average inward movement of the outer WP cave wall will be about 5-8 cm at mid-height of the slot. The lower figure is more probable since effective compaction of the backfill will help to reduce the movement of the wall.

Keeping in mind that the modulus of elasticity of 10.000 to 100.000 m^3 of rock is roughly 5-20 % of that of the crystal matrix, the difference being almost entirely related to stress-induced changes in fracture aperture of large rock masses, it is concluded that the disturbed zone has a considerable extension in a WP cave repository. Actually, referring again to the theory of elasticity, one finds that the radial displacements at a distance equal to 100 m from the outer rock wall of the WP cave under isotropic horizontal stress conditions are approximately 30% of those at the wall, and that they are roughly 50% at 30 m distance. Making the conservative assumption that steep, tangentially oriented fractures are altered within 100 m distance, but that their interconnectivity is increased only within 15 m distance from the wall, it is estimated that the net bulk hydraulic conductivity of the virgin rock is increased by at least 50 times in the vertical direction within 15 m from the outer rock wall. As pointed out previously, the low net swelling pressure of the backfill after the heating period is not able to balance or even significantly reduce the wall movement. Nor will there be any penetration of bentonite sufficiently deep into even several millimeters wide fractures to have a sealing effect on the pervious zone.

6.8 MEANS OF MAXIMIZING THE ISOLATING CAPACITY IN THE CONSTRUCTION PHASE

6.8.1 Preferable types of rock

It is basic to the concept that a low Poisson's ratio will minimize the radial elastic, or in practice, elasto/plastic rebound. This points to subhorizontally structured gneiss as the most suitable type of host rock. The type of fracture sets and usually strong anisotropic hydraulic conductivity of such rock are in support of this suggestion.

It is very essential that the repository should not be located where there is strong primary stress anisotropy in the horizontal plane. The stress ratio should not exceed 2, values higher than 3 being unacceptable.

6.8.2 Construction activities

No significant changes in shape or location of the slot and the confining walls can be made once the excavation has started, which means that sealing or other efforts can be made only locally. Among possible activities to increase the isolation power one recognizes:

- Sealing of very permeable rock-zones identified in the course of the excavation by grouting of long, medium and short boreholes, drilled from the slot.

- Improvement of the penetration and sealing power of the backfill close to the rock by applying a 0.25-0.5 m layer of highly compacted blocks of Na bentonite at the rock wall

The rock quality designation of the hydrogeological conditions in the near field of a repository requires well established methods regarding investigations of the presence, extension and character of hydraulic units. This chapter aims to briefly present and discuss some of these methods as well as methods that need further development.

7.1 GEOLOGICAL METHODS

Designation of the rock quality before the construction of shafts and tunnels is based on investigations in cored boreholes and possibly also percussion drillholes.

The investigation of the core from diamond drilled boreholes should include logging of the rock type and rock alteration as well as open and sealed fractures. The log for open fractures should include the following: orientation of fractures (absolute orientation, when the core is oriented, or orientation relative to the core axis, when the core is unoriented), roughness, infillings, alteration, the relation of the fracture to other structures in the core, ability of water transport, fracture zones, crushed zones and core losses. The sealed fracture logs may be recorded quantitatively (number of fractures per meter of core) or as above. The result of the core logging should be stored in a data base system where the data can be easily retrieved and statistically treated. Geostatistical treatment using variograms may give information on reoccurring intervals of increased fracture frequency in the rock, which can be conceptualized into a network of zones in the rock mass.

The core and borehole logging should specifically note any indication of mechanical instability (core-discing, spalling etc).

The construction of shafts and tunnels will provide detailed information on the geologic and tectonic character of the investigated bedrock. This information should be mapped in detail and stored in the SKB data base for immediately access for interpretation and correlation with other surface and borehole data. The major structures should be correlated with the use of the CAD-system.

Geologic parameters of interest to map are rock type, rock alteration (including discolouring), foliation, folds, fractures (sets and estimated spacing) and fracture zones. Fracture zones are usually hard to study from the surface due to soil coverage of their outcropping parts. Also, there is a limited possibility to study the geologic/tectonic character in boreholes due to the small sampled volume. A tunnel (or shaft) will on the other hand provide excellent possibilities for these studies. The deformational character of the fracture zones should be studied in detail, including rock types, alteration, infillings, fracture characteristics and mechanisms for fracture zone development/tectonic history. This information will form a base for e.g. groundwater flow, migration properties, rock mechanical modelling etc. It is therefore important that the data needs for these calculations are specified so that the requirements on the geologic/-tectonic mapping can be determined.

Techniques for geologic/tectonic mapping should be developed that satisfy the requirements above. One possibly way to achieve this is to use modern laser equipment for positioning of structures which, together with comments regarding geologic/tectonic and hydraulic character, can be directly stored in the SKB data base.

Fracture traces on the tunnel (or shaft) walls should be mapped. This information is needed for a discrete network modelling approach. In the fracture model of Baecher and Lanney /1978/, the information sought for are the trace lengths distribution, the trace orientation (dip and dip direction) and the trace density. In order to use the data for other model approaches it probably will be necessary to store the complete trace map of the investigated area.

There are a number of techniques to estimate trace statistics from planar cuts. Two examples are the scanline method /e.g. Baecher and Lanney, 1978, Priest and Hudson, 1981/ where statistics is made from the traces that intersect a straight line on the sampling window and different area methods where statistics is made from all traces within a simple area (circular disc, rectangle).

7.2 GEOPHYSICAL METHODS

Conventional single hole geophysical logging methods gives information on conditions close to the borehole. Other methods, i.e. the borehole radar, gives high resolution data at distances of tens to hundreds of meters from the borehole. The suite of logging methods that should be applied will partly depend on the physical conditions at the site. It can therefore be recommended to test several logs (resistivity, caliper, temperature, sonic, natural gamma and radar) in the first boreholes and then select the most appropriate ones.

Crosshole geophysical methods may be used when information is required on a plane spanned by boreholes. When planning the drilling program it should be remembered that crosshole geophysical studies can best be performed using boreholes spaced max. 100-200 m apart for crosshole radar and max. 200-400 m for crosshole seismics.

7.2.1 Resistivity logs

The resistivity of the rock may be measured using different configurations. The single point resistance log is the simplest of the systems and it uses an electrode lowered into the borehole and an insulated cable with an electrode buried at the surface to provide the return path for the current flow. The point resistance has a high resolution and it is in many cases possible to detect individual fractures.

The resistivity using normal configuration is measured with a current electrode and a potential electrode in the borehole. The depth penetration of the log is approximately 1 meter. The porosity of the rock can be calculated after certain corrections of the obtained data and thus give information on possible fracture zones that are intersecting the borehole.

The resistivity using lateral configuration is measured with one current electrode and two potential electrodes in the borehole. This configuration is used in order to obtain a better resolution of the fracture zones and the contact between rock formations of different electrical conductivity than that obtained with the normal configuration.

7.2.2 Caliper

The caliper log is used for obtaining an accurate profile of the borehole diameter, since corrections for any changes in this parameter have to be applied to most of the other geophysical logs. In some boreholes the caliper log can be used to identify lithographic and stratigraphic changes, but its main use in competent rock is in the initial identification of fracture zones.

Most caliper tools are mechanical devices and are classified in terms of the number of arms which are in contact with the borehole wall. The arms can be used to change the resistance of a potentiometer, resulting in a voltage variation, or alternatively the movement of the arms give rise to a capacitance change which alters the output frequency of an oscillator. In both systems the positions of the arms over their measurement range is calibrated against the change in voltage or frequency. Calipers with one, two, three, four and six arms are available /IAEA, 1985/.

7.2.3 Temperature measurements

Temperature measurements are normally performed with a thermistor with an relative accuracy of 0.01 degree. The distance between the measuring points may vary between 1 and 5 meters. An uncased borehole can provide a pathway for water flow out of fractures and between fractures or high-permeability zones, thus producing characteristic thermal anomalies.

7.2.4 Sonic measurements

The sonic log responds to the total porosity of the rock mass and can be used for detection of changes in the lithology. The basic tool consists of a piezoelectric transducer; at one end of the sonde a pulse of high-frequency sound (25 - 35 kHz) is emitted which is picked up by one or more receivers in the body of the sonde. The porosity of the rock can be calculated from the compressional wave velocity /Wyllie et al. 1958/.

7.2.5 Natural gamma logs

Natural gamma radiation is measured in units of R/h with a relative accuracy better than 10 % for the low radiation intensities that normally occur in e.g. gneisses. The accuracy is better at higher radiation intensities. The radiation can be measured from a distance of a decimetre or so into the rock. In this manner, an idea is obtained of the mineral composition of the rock only in the immediate vicinity of the drill hole.

7.2.6 Radar measurements

The borehole pulse radar technique has been developed as part of the International Stripa Project with the objective to identify and characterize fracture zones at considerable distances from the borehole. The radar uses very short pulses of electromagnetic energy, which are transmitted and received by dipole antennas inserted into the boreholes. The system uses centre frequencies of 20 - 60 MHz corresponding to wavelengths of a few meters in rock.

Radar reflection measurements have been used to identify fracture zones and determine their position and orientation. The zones often cause strong and well defined reflections originating from the resistivity change at the edges of the zones. The exact orientation of the zones can be determined by combining data from several boreholes. However, directional antennas are currently being developed which makes the true orientation in space possible from measurements in one borehole only.

The borehole radar system has also been used in crosshole

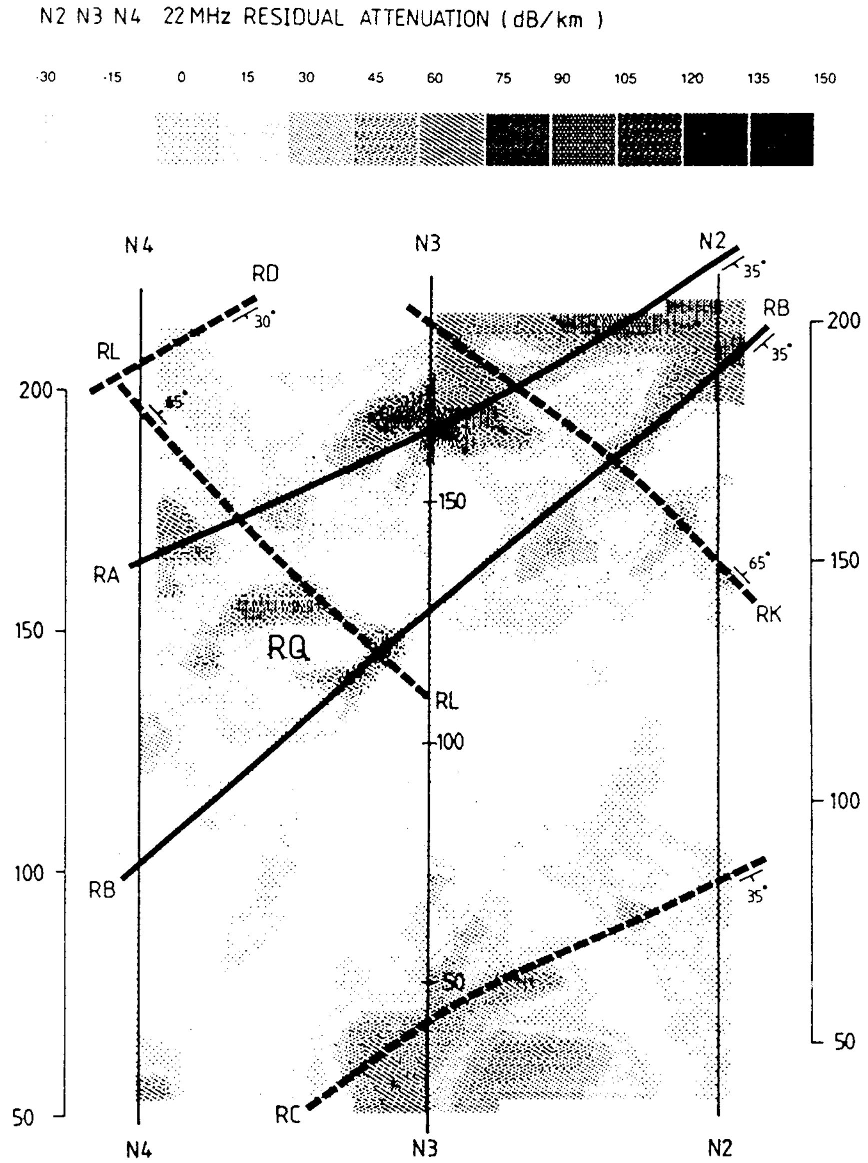


Figure 7-1 Example of crosshole radar measurements at Stripa, Sweden. The solid lines indicates major fracture zones whereas the dashed lines indicates minor fracture zones /SKB, 1988/.

configurations to collect data suitable for tomographic analysis, c.f. Figure 7-1. A tomographic inversion, using either traveltime or amplitude data, gives information on the extent of fracture zones in the plane spanned by the boreholes as well as a quantitative estimate of the electrical properties of the zones. Reflections have also been registered in crosshole surveys and have been analyzed together with single hole reflection data to describe the inhomogeneties in large rock volumes. Radar measurements have been performed at a number of sites and found to be a very efficient instrument for locating and characterizing fracture zones /Olsson et al, 1986/. Crosshole surveys are normally performed by the use of boreholes but crosshole measurements between tunnels are also possible by attaching the antennas to the tunnel walls. Further development of this "tunnel radar" is needed.

7.2.6 Seismic surveys

Seismic surveys are based on the relationship between the propagation of seismic waves and the physical properties of the rock. The wave is normally induced by explosives or engineered devices and picked up by geophones or accelerometers. Single hole surveys may be carried out using several configurations - seismic signals from the surface to the borehole (downhole VSP), from the borehole to the surface (uphole VSP) and from different locations in the same hole (acoustic sounding). Seismic crosshole surveys using tomographic analysis of the data may be used when information is required on a plane spanned by the boreholes, c.f. Section 7.2.6. The resolution of a seismic survey is very much dependent on the geometry of the survey as well as rock type, degree of fracturing and elastic properties of the rock /Olsson et al, 1987/. However, features down to 0.1-0.2 meters may be detected if the frequency of the induced wave is in the order of 100 kHz.

7.3 HYDROGEOLOGICAL METHODS

The principal parameters needed for the modelling of the hydrogeological system around a repository are the geometry of the system, its internal structures and its boundaries, and the hydrogeological characteristics of the rock. In addition, hydrogeochemical data can be used to interpret groundwater movements (methods for obtaining these data are covered in section 7.4). The techniques described below are all except one (water inflow measurements) used in boreholes and a lot of effort should be put on the positioning of these holes. Much of the success of a hydrogeological study depends on the design of the boreholes and on their positions, which in turn are heavily dependent on site-specific factors and on the geological environment being studied.

7.3.1 Measurement of hydraulic head

A hydraulic head monitoring system is designed to measure the variation of the piezometric pressure in sealed off sections in a borehole. The Piezomac 2 system, developed by SGAB , seals off borehole sections by rubber packers equipped with a feed-through for pressure tubes. The packers are connected to a pipe

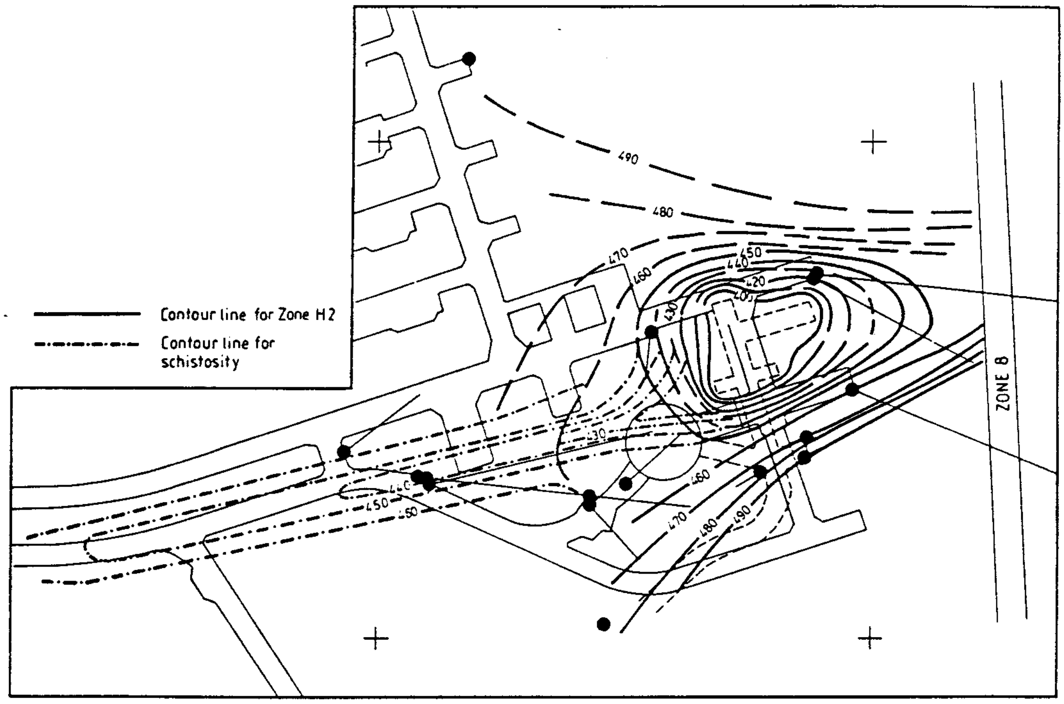


Figure 7-2 Example of groundwater head distribution at the facility for reactor wastes, SFR, in Sweden /Carlsson et al, 1986/.

string which is used for spacing the packers and also for lowering the equipment into the boreholes. Each section is hydraulically connected to a valve in the multipressure probe. The valve opens one section at a time and the pressure in the sections are recorded sequentially by a pressure transducer. The hydraulic head is measured in five different sections in a 56 mm borehole. An optional design of the downhole units enables up to 10 sections to be monitored in boreholes with a diameter of 76 mm. In the standard version, the surface data collection and control unit is designed for simultaneous control and monitoring up to eight individual multipressure probes. Communication via radio and a modem to the telephone network makes remote operations possible, such as reprogramming of sampling times, as well as transfer of data to a central computer. Figure 7-2 shows groundwater head isolines drawn from borehole observations at the facility for reactor wastes, SFR, in Sweden. In this case a horizontal fracture zone and a well developed schistosity and excavations through these geological units resulted in an obvious draw-down of the groundwater head.

7.3.2 Measurement of hydraulic conductivity

The prime parameter of interest to describe the flow characteristics of a hydraulic system is the hydraulic conductivity. It can be measured by a variety of methods, the appropriate ones depending on the expected range of hydraulic conductivity, the aims of the investigation, the scale and concept being considered and pure practical aspects. The methods recommended in this study for single hole measurements are transient injection tests with constant head and pressure build-up tests. Tests in single holes measures an effective hydraulic conductivity in the rock surrounding the borehole on a scale roughly equal to the distance of the packed off section. Multi-hole interference tests should also be conducted for the rock quality designation of major flow-paths over greater distances. These tests may also give indications on the hydraulic anisotropy of the rock.

The transient injection test with constant pressure consists of water injection into a section of the borehole packed off with inflatable packers. The pressure in the section is kept constant during the injection and the flow is measured as a function of time.

In a pressure build-up test the test section is allowed to flow freely until steady-state conditions are obtained. The section is then closed and the pressure build-up is measured versus time. Both injection tests and pressure build-up tests are preferably performed when the groundwater head is above the datum level e.g. in underground excavations. These tests are most suitable when the hydraulic conductivity is expected to be higher than 10^{-10} m/s.

In an interference test, a controlled disturbance is introduced in a selected section of a borehole and the response is monitored in different sections in surrounding boreholes. The disturbance in the active borehole may be obtained by pumping (surface boreholes) or by pressure release under regulated flow conditions (underground boreholes). The interference test will give information of the hydraulic properties between the boreholes and especially the hydraulic connectivities i.e. the interconnection of water bearing fractures and fracture zones.

A helpful tool, when deciding the location for measurements of hydraulic conductivity in a borehole, is the so called spinner. It measures the flow in highly conductive sections in a borehole. From these data, the transmissivity of the section can be calculated since it is proportional to the flow in the section under the assumption that the rock consists of planar parallel hydraulic conductors /Earlougher, 1977/.

7.3.3 Measurement of water inflow

Measurement of water flow into shafts and tunnels is an important source of information on the hydraulic character of the near-field of a repository. Practical limitations exclude inflow measurements in the entire repository but certain sections of special interest should be recorded.

Measurements of the inflow distribution can be made on several levels of ambition. The best picture would be obtained if a section of special interest is partially sealed off for careful registration of inflow, humidity and temperature of the ventilation air. Another way of conducting the measurements is to divide the excavation in subareas and measure the inflow into each subarea. For example, one could use the method with plastic sheets used in the Stripa 3D experiment /Neretnieks et. al., 1985/. A lower level of ambition is to place measuring weirs for the total tunnel flow relatively close and visually determine spots with much inflow. It may not be necessary to maintain the same level of ambition along the complete tunnel length. However, the highest ambition possible should be used at the tunnel sections where the most ambitious fracture trace recordings are made. When documenting the tunnel inflow the measured flows should be coupled to the actual absolute position of the inflow point (area). Methods for measurements of very small amounts of inflow are not at hand but should be developed.

7.4 HYDROGEOCHEMICAL METHODS

Comparison between the characteristics of groundwater sampled at different locations and time periods may aid the interpretation of the groundwater flow pattern. The samples should be taken both in the underground boreholes and at the locations were the inflow measurements are conducted. The waters should be analyzed for the main constituents (Ca, NA, K, Mg, Fe, HCO_3, SO_4, SiO_4 and Cl). In addition, a multitude of minor or trace constituents, as well as characteristic properties such as Eh and Ph, which can be used to differentiate groundwater types and flow patterns should be analyzed and determined. Examples of minor and trace constituents are tritium, whose presence in significant quantity may indicate relatively young groundwater and nitrates, which can indicate recharge by irrigation water.

8 GENERIC ROCK QUALITY DESIGNATION OF A KBS-3 REPOSITORY

8.1 BACKGROUND

Off-shore the Swedish mainland at Forsmark and sited below the Baltic Sea is the final repository for reactor waste, SFR. Before and during the construction of this repository and access tunnels, an immense geoscientific program was carried out during the years 1979-1986. Techniques and measurements used included core-drilling of pilot boreholes and observation boreholes, corelogging, hydraulic single and crosshole testing, head monitoring and tunnel mapping. The techniques used and results obtained are thoroughly described by /Christiansson, 1986/ and /Carlsson et al, 1986/.

The rock for the repository was found to host a number of major fracture zones. Vertical or subvertical zones delineated blocks of rocks which were suitable for hosting the SFR repository. These delineated blocks are in the following text regarded as the assigned rock for a KBS-3 repository and a program will be out-lined on how to designate the rock quality regarding hydraulic properties and conditions during the construction of such a repository. In this exercise, the KBS-3 repository will be located at 500 m depth where the geological and hydrogeological conditions are assumed to be identical with those encountered at the depth of the current SFR repository.

The detailed knowledge of the bedrock, which was obtained during the SFR construction, is assumed not to be available when starting lay-out and construction of a generic KBS-3 repository. However, the same pre-investigations and background data as for the SFR are assumed to be at hand and a first model of fracture zones and rock quality from borehole investigation is available.

The SFR repository is located and constructed at depth down to 150 m below the sea level, while a generic KBS-3 repository will be sited down to 500 m. In a scenario comprising these depth, assumptions have to be taken on the bedrock properties versus depth, occurrence and geometry of fracture zones, etc. In the current scenario and the scenario comprising WP-Cave repository at Forsmark (Chapter 9), the following assumptions on hydraulic properties and fracture zone geometry (Figure 8-1) are taken:

1. Horizontal fracture zones like zone H2 at SFR are repeatedly occurring in the bedrock at levels given in Figure 8-2. All these horizontal zones are dipping 14° towards the south. The zones are

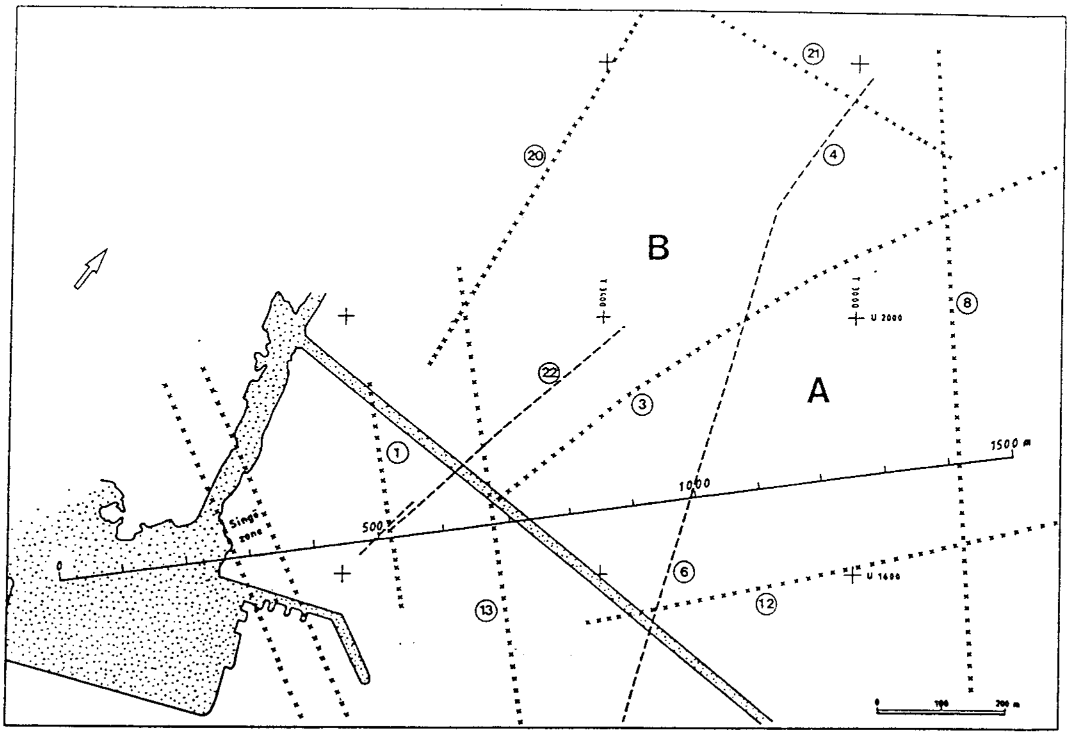


Figure 8-1. Fracture zones identified and assumed at site in Forsmark for the scenario with KBS-3 and WP-Cave repository.

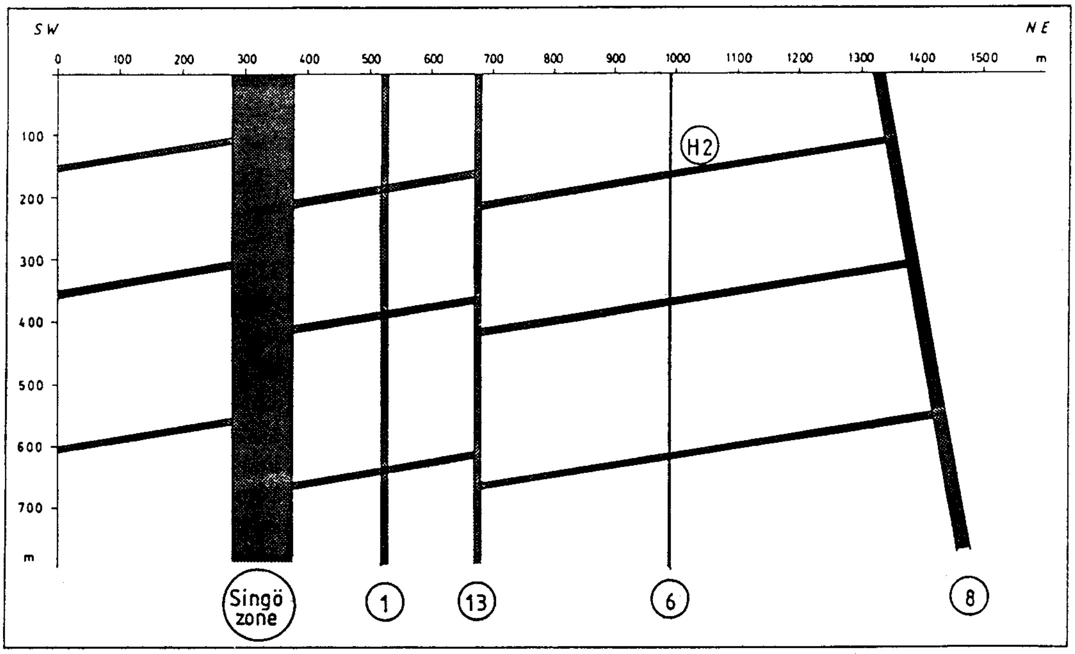


Figure 8-2. Section through the rock block and adjacent blocks assigned for the repository scenario at Forsmark.

on the average 10 m thick with an average hydraulic conductivity of $2\ 10^{-6}$ m/s. They are also encountered in the whole area assigned for the repository.

2. Vertical fracture zones encountered at the depth of SFR are all assumed to be continuous down to depth deeper than 500 m. Some of the zone have a slight deviation from the vertical but still continuous downward. Table 8-1 summarizes the width, dip and hydraulic conductivity of the fracture zones.

3. The frequency of transmissive fractures as observed in the SFR repository is one fracture per 82 m^2. At the depth of 500 m, this frequency is assumed to be one per 200 m^2.

4. The gross hydraulic conductivity of the rock blocks bounded by the identified fracture zones is assumed to decrease with depth according to the relationship given by Carlsson et al /1986/:

$$K = 8.9\ 10^{-6}\ D^{-1.30} \qquad (8-1)$$

where K = hydraulic conductivity
D = Depth below sea level

Table 8-1. Assumed data on width, dip and hydraulic conductivity of fracture zones at SFR considered in the KBS-3 and WP-Cave scenarios.

Fracture Zone No	Width (m)	Dip (°)	Hydraulic Conductivity (m/s)
Singö Zone	100	90	$2\ 10^{-6}$
1	10	90	10^{-6}
3	10	90	10^{-7}
4	10	80 NW	10^{-6}
6	2	90	10^{-7}
8	20	80 NE	$2\ 10^{-6}$
12	10	90	10^{-6}
13	10	90	10^{-6}
20	10	90	10^{-6}
21	10	90	10^{-6}
22	2	90	10^{-7}
H2	10	14 S	$2\ 10^{-6}$

8.2 ROCK QUALITY DESIGNATION STAGES

The generic repository at Forsmark will be designed to host 1100 tons of spent fuel. This amount requires 800 deposition holes. In the preliminary design, 1200 holes are however out-lined since a certain number of holes is expected to be discarded due to an adverse rock quality.

In the following text, the rock quality designation and the description of the repository construction performance will be described in the following stages comprising:

1. First lay-out of the repository
2. Construction of the access shaft
3. Construction of the access tunnels
4. Construction of the central (main) tunnel
5. Construction of the storage tunnels
6. Construction of the deposition holes

The designation of the rock quality has the general objective to identify the larger adverse bedrock features in the large scale and then to obtain more detailed knowledge as the repository approaches the very ultimate stage, the deposition holes. The designation further aims at establishing the interconnectivity between water transmissive fractures and between major bounding fracture zones and the water transmissive fractures in the assigned rock block for the repository. In the designation process non-destructive investigative methods will be used. A minimum of boreholes drilled from the repository block into bounding major fracture zones are aimed at. Instead, boreholes should be sited and drilled where tunnels will be considered in the forthcoming stages.

In the first repository lay-out stage, the major fracture zones are avoided. During construction of the access shaft special attention is focused on the occurrence of horizontal zones. These are usually hard to detect from ground surface investigations and their presence might change the level of the repository in the bedrock.

From the access tunnels, pilot boreholes for lay out and siting of central tunnel will be drilled, as described in Chapter 5. As a result, the block designed for storage tunnels and deposition holes will be described from investigations performed around its three peripheral sides by a tunnel and long horizontal boreholes as illustrated in Figure 8-3. The use of geophysical cross-hole measurements will give reveal features within the block considered. The main central tunnel will be sited and constructed along one the pilot borehole drilled. After tunnel construction, also the fourth periphery will be covered by pilot boreholes perpendicular to the tunnel at its inner end, c.f. Figure 8-3.

The descriptive model of the repository block, which is

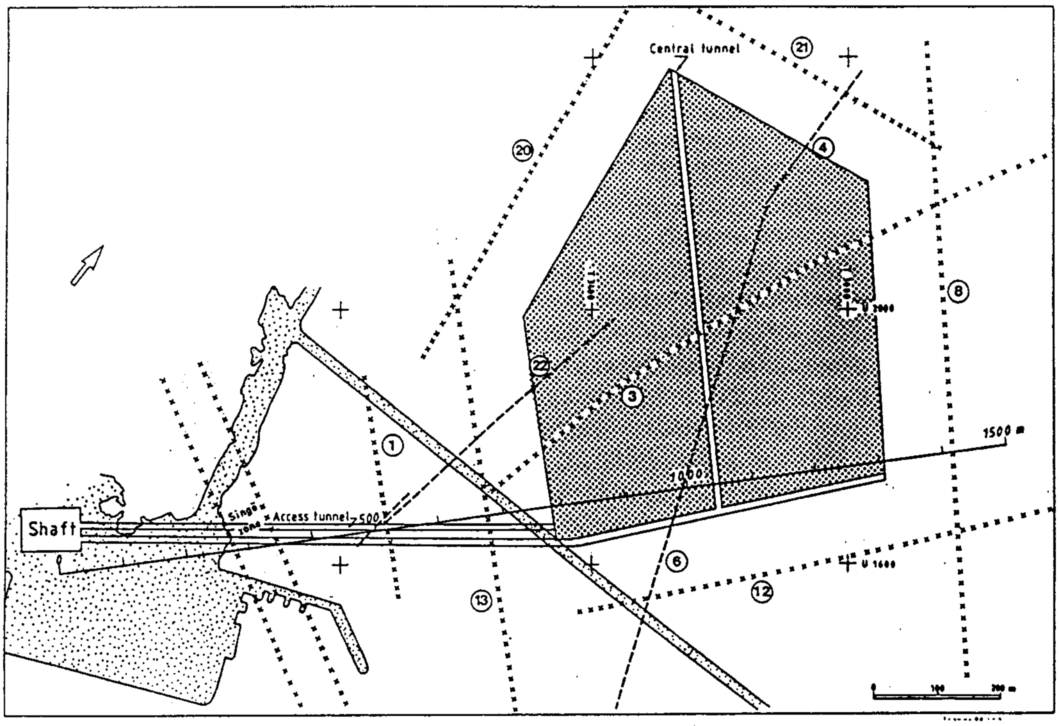


Figure 8-3. Preliminary lay-out of a KBS-3 repository with access shaft and tunnels in the Forsmark area.

now at hand, is the base for siting of the storage tunnels. Prior to the construction of these tunnels, pilot boreholes at every third to fifth tunnel will be drilled and investigated to designate the rock mass quality in the repository block. At this stage, minor continuous fractures and fracture interconnectivity are addressed and the network of these will be described in the improved descriptive model of the block. Where necessary, the lay-out of the storage tunnels might be changed or some tunnel sites even discarded before the construction.

In the repository construction program, the storage tunnels will successively be constructed together with the deposition holes. Thus, after approximately one third of the repository storage tunnels have been constructed, the siting and drilling of deposition holes starts.

The siting and construction of the deposition holes as described in Chapter 5 includes the following sub-stages:

1. Preliminary siting of the holes
2. Pilot drilling and testing of every fifth hole
3. Improved siting of the holes

4. Pilot drilling and testing of the selected remaining holes
5. Disqualification process
6. Final drilling and testing of the deposition holes
7. Final approval process

8.3 FIRST LAY-OUT OF THE REPOSITORY

The preinvestigation for a preliminary lay-out of a KBS-3 repository at Forsmark reveals a number of fracture zones. These are interpreted from seismic soundings and cored boreholes together with borehole logging and testing. The Singö zone, zone 1, 4, 8, 12, 13, 20 and 21, c.f. Figure 8-1, are described as major fracture zones. These zones should therefore be avoided within the repository. The remaining zones in Figure 8-1 are described as minor and will be recognized within the rock blocks when encountered during the construction. Measures could then be taken to seal or reinforce them at penetration. Further, deposition holes or storage tunnels crossing these features might be discarded in the final repository design.

With the available space at the SFR-area, a final repository according to the KBS-3 concept will be placed at the level of 500 m below the sea level. The repository will consist in the first lay-out of one part sited in two rock blocks (A and B in Figure 8-3).

In the preliminary lay-out, the access shaft is located on the main land at the same place as the present access to SFR. From the shaft, two access tunnels starting at 500 m level (below sea level) leads to the repository.

The outer part of the repository is sited at 100 m distance from the bounding major fracture zones. The storage tunnels from the centre main tunnel will be sited with a spacing of 33 m. The first lay-out of the repository is illustrated in Figure 8-3.

8.4 ACCESS SHAFT

8.4.1 Investigations

Prior to the shaft construction, vertical boreholes will be drilled along the centerline of the shaft and at close distances outside of the future shaft walls. Core logging and borehole logging will reveal if and where adverse features as transmissive fractures could be anticipated during the shaft construction. In order to designate the quality of the rock and the adverse features encountered, hydraulic testing will be performed in these boreholes. Conventional double-packer testing will be performed where sections of increased hydraulic conductivity are recognized. Based on these results and the results from core and borehole logging, multiple packer system will be installed before a large scale pumping test will simulate the shaft influence and at the same time designate the rock quality in the neighborhood of the shaft.

The main rock quality designation will however take place during the actual shaft construction. In vertical boreholes located at different distances from the shaft, head measurements during the construction will reveal the degree of hydraulic connection between the shaft and the surrounding rock mass and adverse features recognized. The access shaft is however located at about one kilometre distance horizontally from the repository area and probably hydraulically screened by major vertical or subvertical fracture zones (the Singö zone and zone 1 and 13) which implies that only major hydraulic features would be of importance when looking for hydraulic interconnections between the access shaft and the repository area.

Horizontal fracture zones are assumed to be frequently present in the Forsmark area. In the investigations for the power plants at Forsmark, horizontal zones were encountered in the surficial part of the bedrock /Carlsson, 1979/. Pre-investigations for location of SFR, and in this concept also for location of a KBS-3 repository, reveal the existence of zones of high hydraulic conductivity at depth. In the current scenario such large subhorizontal zones are assumed to be present at 150, 350 and 600 m depth at the shaft, c.f. Figure 8-2. The upper zone has been designated from the pre-investigations to have a hydraulic conductivity at $2x10^{-6}$ m/s. However, the extension and continuity of the zone is not fully known but the knowledge will be improved during the construction of the access shaft.

During the shaft construction, vertical pilot boreholes will not be needed, since such boreholes and borehole investigations already have been performed before the construction. However, in the holes drilled within the skin-zone of the shaft, pre- and post-testing of the hydraulic properties should be carried out in order to designate the rock quality of the skin-zone. Recordings of inflow to the shaft versus time, the location of the inflows together with the status of the shaft construction (excavation rate, grouting etc) will be carried out. Head measurements versus time in the adjacent boreholes equipped with multiple packer system are performed simultaneously with the inflow measurements.

Supplementary boreholes from the shaft into the surrounding bedrock are drilled to penetrate identified features and also to add more information on head distribution around the shaft. Of special interest in the designation of the rock quality is the groundwater chemistry. Experiences from SFR show that the groundwater quality exhibits an excess of chloride in comparison to the present Baltic sea-water. This situation is probably more pronounced at depth, and an assumption of increasing salinity with depth might be used in studying the changes of quality of the inflowing water to the shaft. An increase should than indicate that water from depth is introduced into the actual level while the opposite is valid when decreasing salinity is encountered versus time.

8.4.2 Results

The results from the construction of the access shaft given below are generic based on the knowledge gained during the construction of SFR and from conditions in mines and other underground facilities.

The results show that the upper horizontal fracture zone, similar to zone H2, recognized during the pre-investigation campaign, has a thickness of about 10 meters. The zone is located at a depth of 150-160 m depth and is highly permeable. Pregrouting had to be carried out at this zone and at another zone located deeper. No major problem was encountered when these zone were penetrated. However, even after grouting, an inflow of about 60 l/min from the upper zone was measured indicating that the grouted skin-zone would have a hydraulic conductivity of about $5x10^{-7}$ m/s. The fracture zone itself was tested to have a hydraulic conductivity of $2x10^{-6}$ m/s over distances of up to 300 meters.

Mapping of fractures and points of inflow at the shaft wall reveals that the distances between hydraulically conductive fractures in the defined rock mass on the average is in the order of 4-6 m in the upper part of the bedrock above the upper horizontal fracture zone. Below the zone, the distance is about 7-10 meters. At the repository depth (400-500 m), the average value is more than 14 meters. The inflow points are mainly concentrated to subvertical fractures having orientation N40W and NE and subhorizontal fractures.

The hydraulic conductivity measured in sections of 10 meters in boreholes in the centre of the shaft and in the surrounding bedrock close to and before the shaft sinking indicates a slight decrease versus depth, c.f. Figure 8-4. Besides the subhorizontal zones at 150-160, 350-360 and 600-610 m depth, other sections in the boreholes also exhibit increased fracture frequency and hydraulic conductivity.

The influence of the shaft construction on the rock quality in the skin zone might be illustrated as changes in hydraulic conductivity before and after the construction. The conditions before the construction is il-

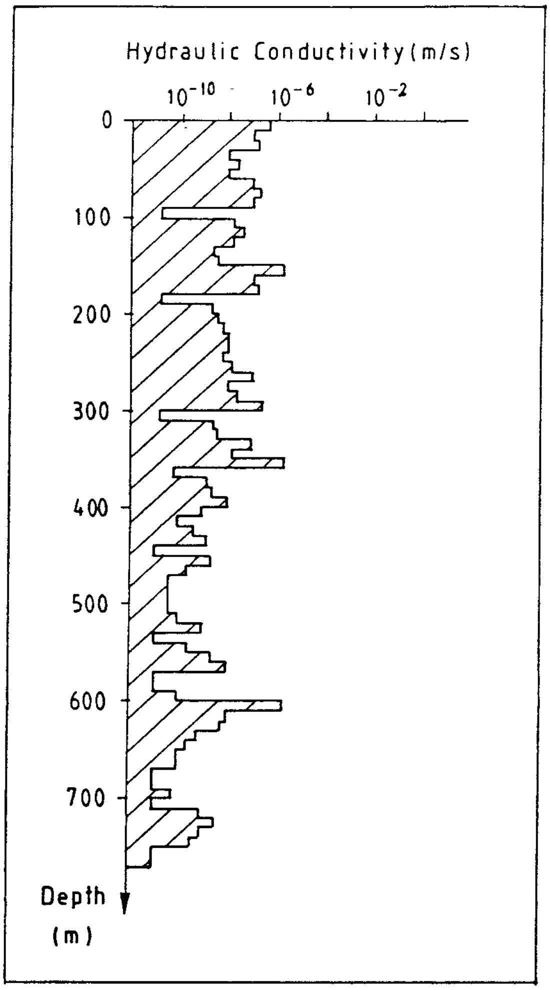


Figure 8-4 Generic diagram of hydraulic conductivity in 10 meter sections versus depth in a pilot borehole for the access shaft to a KBS-3 repository at Forsmark.

lustrated by the hydraulic conductivity distribution from the pilot boreholes given in Figure 8-4. After the construction, two types of investigations reveal the changes: renewed measurements in the boreholes within the created skin-zone and calculation of the gross hydraulic conductivity from inflow measurements and head monitoring. An average influence, expressed by a reducing factor of 0.20 on the hydraulic conductivity can be recognized.

In summary, the results of the rock quality designation during construction of access shaft implies an improved knowledge regarding:

- Position and properties of subhorizontal fracture zones

- Hydraulic properties of the virgin rock mass at and around the shaft

- Hydraulic properties and size of the skin-zone around the shaft

- Frequency and positioning of hydraulically conductive fractures

- Bedrock geology

- Size and orientation of fractures and their groupings

However, limited additional information is obtained on the actual repository rock block since the shaft is located outside the repository block.

8.5 ACCESS TUNNELS

8.5.1 Investigations

The actual construction of the repository at the prescribed depth starts with two parallel access tunnels at 500 m depth, c.f. Figure 8-5. Two pilot boreholes, each about 200 m in length are drilled after every 100 meter of blasting. In these boreholes, radar measurements are conducted and groundwater outflow measured in combination with spinner registration along the boreholes. After passing through the Singö zone, hydraulic measurements using multiple-packer system will also be used.

After passing the Singö zone, a limited number of target boreholes will be drilled in order to validate the existence and geometry of interpreted fracture zones from the pre-investigation stage. In these boreholes, a permanent multi-packer system will be installed, testing performed and head monitoring initiated. The main purposes of the target boreholes are to ascertain that the requested distance of 100 meters between major bounding fracture zones and the tunnels are kept and to test these zone.

After reaching the actual rock block for the repository (block A in Figure 8-3), the access tunnels will continue as one tunnel along one side of this block. The tunnel will be parallel to fracture zone 12 at a distance of 100 m. Pilot boreholes in the same manner as for the access tunnel will be drilled and tested before the tunnel construction. From this tunnel, long horizontal boreholes, about 700 m in length, are drilled to cover two additional peripheral sides of the repository block and the block itself. From the tunnels, vertical boreholes at distances of 150-200 meter should be drilled to check the existence of subhorizontal fracture zones above and below the repository.

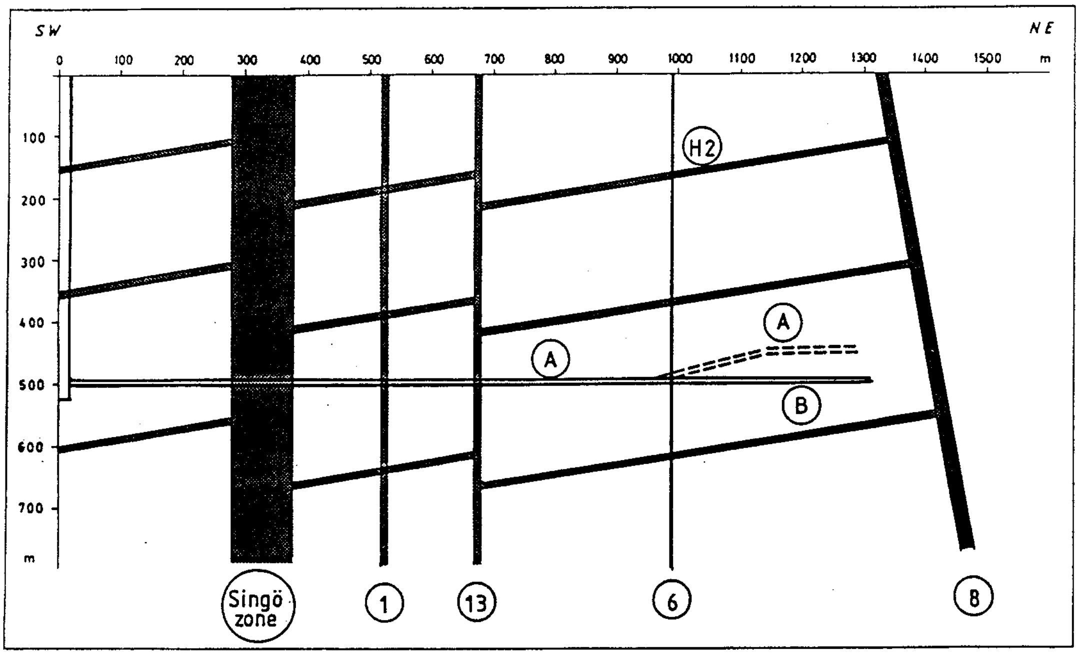


Figure 8-5. Cross-section through the repository, access tunnels and shaft, first lay-out and adjustments. The tunnel A will constructed, not tunnel B.

The layout of the KBS-3 repository at Forsmark in the block A and B implies a repository of 400-500 m extension in NW-SE direction. The pilot boreholes should have a spacing which allows radar cross-hole measurements to be conducted, that is not more than about 100 m. Figure 8-6 shows the lay-out of these long horizontal pilot boreholes where the centre hole should be sited at the designed central tunnel in the repository.

The main purposes of the long boreholes are to confirm the "respect distance" to the major fracture zones by use of radar measurements, identify and designate additional zones encountered and to establish a monitoring system to be active during the forthcoming construction of the central and storage tunnels.

The results from the long pilot boreholes will be the base on which the central tunnel is finally sited. However, this siting should preferably be along one of the pilot boreholes. Efforts should be assigned to cover the nature of fracture zone 3 which cut through the repository. There are two main principles on how the zone can be approached. Firstly, the zone 3 can be recognized as a major bounding fracture zone and the "respect distance" of 100 m should be kept. Secondly, the zone is considered as a minor one and can be accepted in the repository rock mass block. This means that only a limited number of the storage tunnels or parts of

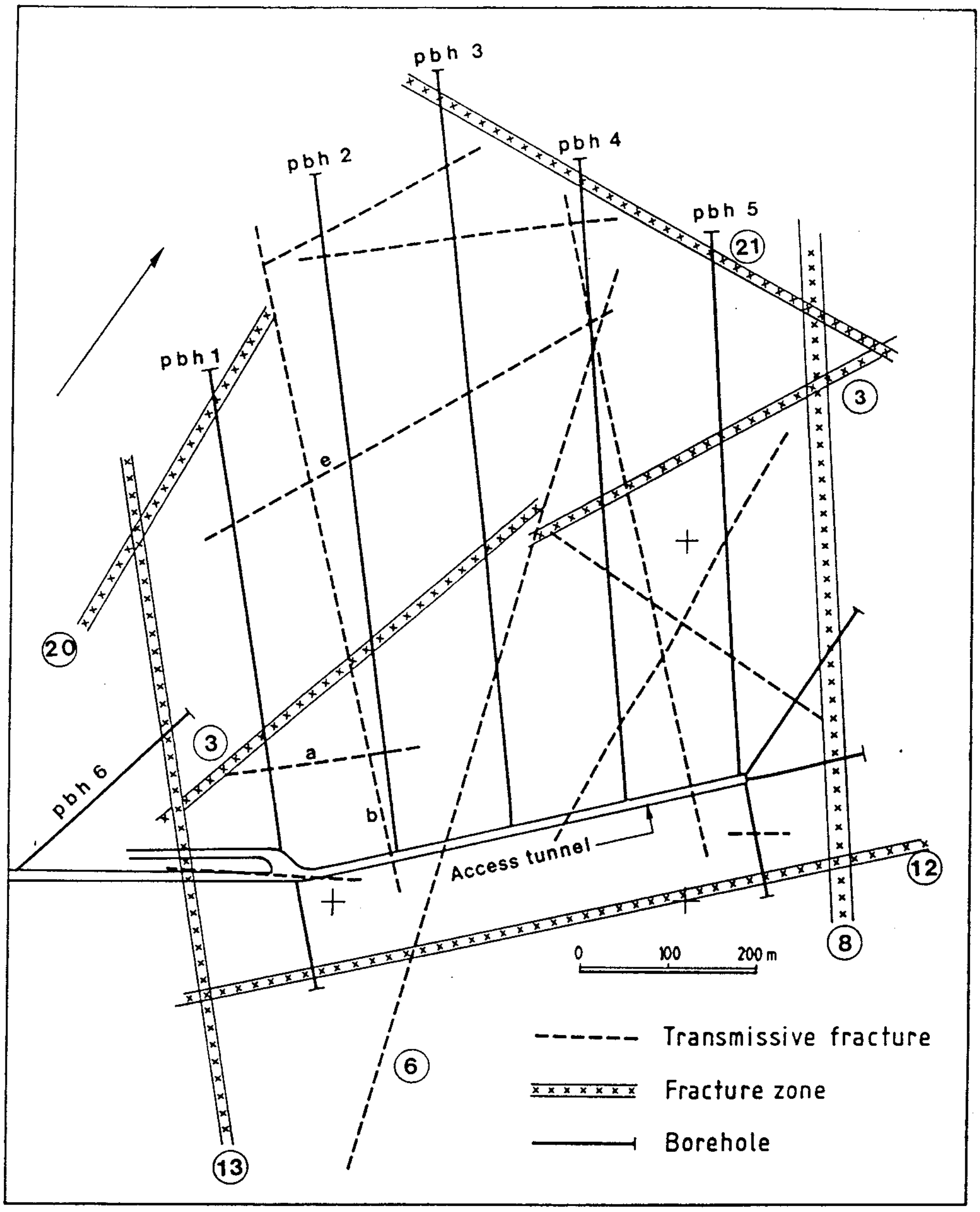


Figure 8-6. Preliminary model of fracture zones and main transmissive fractures within the repository block before start of construction of central tunnel.

them have to be discarded during the further construction. The second principle is taken in the current study and the "respect distance" of 100 m is not applicable.

A first lay-out of a centre main tunnel shows that it will penetrate both zone 6 and zone 3. Prior to the construction of that tunnel, a long pilot borehole will be drilled and the same investigations as mentioned for the other pilot boreholes would be conducted if the tunnel is not sited along one of the first drilled long horizontal pilot boreholes.

At this stage additional information should be obtained from target boreholes drilled from the access tunnel towards the bounding major fracture zones as illustrated in Figure 8-6. These boreholes will also be used to designate the rock quality of the bedrock within the zone covered by the respect distance.

8.5.2 Deviations and adjustments of the repository lay-out

The flexibility of a KBS-3 repository concept is manyfold as described in Chapter 5. The one hundred meters "respect distance" should continuously be checked by radar measurements or where necessary by a limited number of target boreholes. These are needed in order to designate the hydraulic properties of these zones and to monitor the impact on the groundwater head during repository construction. The later will then give information on possible contacts between the repository and the bounding fracture zones.

During and after the construction of the access tunnel in the actual rock block assigned for the repository, a thoroughly mapping of the bedrock fractures, water (and tracer) inflows etc is performed. This tunnel is the very first construction in the repository rock block as such, and a mapping should therefore primarily give information on the frequency of water transmitting features. This parameter is crucial and decisive when siting the deposition holes within the repository block (qualifying or disqualifying). Therefore a first estimation of the percentage of generic deposition holes to be accepted in a tunnel having the properties as designated by the access tunnel and the long horizontal pilot boreholes can be made. The result should be used in a revised study of the safety (safety assessment) of the whole repository, reinforcement necessary and repository qualification. Since such exercise probably is time consuming, a longer interruption in the repository construction is foreseen at this stage.

Deviation in orientation of the bounding zones may cause the size of the repository to in- or decrease. The best knowledge of these zones is undoubtedly obtained by target drilling. One way to do it will be in a fan array, c.f. Figure 8-6. Such drilling can be made before the long horizontal pilot boreholes are oriented and drilled.

A more delicate situation may arise if additional subhorizontal highly permeable zones are encountered at close distances below the repository. If such a zone will be found by the vertical boreholes, a new lay-out has to be considered at a new level with at least 100 m distance to these features. The possibility of the existence of such a zone of large extent should however be limited since most of the preinvestigation boreholes will be sited in order to detect such features.

The generic repository at Forsmark will be adjusted during the construction of the access tunnel in the rock block A, due to results from target boreholes. The geometry of the horizontal zones which are thoroughly checked by these boreholes will indicate that the respect distance will be less than 100 m according to the first lay-out of the repository at 500 m depth. Thus, in the current scenario the geometry of the first lay-out repository has to be changed. The SW-dipping fractures zones means the northeastern part of the repository has to be sited at a higher level than 500 m depth which is the level for the southwestern part as illustrated in Figure 8-5.

Deviations in geometry of the major bounding fracture zones, mainly concerning the direction of zone 8 will decrease the space available for the repository in comparison with the lay-out designed by the fracture-zone interception at higher levels. Further, the minor zones 6 and 4 are present together with a new one, zone 9, c.f. Figure 8-6. All these minor zones will be considered in the siting of the storage tunnels.

8.6 CENTRAL TUNNEL

8.6.1 Background for siting of central tunnel

From the results of measurements in the long pilot boreholes (pbh1-5) and target boreholes, a preliminary model of the fracture zones and main transmissive fractures (adverse features) is at hand, c.f. Figure 8-7. Fracture zone 6 has been penetrated by pbh3 and 4, c.f. Figure 8-7 and fracture zone 3 is identified as a 10 m wide zone but its continuity seems to be interrupted by zone 6. The bounding major fracture zone 20 has not been found in pbh2 and crosshole radar measurements indicate that it is cut by a fracture running more or less parallel to the pilot boreholes. Zone 4 was not recovered in the boreholes.

The picture thus obtained and given in Figure 8-7 indicates that major adverse features are found at spacing distances of 50 to 200 m. The mapping of the access tunnel gave a more detailed map of fractures but only those which could be interpreted as continuous between the boreholes and boreholes and tunnel are illustrated in the figure. After pilot borehole pbh1 was drilled and tested, multiple-packer system was installed and head monitoring initiated. During drill-

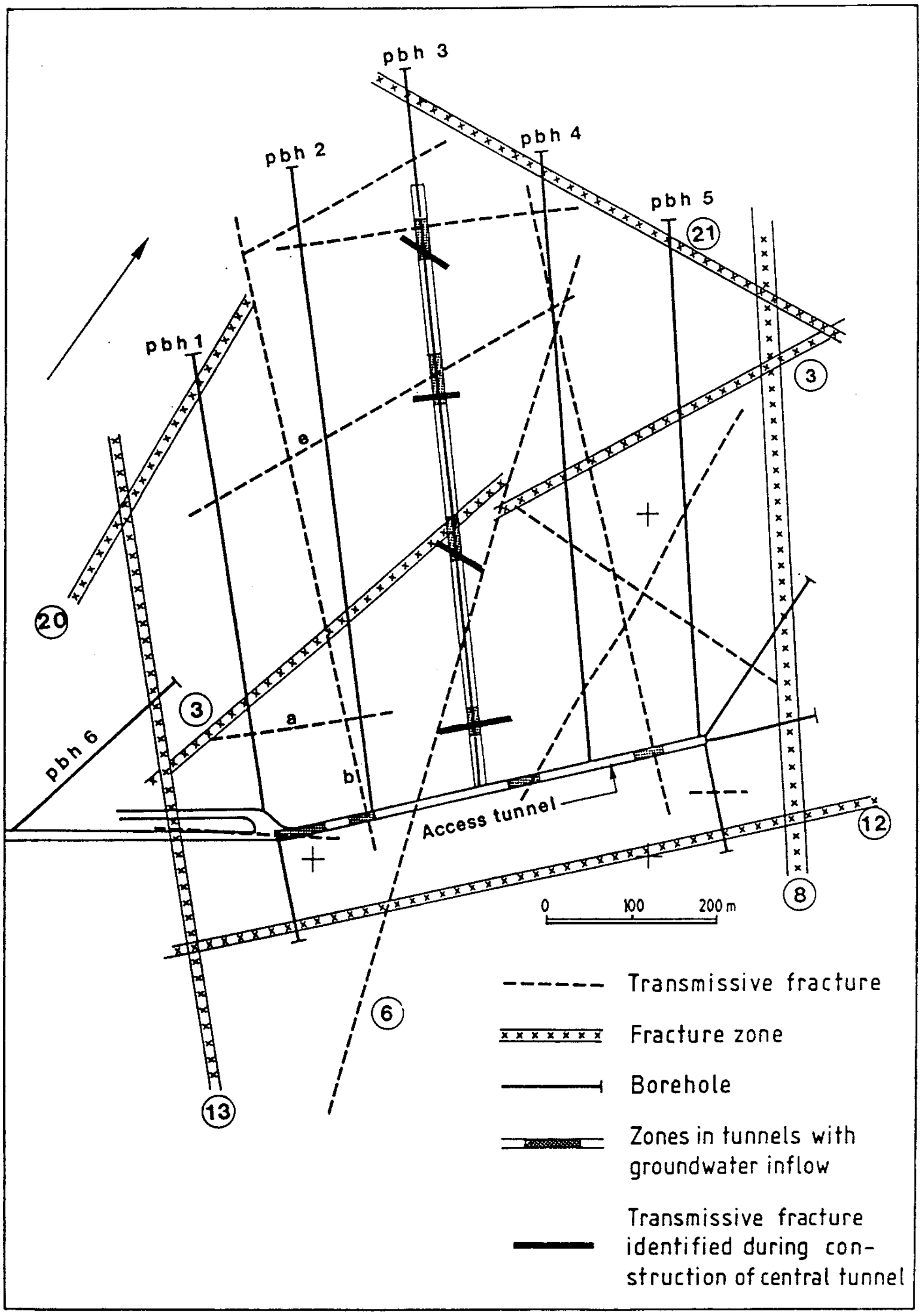


Figure 8-7. First lay-out of a KBS-3 repository at Forsmark.

ing of the next pilot borehole, changes in head were registered in sections including feature a in borehole pbh2. When fracture zone 3 was penetrated, head changes were also monitored in the section in pilot borehole pbh6 encompassing zone 13 and in pilot borehole pbh1 covering feature c, c.f. Figure 8-7. Similar observations were made during the drilling of the other pilot boreholes but in these cases more sections in the additional pilot boreholes were successively equipped.

The first lay-out of the repository was changed regarding the level of the northeastern part as mentioned in Section 8.5.2. The pilot boreholes did not give any indication that more adjustments were needed at the current stage since the bounding major fracture zones were recovered in boreholes and measurements as predicted. Thus the central tunnel was sited along the pilot borehole pbh3 with storage tunnels being laid-out more or less perpendicular to that tunnel and parallel to the access tunnel. However, more detailed information are needed before the final siting of these later tunnels can be made.

8.6.2 Construction of central tunnel and rock quality designation during and after construction

Before the construction of the central tunnel starts, the multiple-packer system installed in the pilot boreholes are revised and borehole pbh3 plugged along its whole length. In the sealed off sections in the target and pilot boreholes, non-sorbing tracers are injected. Each section should be injected with different tracers.

During the blasting of the central tunnel, groundwater head in the adjacent boreholes and inflow and tracers in the access tunnel are detailed monitored. Also in the constructed tunnel, inflow and tracer appearances are mapped and measured. Before crossing fracture zone 3, pre-grouting is carried out with simultaneously recordings of the amount of grout and water injected, responses in head etc.

The mapping and measurements of groundwater inflow have to be done in well-controlled forms. The use of plastic sheets, like in the Stripa mine /Abelin et al, 1987/, should be improved and installed as soon as possible after construction completion. New techniques in inflow measurements and tracer detection in detailed scale need to be developed.

In the current stage no drilling of observation or target boreholes are foreseen. The rock block assigned for the repository is instead kept as intact and free from boreholes as possible. The designation of the rock quality should instead be performed on the basis of non destructive measurements as geological mapping, inflow and tracer measurements, head observations and tunnel radar measurements. The latter technique needs to be developed to include cross-measurements between a tunnel and boreholes.

8.7 STORAGE TUNNELS

8.7.1 Siting background

In the first lay-out of a KBS-3 repository at Forsmark, the storage tunnels are sited parallel to the access tunnel from the central tunnel with a spacing of 33 m. As base for an improved lay-out of the storage tunnels, the results from the long pilot boreholes and from mapping of the central and access tunnels are at hand. Also all crosshole measurements, both geophysical and hydraulic, have provided valuable information all put together into an improved comprehensive conceptual model of fracture zone and transmissive fracture occurrence and geometry.

Figure 8-8 shows a model of transmissive major fractures based of all investigations performed before the siting of the storage tunnels. Based on this model, a

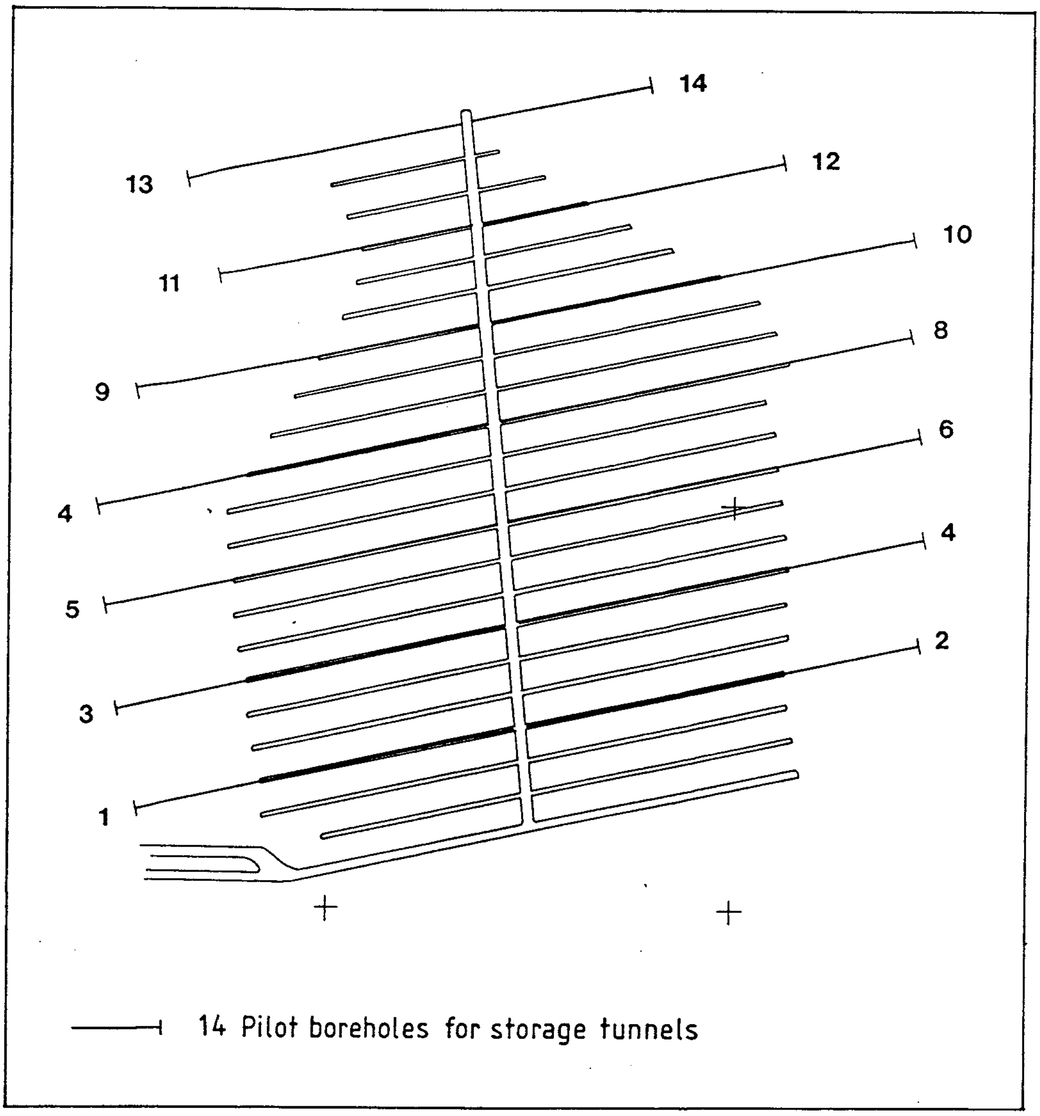


Figure 8-8. Lay-out of storage tunnels and pilot boreholes.

lay-out of the storage tunnel is made. In this lay-out, c.f. Figure 8-8, pilot boreholes for storage tunnels will be drilled along every third tunnel. These boreholes will reach and penetrate the major fracture zones bounding the assigned repository area.

8.7.2 Pilot borehole investigations

The 14 pilot boreholes designed in Figure 8-8 will be core drilled and subjected to different geophysical and hydraulic logging and crosshole measurements. The objectives are to reidentify already found transmissive fractures and fracture zones, to control the existence of additional transmissive fractures and to characterize fractures identified. In the process of testing and interpretation, section already equipped in the long pilot boreholes pbh1 through pbh6 will also be used. Thus a large variety of possible ways in testing and interpretation is available which calls for a comprehensive data storage and handling system in the evaluation process.

During the drilling of the pilot boreholes, there are a number of issues which need specially attention as the continuation of fracture zone 20, the hydraulic nature of zone 3. The first issue might be met by prolonging the pilot borehole sbh13.

In the end of the central tunnel, vertical boreholes are needed upwards and downwards to control the distance to horizontal fracture zones above and below the repository. These boreholes should be drilled in an early stage after the central tunnel is completed and used later when performing all tests in the spbh-boreholes.

The results given as a generic model of the occurrence and geometry of transmissive fractures is illustrated in Figure 8-9. In comparison to the model compiled from only the long pilot boreholes, the new one has adjusted some of the fractures and additional ones are considered. Also set of minor transmissive features are identified.

8.7.3 Final lay-out for construction

From the new model comprised, a final lay-out of the storage tunnels is made, c.f. Figure 8-10. This lay-out differs from the first one in the sense that 745 m more storage tunnels are now possible to construct due to improved knowledge of the bounding major fracture zones.

Some storage tunnels can at this stage be discarded or be considered to be constructed in a reduced length due to the occurrence of adverse features. Such parts of

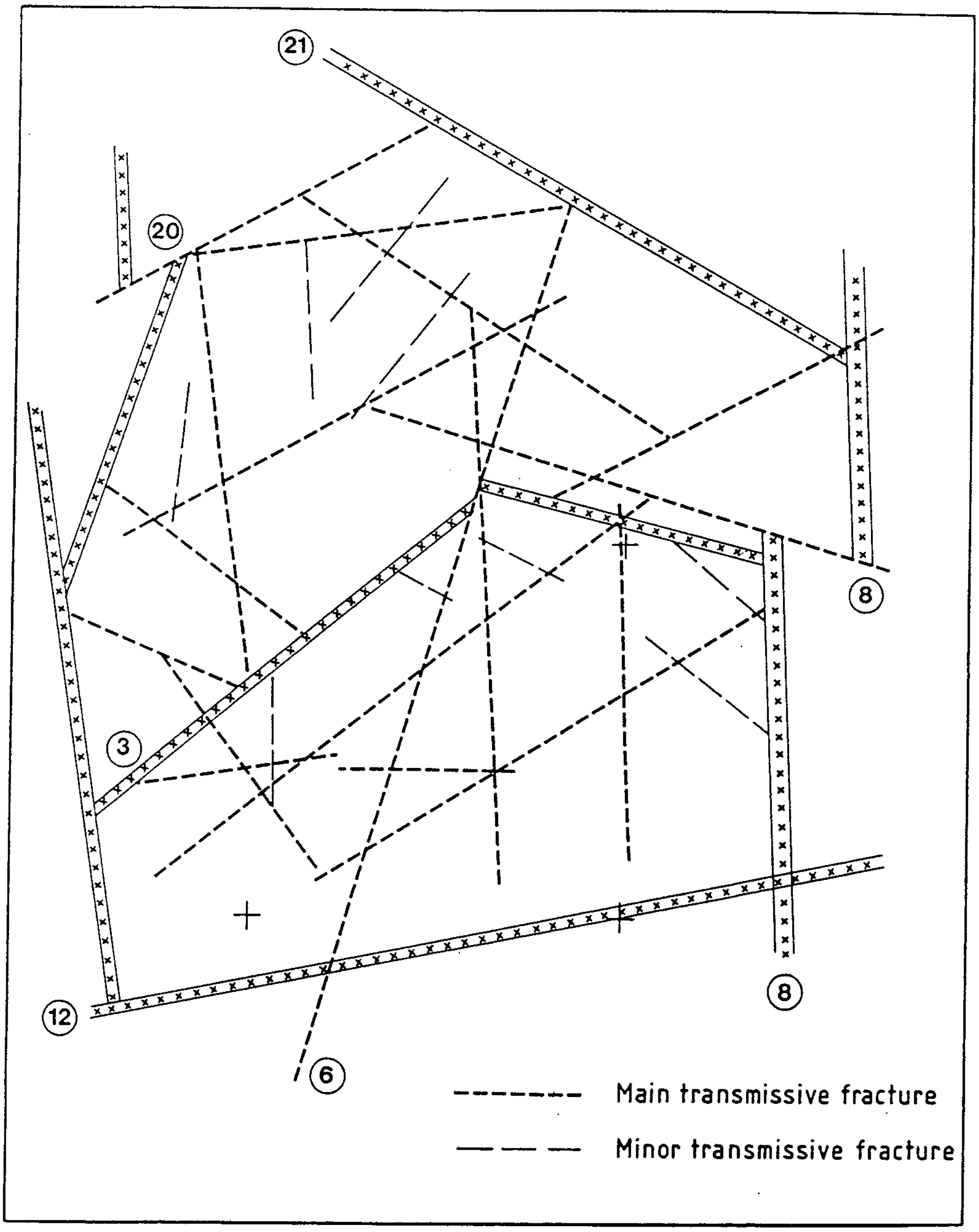


Figure 8-9. Model of fracture zones and main transmissive fractures within the repository block before start of construction of the storage tunnels.

the laid-out storage tunnel system are shown in Figure 8-10, in total 240 m. The total length of the laid-out storage tunnels is 9330 m (about 1500 deposition holes) and in the first disqualification about 2.5% is excluded, corresponding to about 40 sites for deposition holes.

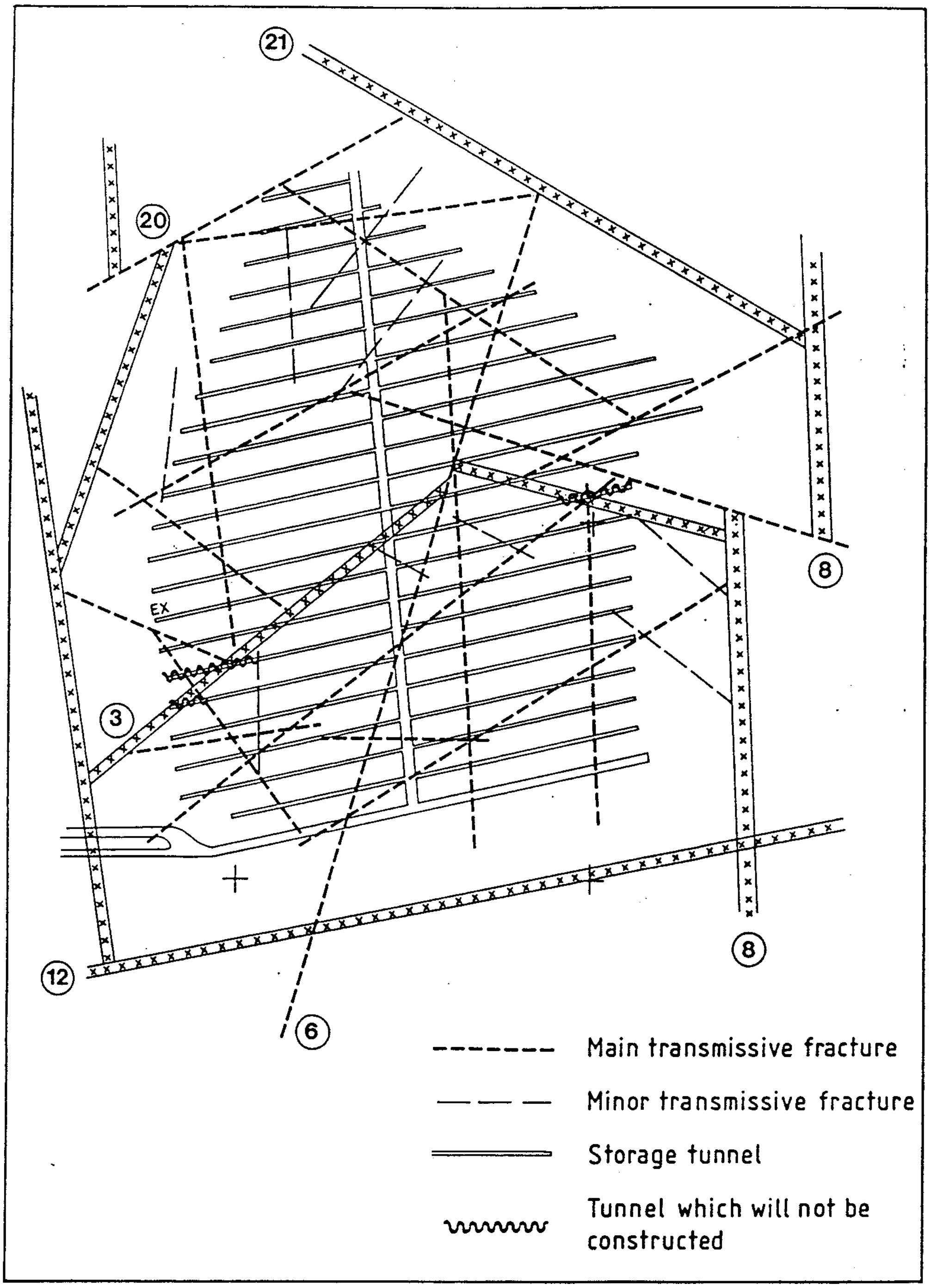


Figure 8-10. Final lay-out of storage tunnels based on the fracture model given in Figure 8-9.

8.7.4 Construction of the storage tunnels

When entering into the construction stage for the storage tunnels, it has to be recognized that after the construction of a certain number of tunnels, the siting and drilling will start of the deposition holes. This means that two operations will be going on simultaneously; construction and rock quality designation of the storage tunnels versus siting and rock quality designation of the deposition holes. In the following text the two activities are described separately.

The construction of storage tunnels start in the innermost part of the repository area. Before starting all pilot boreholes will be re-equipped with multiple packer system. Tracers will be injected in the sealed off sections. In those storage tunnels which are about to be constructed and where pilot boreholes are drilled (every third), the boreholes will be plugged along its whole length before the construction starts.

During tunnel construction, head changes in boreholes and water and tracer inflow in tunnels being constructed and already constructed should thoroughly be measured. During and after construction, the tunnels will be geologically detailed mapped. By the use of tunnel radar, measurements of the fracture system in the adjacent rock will be made.

After the construction of about 10 tunnels (5 m on each side of the central tunnel) siting of deposition holes will start, c.f. Section 8.8.

8.8 DEPOSITION HOLES

Deposition holes will be sited according to criteria on rock quality. Such criteria are currently not at hand but should be considered in good time before the final decision on repository lay-out will be taken. After a storage tunnel is constructed, a very detailed mapping of the tunnel should be carried out, especially the tunnel floor. A first lay-out of the deposition holes with a spacing of 6 m will reveal the holes which will be penetrated by main transmissive fractures identified or which are sited in zones or areas of highly fractured rock. Such sites for deposition holes will be excluded.

At each fifth site a pilot borehole of the double depth of the deposition hole will be drilled. Tunnel EX in Figure 8-10, for example, will include eight pilot boreholes (dpbh1-8), see Figure 8-11. These holes constitute a vertical section which is investigated by cross-hole hydraulics and geophysics as well as single-hole tests. These investigations will reveal if and where adverse features like transmissive fractures are present at the site for the remaining deposition hole. A generic result for tunnel EX shows that 14% of the 43 sites will be excluded in the first stage and another 35% in the second (after cross-hole investigations), thus in total almost half of the sites. This is a high figure but should be regarded as very generic. Further, every sites which include an adverse feature, defined

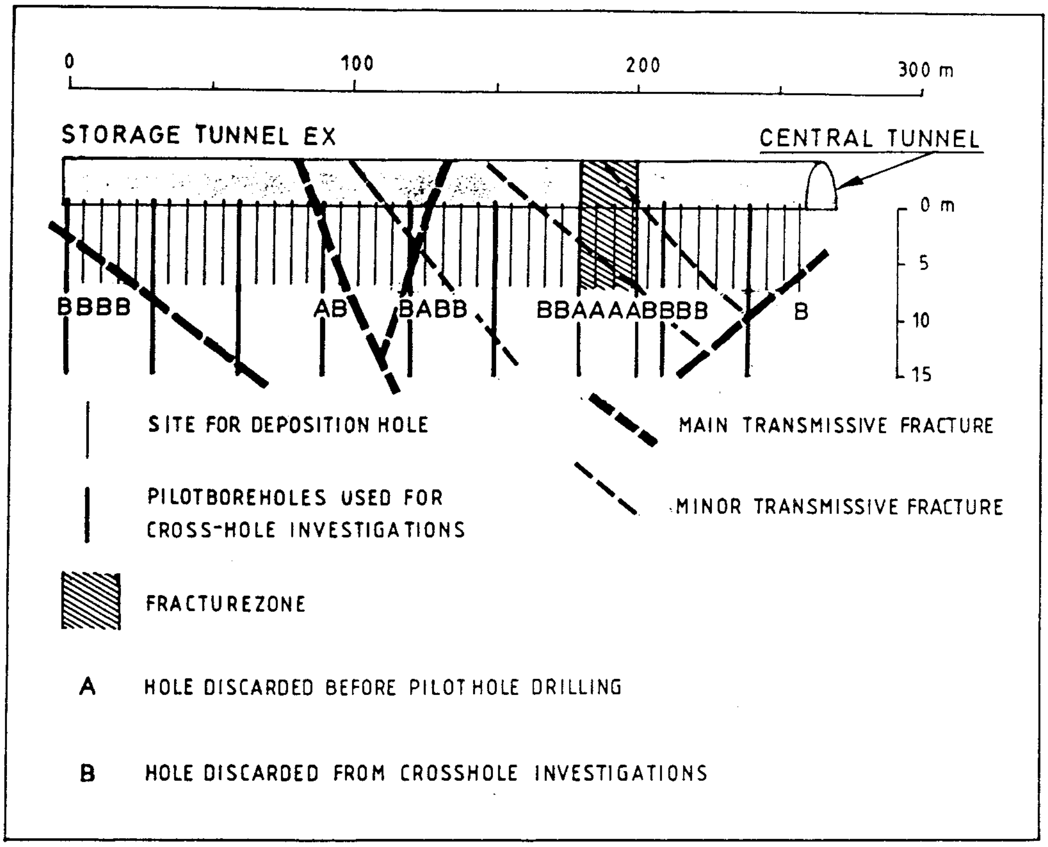


Figure 8-11. Profile along storage tunnel EX in Figure 8-10 with pilot boreholes for deposition holes and adverse features identified by geological mapping and crosshole measurements.

above, are excluded but these features might be reinforced by engineering measures or might be accepted (criteria needed).

After crosshole measurements and a second stage of site discarding, the investigations are focused on the accepted sites for deposition holes. As a first step, a pilot borehole is drilled at each site and tested. Such a test can hydraulically be done and evaluated in such a way that the results might be used in a stage 3 of sites acceptance as deposition holes. However, in this case, criteria or recommendations are needed. If the results of the tests turn out to be acceptable, that means for instance no transmissive fractures observed and no hydraulic connections established, the deposition hole is drilled (1.5 m in diameter).

After final construction of each deposition hole, detailed geological mapping is performed. Further, the entire hole is tested hydraulically and the results compared with some kind of criteria set up. There might also be a need for testing of individual or part of individual transmissive fractures within the hole. Testing technique and equipment for such testing need to be developed. These tests are needed when effects of transmissive parts (aperture variation) in a fracture should be considered in a disqualification process.

9 GENERIC ROCK QUALITY DESIGNATION OF A WP-CAVE REPOSITORY

9.1 BACKGROUND

The geology and hydrogeology in the SFR-area is outlined in Chapter 8. The siting of a WP-Cave repository in the Forsmark area is assumed to be at the same place as the siting of the KBS-3 repository. Data on identified fracture zones and gross hydraulic conductivity are given in Section 8.1.

9.2 ROCK QUALITY DESIGNATION STAGES

The repository is designed to host 1500 tons of spent fuel, i.e. only one WP-Cave is required. The dimensions of the repository are those given in Chapter 6 and are summarized in Table 9-1.

Table 9-1. Dimensions and geometric properties of a WP-Cave repository at Forsmark.

Component	Depth below sea level (m)	Dimension (m)
Hydraulic Cage		
Top level	132.5	
Height		450
Diameter		230
Bentonite-sand barrier		
Top level	220	
Height		275
Diameter		130
Waste storage cavern		
Top level	282	
Height		155
Diameter		64

In the following description of the rock quality designation, the construction of the WP-Cave will be divided into five stages comprising:

1. Construction of access shafts and tunnels
2. Construction of annular tunnels
3. Construction of the hydraulic cage
4. Construction of the bentonite-sand barrier
5. Construction of waste storage cavern

When designating the rock quality, the general concept is to identify adverse features in the large scale and

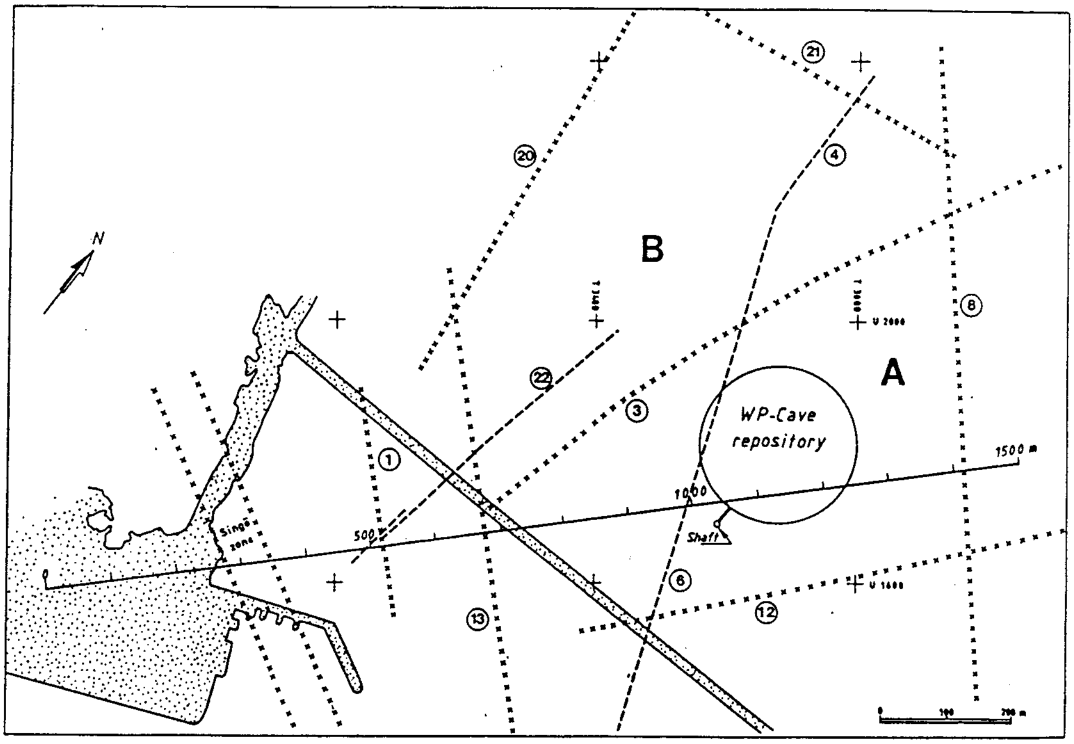


Figure 9-1. Preliminary lay-out of a WP-Cave repository with access shaft and tunnels in the Forsmark area.

then to obtain more detailed knowledge as the repository approaches the very ultimate stage. The designation further includes the establishment of the interconnectivity between water transmissive fractures and between major bounding fracture zones and water transmissive fractures in the assigned rock block for the repository. The use of non-destructive investigative methods in the designation process is also included in the current concept. A minimum of boreholes drilled from the repository block into bounding major fracture zones are aimed at. Instead, boreholes should be sited and drilled where tunnels and shafts will be considered in the forthcoming stages.

In a first lay-out of a WP-Cave repository, illustrated in Figure 9-1 and 9-2, the major vertical fracture zones are avoided. A respect distance of 100 m to the major vertical fracture zones 8 and 12 is kept. Within the repository area there are two subhorizontal zones present. These zones will be penetrated by the repository as illustrated in Figure 9-2.

The access to the repository and to its different components are achieved by establishing shafts from an artificially constructed island in the Baltic. In comparison with the KBS-3 lay-out, the WP-Cave lay-out will not include long access tunnels from the mainland.

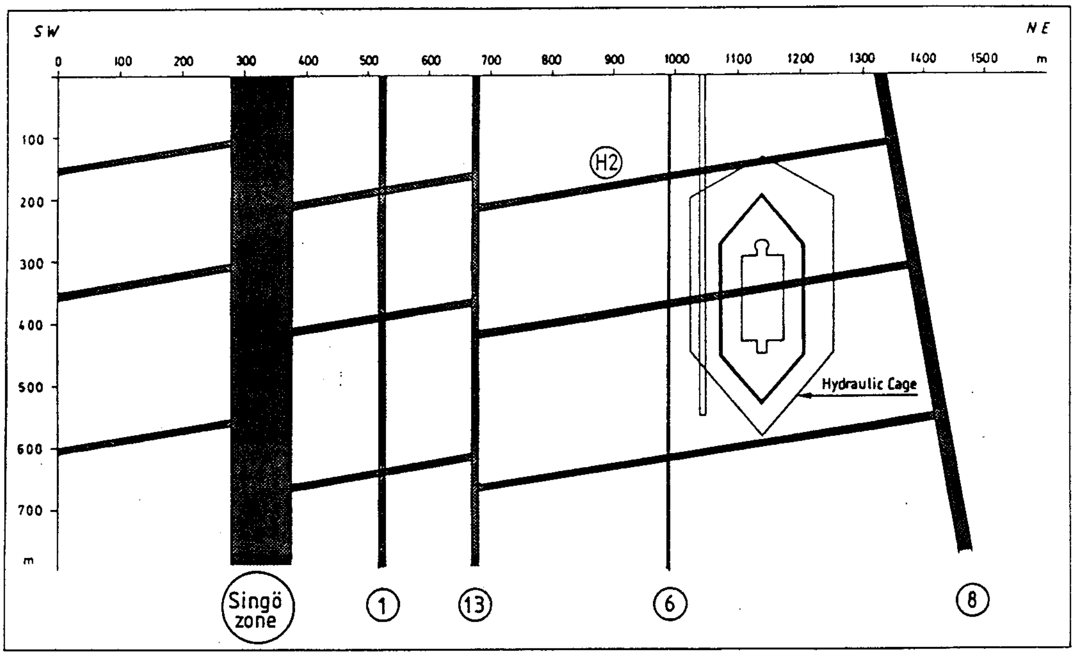


Figure 9-2. Cross-section through a WP-Cave repository with access tunnels and shaft, first lay-out.

9.3 ACCESS SHAFT

The investigation prior and in connection to the construction of the access shaft to the hydraulic cage will comprise the same techniques as described for the access shaft to the KBS-3 repository. These investigation comprise drilling and testing in vertical boreholes at the site of the shaft and at various distances from the shaft.

The main rock quality designation will take place during the actual shaft construction. In vertical boreholes located at different distances from the shaft, head measurements during the construction will reveal the degree of hydraulic connection between the shaft and the surrounding rock mass and adverse features recognized. The access shaft is located in the same rock block as the WP-Cave repository and information obtained from the shaft is thus valuable and applicable on the further construction and designation of the annular tunnels and the hydraulic cage. The investigations will reveal the existence of two major subhorizontal fracture zones as predicted from the pre-investigations.

9.4 ANNULAR TUNNELS

Three annular tunnels along the periphery of a circle with radius 115 m will be constructed at the levels of

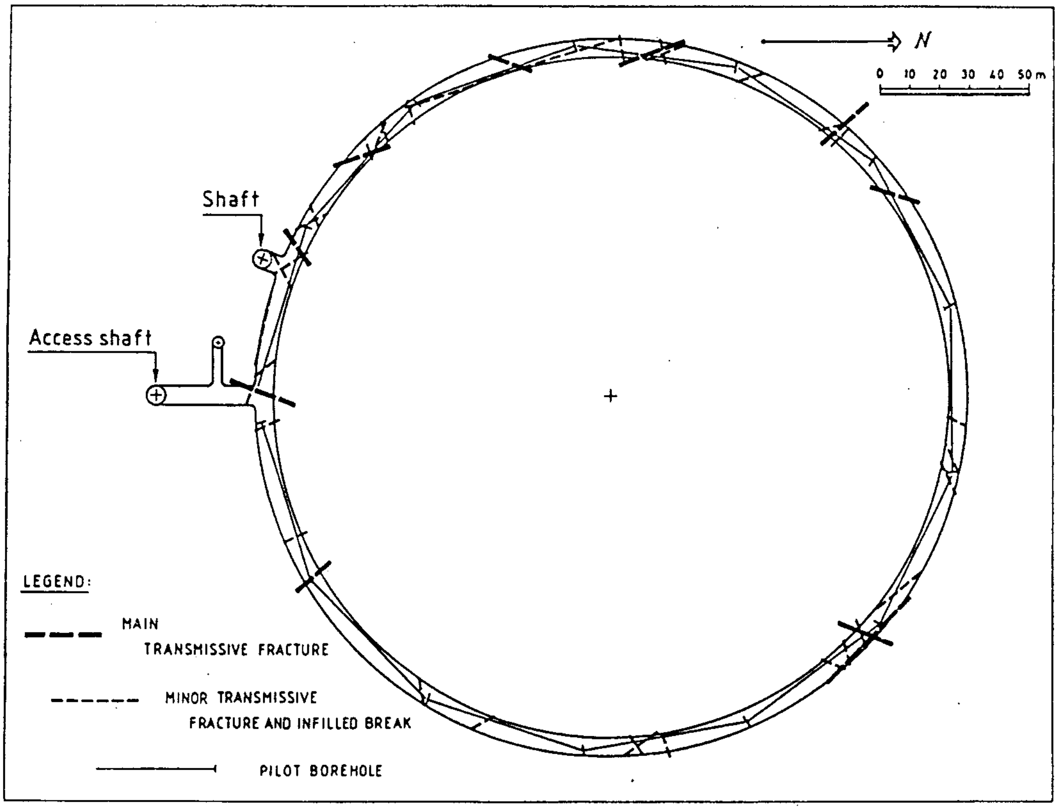


Figure 9-3. Lay-out of pilot boreholes in the upper annular tunnel and results obtained from borehole testing and tunnel mapping.

200, 325 and 450 m below sea level. The construction will start with the upper tunnel. Pilot boreholes of 55 m length will be drilled at the front of the tunnel at intervals of 55 m. The pilot boreholes will be tested in the same way as the pilot boreholes used in the designation of the access tunnel at the KBS-3 repository. Figure 9-3 illustrates the pilot borehole lay-out and the generic results obtained (for the annular tunnel at 450 m level).

Before the construction of the middle annular tunnel (at 325 m level) starts, two groups of observation boreholes will be drilled. The first group of boreholes are sited at positions for future drainage curtain boreholes. In total 16 vertical, 140 m long boreholes will be drilled with a spacing of about 45 m and 16 inclined boreholes upwards are drilled. These latter boreholes will have a length of about 130 m. The borehole are core-drilled and tested by both single and crosshole techniques. The vertical borehole will stop about 10 meter above the middle annular tunnel.

The second group of observation boreholes comprises 8 boreholes drilled from the annular tunnel, 4 vertically upwards and 4 inclined downwards. The purpose of these boreholes is to establish observation points in the bedrock outside the hydraulic cage to be used mainly during the construction of the drainage curtain, c.f. Section 9.5.

The results from the drilling and testing of the observation boreholes will be compiled in a conceptual model of adverse features in- and outside of the future curtain between the top of the hydraulic cage and the middle annular tunnel. Based on this model, sections will be sealed off by packer and head monitoring equipment installed. In the sections non-sorbing tracers might be added to facilitate future studies of interconnectivity between and along identified transmissive zones when constructing the two other annular tunnels and the final borehole curtain.

During construction of the middle annular tunnel, which will be performed in the same way as the upper one, measurements of changes in head will be conducted and water and tracer inflow in the tunnel will be observed. This information will reveal if and where fracture interconnectivity over long distances exists.

Before the construction of the lower annular tunnel starts, two groups of observation boreholes are drilled. The first group is drilled vertically downwards as in the case of the upper annular tunnel and the second drilled inclined downwards. The same procedure of testing and monitoring installation is performed as for the earlier drilled boreholes.

The construction of the lower annular tunnel is performed in the same way as the two previous. However, no observation boreholes of the first group are drilled at this tunnel. Instead the drilling of the boreholes in the actual drainage curtain will start by performing the lower inclined boreholes, totally 230 boreholes. During this work which is described in Section 9.5, measurements of head changes are performed in the observation boreholes together with measurements of changes in groundwater inflow in the annular tunnels and the access shaft. Before the drilling of the drainage curtain boreholes starts, observation boreholes of group 2 are drilled vertically downwards, tested and equipped with packers and tracers, c.f. Section 9.5.

The procedure of head monitoring is continued when the remaining boreholes in the drainage curtain are drilled from the middle and upper annular tunnels. The curtain boreholes should be subjected to geophysical investigation in order to map the existence and geometries of transmissive zones in the curtain. The function of the curtain is to effectively drain and short-circuit the groundwater flow in the bedrock leaving a rock volume over which no hydraulic gradient will be present after closing of the repository. Therefore, a good interconnection between the curtain and transmissive features in the bedrock is required. The testing of the function of the curtain is described in the following section.

9.5 HYDRAULIC CAGE

After and during the construction of the annular tunnels and the drainage boreholes, in total 920 boreholes, the hydraulic cage should be designated. Testing of the drainage curtain boreholes should therefore be performed. A good interconnection between the boreholes and the bedrock is aimed at so the drainage curtain can acts as the designed hydraulic cage, equalizing the hydraulic gradient over the repository area.

The designation should aim at finding and quantifying transmissive features in the bedrock, their water yielding capacity and connection with the boreholes, as well as their function as drains in the rock. The process will therefore primarily involve monitoring of groundwater inflow to the individual boreholes and borehole testing. In order to verify the effectiveness of the cage, groundwater head should be monitored within the circumscribed rock volume. To do this, boreholes would have to be drilled into the rock volume surrounded by the cage. However, such an operation will introduce adverse features in the rock volume assigned for the repository. Therefore other means should be used to measure and monitor the groundwater head in that volume.

One way to monitor the head within the repository rock volume would be before constructing the shafts for the bentonite-sand barrier. Along the centre line for the two access shafts to the bentonite-sand barrier, pilot boreholes from the ground surface should be drilled and equipment for head monitoring installed before the stage where the bentonite-sand barrier is constructed. The distance from the hydraulic cage to these boreholes will be at closest 55-60 m. It should be considered if those boreholes even should be drilled before the construction of the annular tunnels start. In such a case, sections in these boreholes could be injected with non-sorbing tracers so that interconnectivity might be estimated where the tracers later are identified in the annular tunnels or in the drainage curtain.

The cage should, beside drain the rock volume assigned for the repository, also short-circuit the flow path in the surrounding bedrock. To designate the effectiveness of the objective, boreholes with head measurements are needed outside the cage. Such boreholes will than be adverse features in what is called the respect distance to the surrounding fracture major zones. However, the presence of observation boreholes outside the cage will not affect the function of the cage nor will it create obstacles for the repository within the circumscribed bedrock volume. Thus observation boreholes will be drilled out from the annular tunnels to form a system for groundwater head monitoring outside the hydraulic cage (group two observation boreholes described in Section 9.4).

Figure 9-4 illustrates the observation boreholes drilled from the annular tunnels. In total, 16 observation holes will be drilled, tested and equipped with head monitoring system. These boreholes will be drilled at the same time as the pilot boreholes in the drainage curtain, c.f. Section 9.4. That means that they could serve as observation boreholes during the construction of the drainage curtain. Before the drilling of the curtain, the sealed off sections in these observation boreholes can further be injected by non-sorbing tracers. Observation of inflow in the drainage curtain will then reveal the existence of interconnections of fractures between the observation boreholes and the hydraulic cage.

Additional testing methods (designation methods) can be used to designate specific features in the cage. This can be done by the use of multiple packer system in groups of neighboring boreholes and hydraulic interference testing.

9.6 BENTONITE-SAND BARRIER

Pilot boreholes for the two main access shafts to the bentonite-sand barrier will be drilled, tested, and equipped with packers and tracers before the construction of the annular tunnels to the hydraulic cage commences as described in Section 9.5. During the construction of these tunnels, groundwater head changes versus time will be monitored in these pilot boreholes.

Two slot tunnels at 275 and 450 m level are constructed at the periphery of a circle of diameter 130 m. In fact the circle is a 16-sided polygon as illustrated in Figure 9-5. These tunnels will be excavated without any pilot boreholes. During the excavation groundwater flow will be recorded into the tunnels and head monitored in the observation boreholes from the annular tunnels. The rock volume in vertical direction between the two slot tunnels will be investigated by 16 vertical boreholes in the same way as described in Section 6.5.

During the excavation of the slot for the bentonite-sand barrier, a thoroughly mapping of the bedrock will be carried out in combination with radar measurements between the slot and the boreholes in the drainage curtain. The major horizontal fracture zone encountered at 325-350 m level in boreholes will also be found in

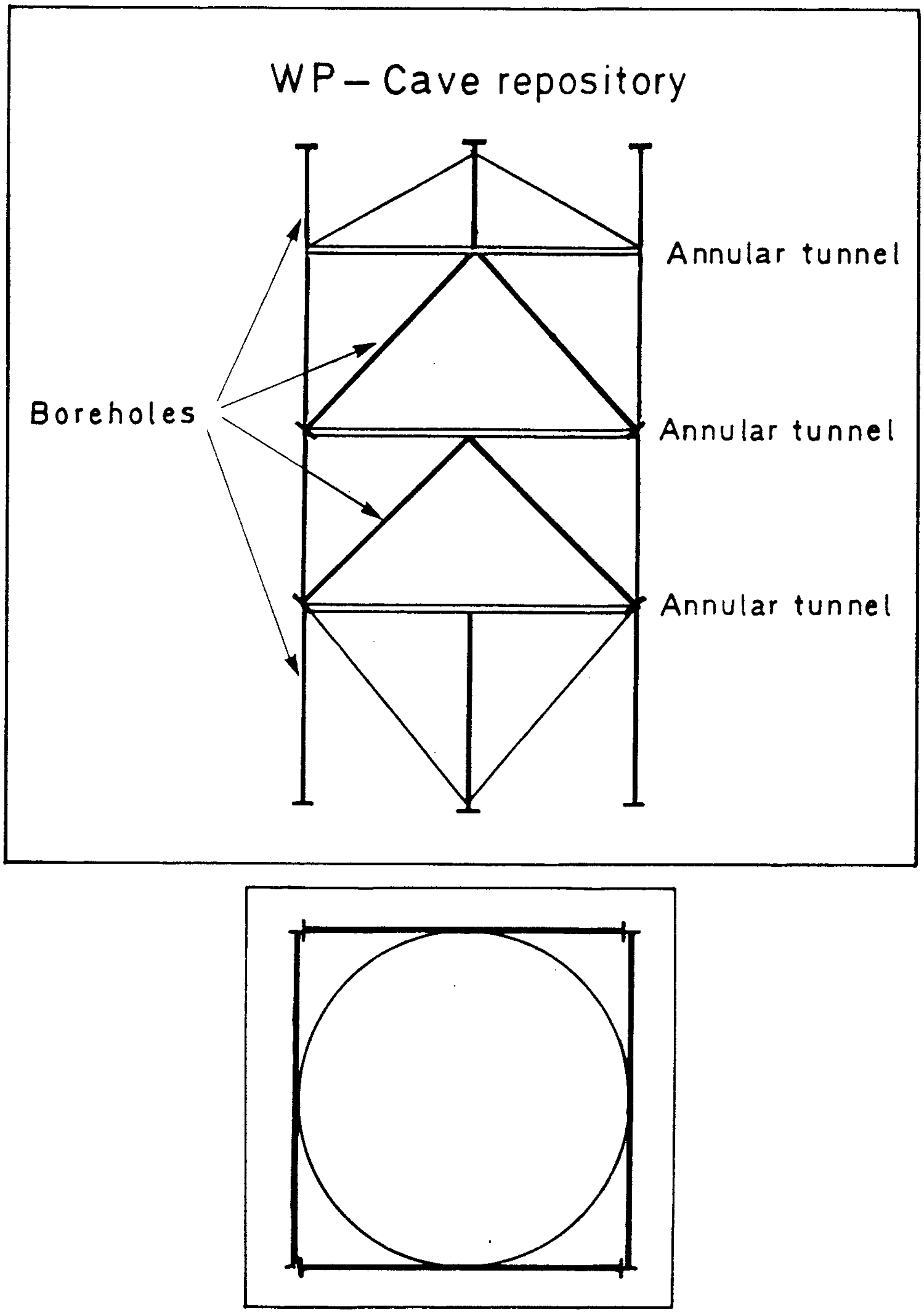


Figure 9-4. Lay-out of observation boreholes from the annular tunnels into the surrounding bedrock outside the hydraulic cage.

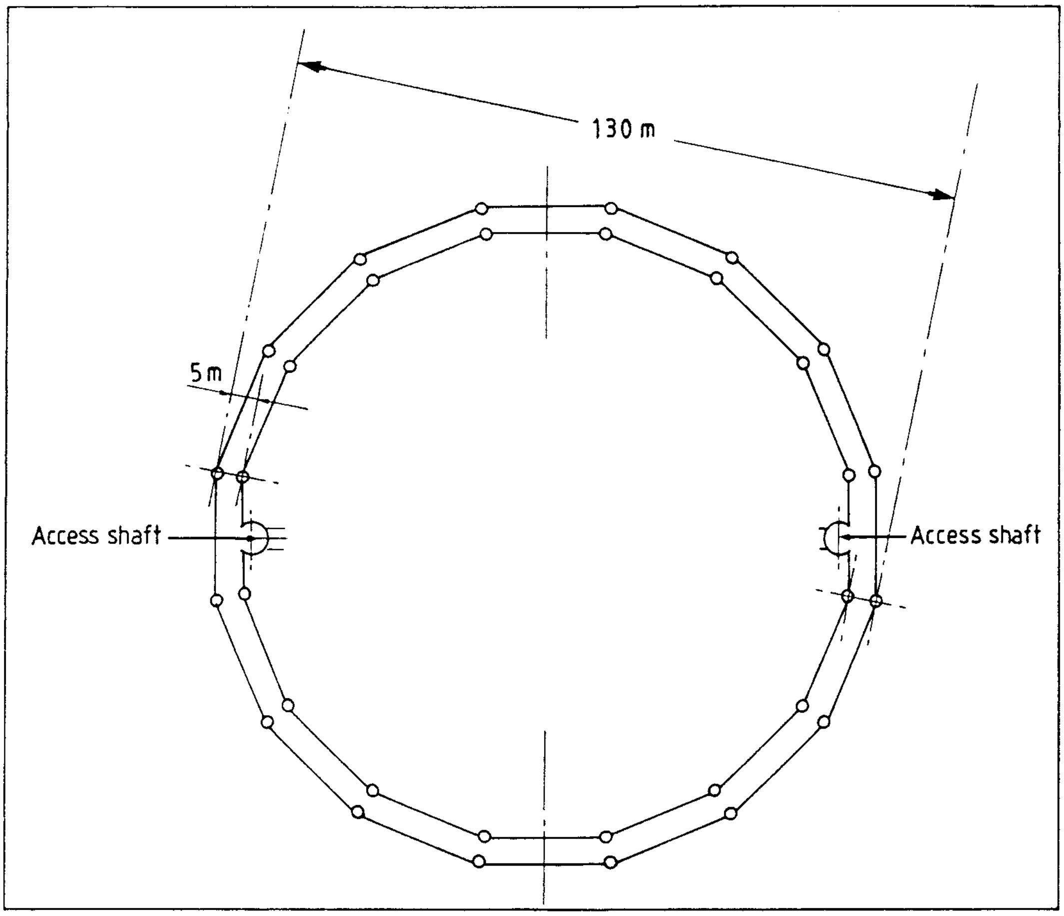


Figure 9-5. Slot tunnel in the bentonite-sand barrier.

the slot. This zone will be subjected to a testing campaign comprising hydraulic interference testing along the zone between the hydraulic cage and the bentonite-sand barrier.

9.7 STORAGE CAVERN

In the storage cavern, the major efforts will be devoted to the designation of the horizontal fracture zone crossing the cavern at the level of 340-350 m. At this stage in the WP-Cave construction, no free flowing groundwater should theoretically be found within fractures or fracture zones due to the drainage by the hydraulic cage and the sealing by the bentonite-sand barrier. Hydraulic testing of the horizontal zone should therefore be conducted by injecting water in boreholes. The holes will be pilot boreholes for the central tunnel. A detailed geological mapping of the entire system of tunnels and shafts comprising the storage cavern will be carried out. It should be remembered that before the storage cavern is constructed, a huge stock of information from the construction of the cage and the slot is available. A predictive model of the geology and fracture system in the storage cavern is therefore possible to set up. The construction of the storage cavern can thus be recognized as a large validation of the predictive model set up.

Since the bedrock around the cavern will be heated to 150 ^{o}C, properties for heat and vapour transport should also be designated.

10 SOME PRELIMINARY MODELLING EXERCISES

10.1 GROUNDWATER INFLOW TO A TUNNEL

There are two main hydraulic parameters which reflect the hydrogeological conditions within a bedrock during the construction of an underground facility, namely the groundwater inflow and the groundwater head. The first parameter is usually measured as the sum of inflow from various sections of a tunnel or an underground construction. It can also be monitored as a direct inflow into a sealed off minor area of a tunnel wall as has been carried out in the Stripa Project /Abelin et al, 1987/. The second parameter, the groundwater head, is usually measured in sealed off sections in boreholes drilled from the ground surface or from underground facilities. Indeed, there is no other way of measuring this parameter than through boreholes.

For rough calculations of the groundwater inflow Q per m of a tunnel at depth h meter below the groundwater table, the following expression is commonly used /Goodman et al, 1965/:

$$Q = \frac{2 \pi K_{eff} h}{\ln \left(\frac{2h}{r_w}\right)} \qquad (10\text{-}1)$$

The effective hydraulic conductivity, K_{eff}, or gross hydraulic conductivity of the bedrock is an integrated mean value over the rock assumed as a porous media. The radius of the tunnel r_w expresses the radius of a circle having the same area as the actual tunnel. A 25 m^2 tunnel at 500 m depth should be expected to have an inflow versus effective hydraulic conductivity as illustrated in Figure 10-1.

A gross hydraulic conductivity value can be estimated from inflow measurements. Normal ventilation of tunnels will transport water in vapour phase corresponding to an inflow of 0.02 l/sec and kilometre tunnel. This value is a mean value obtained from the Stripa experiments. Such an inflow value would correspond to an effective hydraulic conductivity of 4.10^{-11} m/s.

It should be possible during a repository construction to establish measurement points at every 50 m tunnel. A daily recording of the total inflow of water in a 50 meter tunnel at 500 m depth should be around 20 l when the hydraulic conductivity is in the order of 1.10^{-11}

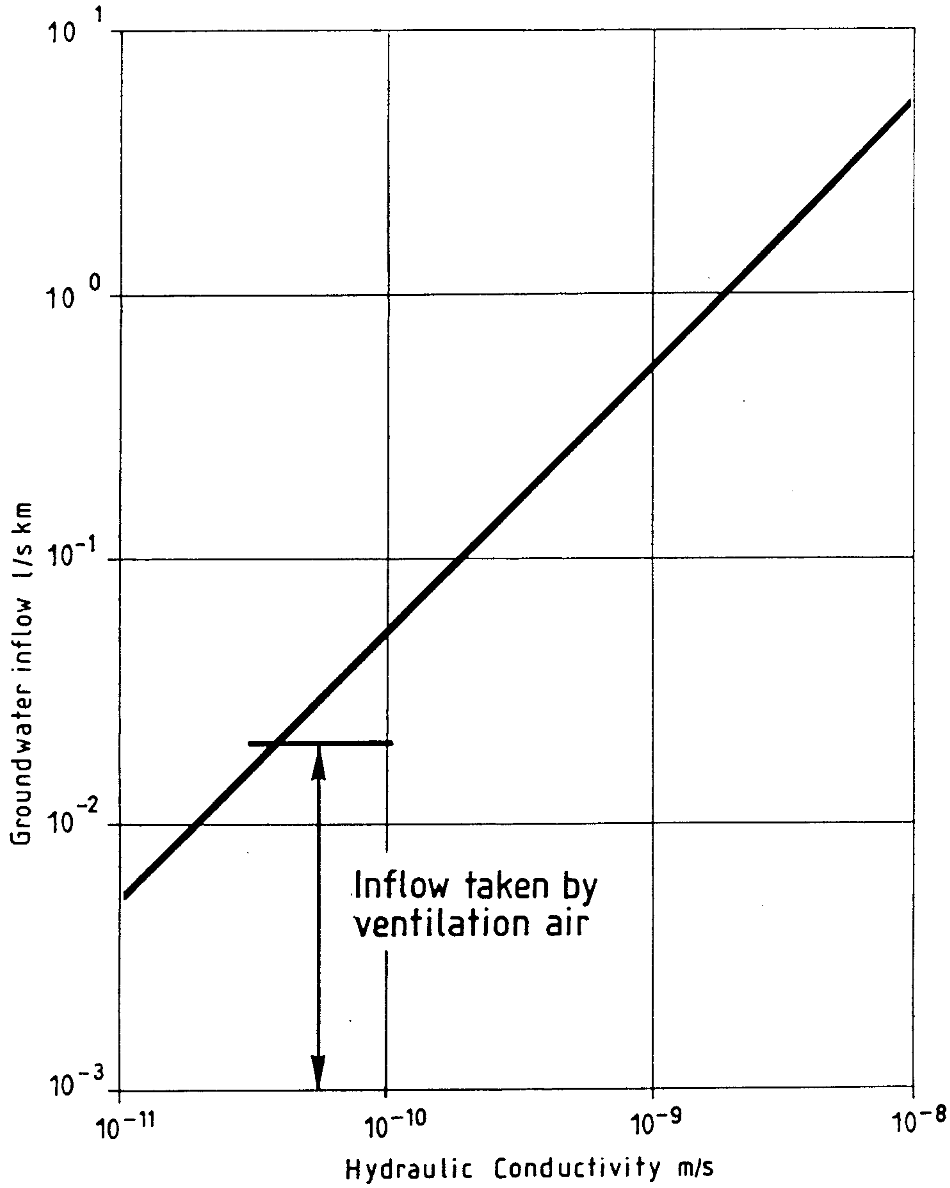


Figure 10-1. Inflow (l/sec and km tunnel) to a tunnel located 500 m below the ground water level versus effective hydraulic conductivity (m/sec).

m/s for the surrounding rock mass. However, it is likely that the amount of inflowing water corresponding to this hydraulic conductivity will be transported by the ventilation air and thus not measurable as water flowing at measurement stations. The lower limit for hydraulic conductivity estimation by inflow measurement in ordinary ventilated tunnels could be set to correspond to a real water flow measured as twice the water amount being transported by the ventilation air. That corresponds to about 180 l/day and 50 m of tunnel, meaning an effective gross hydraulic conductivity of around 1.10^{-10} m/s when the ventilation moisture transport is included. It should be noted that the figures given refer to condition illustrated as one single tunnel at a depth of 500 m below the groundwater table.

10.2 NUMERICAL MODELLING OF GROUNDWATER INFLOW

Water inflow to a tunnel from a minor fracture zone has been illustrated and estimated by numerical modelling. A one meter wide fracture zone, connected to major bounding fracture zones, surrounding the repository bedrock, is penetrated by a tunnel. The tunnel is located at 500 m depth and 100 m from a bounding major fracture zone, c.f. Figure 10-2. The inflow of water from the minor fracture zone is calculated at 3.5 l/min when the zone has an average hydraulic conductivity of 10^{-7} m/s. When an additional parallel tunnel is constructed 33 m from the first one, the inflow to the first tunnel

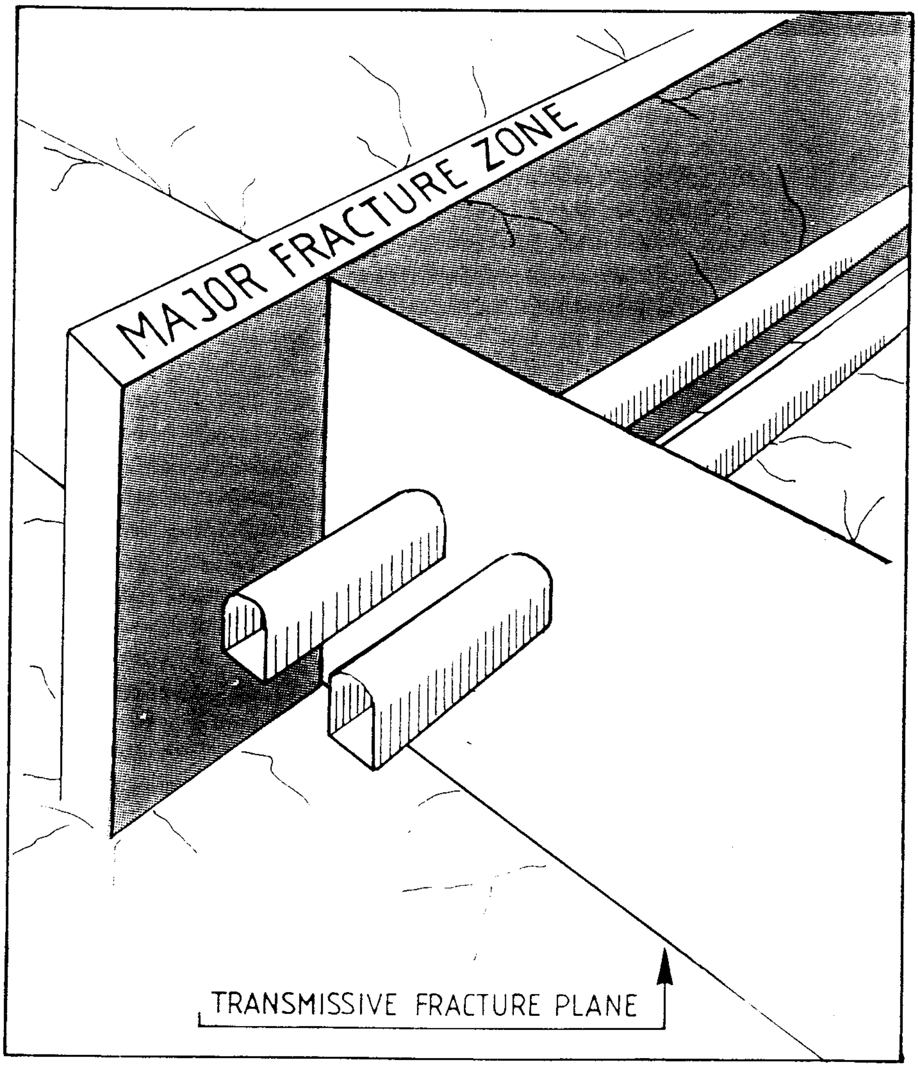


Figure 10-2. View of storage tunnels crossing a minor fracture zone which is connected to major bounding fracture zones.

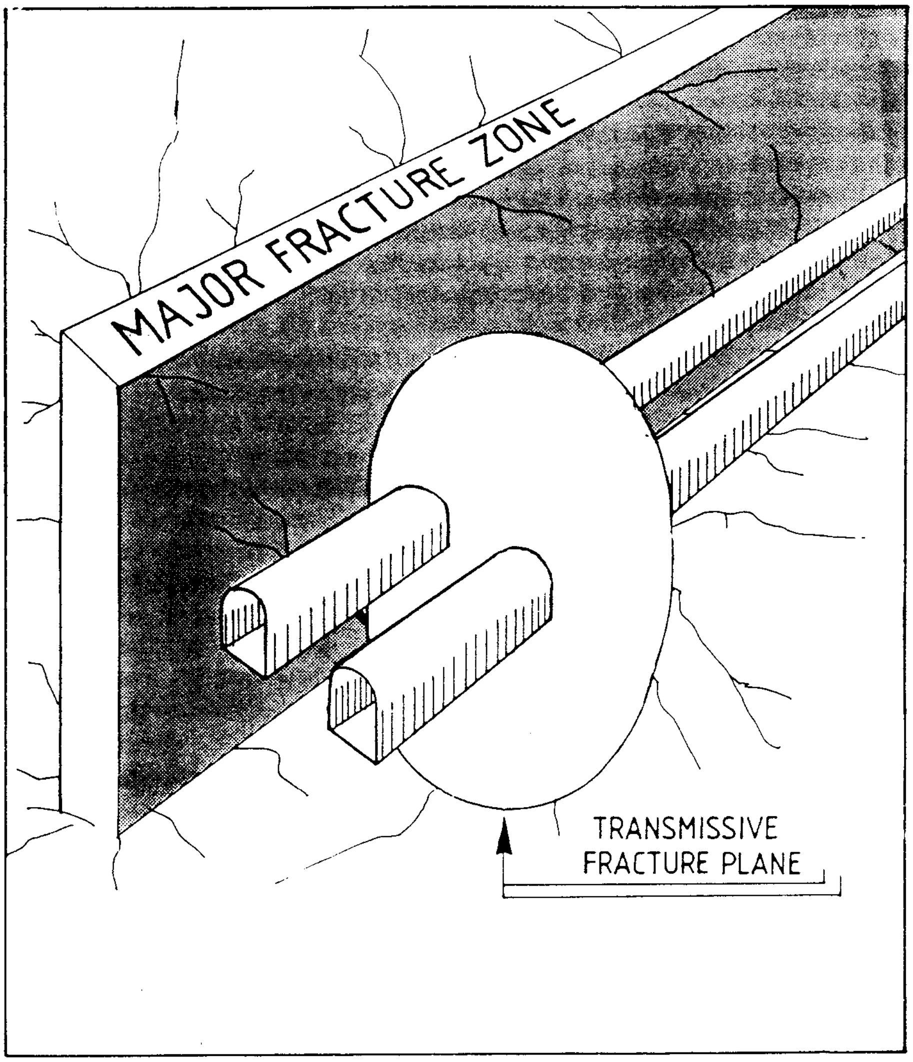


Figure 10-3. View of storage tunnels crossing a minor fracture zone which has no interconnections to the bounding major fracture zones.

is reduced by 40 %. A third parallel tunnel will further reduce the inflow to about half of the original. The inflowing water will mainly be taken from the bounding fracture zones in these cases.

The minor fracture zone intersected by the tunnels may be less permeable, or grouting is used where the second tunnel penetrates the zone. If such conditions are prevailing, a decrease of only 10% is expected in the inflow to the first tunnel due to the second tunnel.

The inflow values and their changes due to additional tunnels being constructed can be used to designate if the same features are penetrated by different tunnels. However to perform such interpretation, good quality is required on the inflow measurements. For instance if the same fracture as considered above has a limited extension and is not connected to the surrounding bounding major fracture zones, a smaller inflow will be measured, about 0.3 l/min when only one tunnel is constructed. A decrease in inflow will be monitored when an adjacent tunnel will be constructed penetrating the same feature as illustrated by Figure 10-3. However the inflows are of such a dignity that very accurate measurements are required to verify the impact from the second tunnel.

The groundwater head is the other main parameter to be monitored during a rock quality designation process. Chapter 7 describes some techniques available for these measurements. Impact on the head due to a tunnel construction is a transient process. Before reaching a steady state, the changes in head versus time after the disturbances are applied, can be used to quantify the hydraulic properties of the bedrock or of the adverse features within the bedrock. However, values of the groundwater inflow is also required.

By utilizing only the responses and their transient behaviour, qualitative informations can be achieved on the existence of transmissive fractures, their location in a borehole and hydraulic connections to other boreholes. Later in a designation process, hydraulic testing of the identified features can be carried out under controlled conditions which will make quantitative evaluation of properties much easier.

In order to illustrate the transient behaviour of the groundwater head, a tunnel crossing a horizontal fracture is modelled in a vertical plane perpendicular to both the fracture and the tunnel, c.f. Figure 10-4. The model-code used, TRAFRAP-WT /Huyakorn et al, 1987/, is especially designed to treat fractures as linear features in a two-dimensional domain. Hydraulic properties of the bedrock and the fracture are set according to Table 10-1. Few and contradictory data are available on the specific storage of a fracture. Data given by Black et al /1987/ and illustrated in Figure 10-5 are used for the specific storage coefficient of the fracture. The fracture width is set at 5.4 10^{-5} m.

Table 10-1. Data on hydraulic properties in modelling of the transient behaviour of groundwater head according to the model shown in Figure 10-4 and 10-8.

Hydraulic Unit	Hydraulic conductivity	Spec storage coefficient
Rock Mass	1.0 10^{-9} m/s	1.0 10^{-6} 1/m
Fracture	1.9 10^{-3} m/s	1.0 10^{-12} 1/m

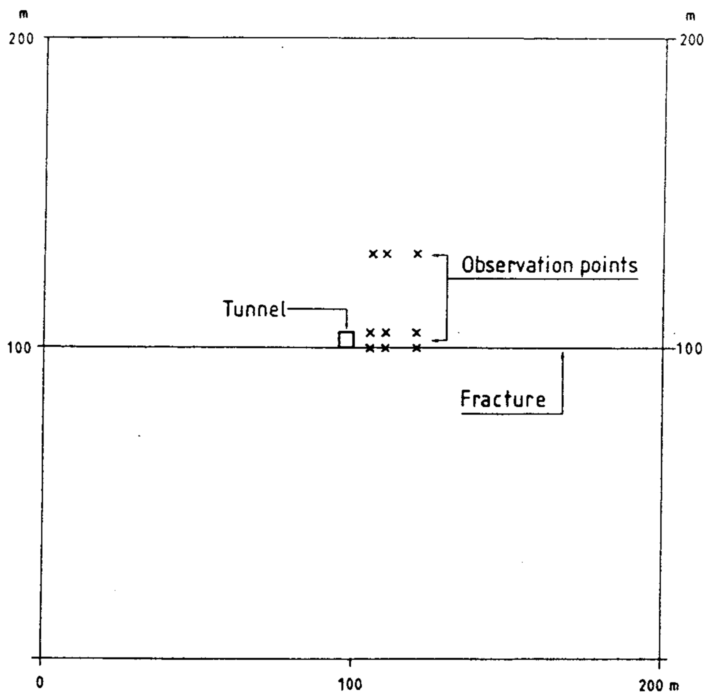


Figure 10-4. Geometry of a vertical plane including a horizontal fracture modelled to study the transient behaviour of groundwater head due to tunnel penetration where the fracture is intersected by the tunnel.

The model illustrates a plane 200 m times 200 m into which a tunnel is penetrating in the centre. The tunnel 5x5 m^2 in square area, has been given a hydraulic head of 500 m below the groundwater table, corresponding to its vertical position. The impact on the head in the fracture and the bedrock is presented in percentage of the applied disturbance, 500 m head drop. Non-flow boundaries are set around the plane.

The results given as head changes versus time after the tunnel penetrated the modelled plane, are presented in Figure 10-6. Points in the horizontal fracture at distances 5, 10, 20 meters from the tunnel have very rapidly responded in a head changes. After about 1 hour, a drawdown of more than 60 % of the tunnel drawdown is obtained at 5 m distance in the fracture. Points in the rock mass located 5 m from the fracture will react later in a head drawdown. After about 1 hour, a drawdown can be noticed and after 10 hours, a drawdown of 50 % of the tunnel drawdown can be observed 5 m from the tunnel and 5 m from the fracture plane.

Further out from the fracture into the rock mass, the response will be observed after longer times. For instance at 30 m distance from the fracture, the response will start after about 20 hours as illustrated by Figure 10-6.

The flow situation at various time intervals after tunnel penetration (1.2, 14, 220 hours) illustrated as lines of equal groundwater heads are presented in Figure 10-7. The influence of the fracture on the flow situation is clearly visualized in the figure.

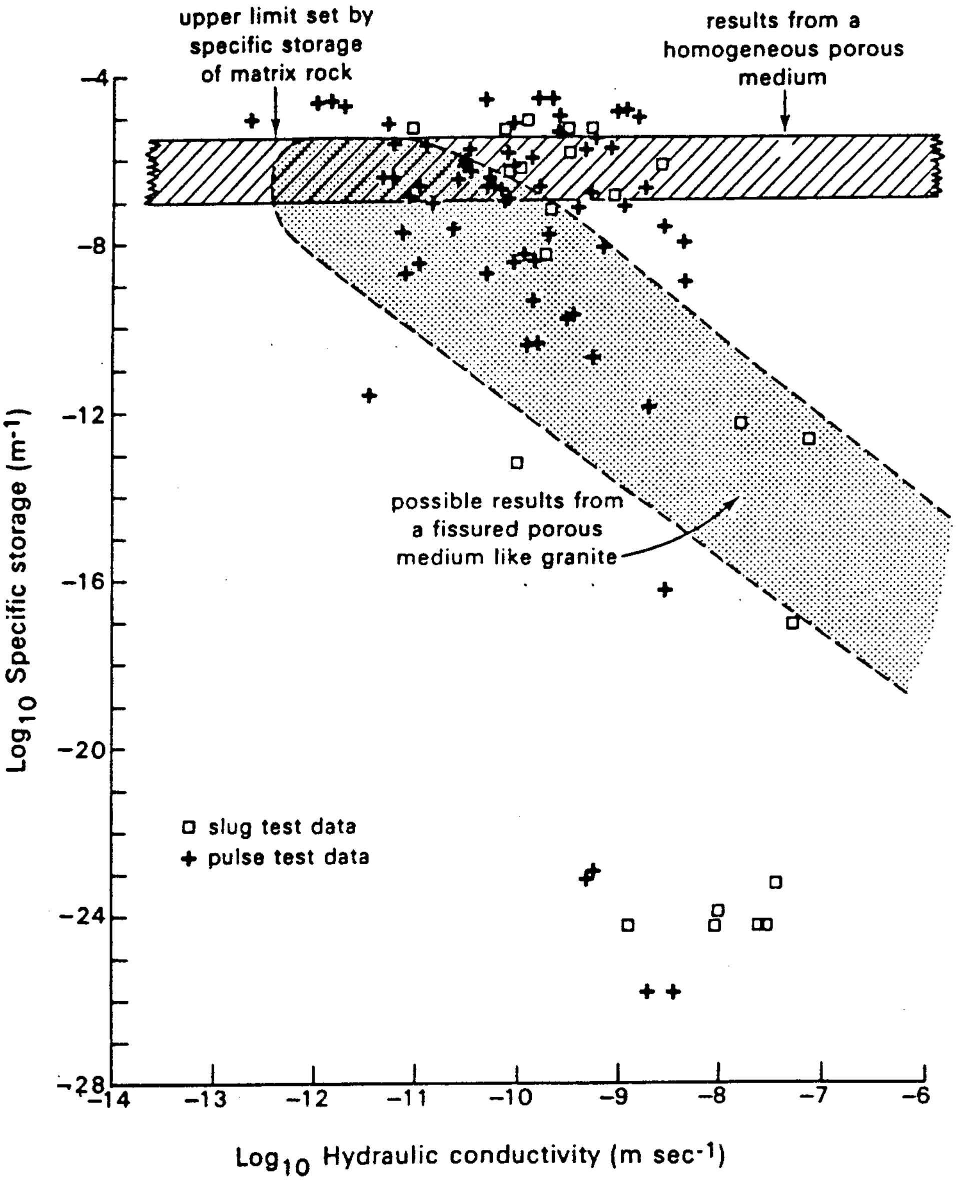


Figure 10-5. Hydraulic conductivity versus specific storage coefficient obtained from slug and pulse tests in boreholes at Stripa and presented by Black et al /1987/.

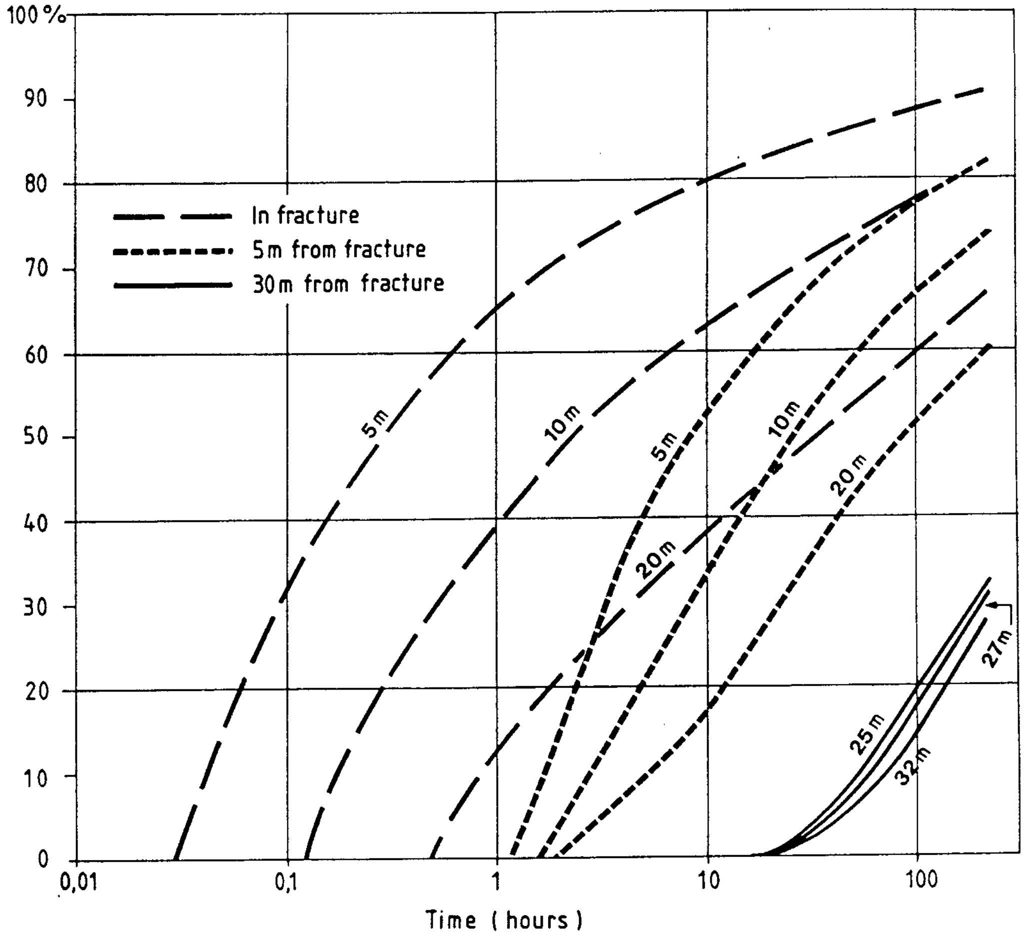


Figure 10-6. Changes in groundwater head as percentage of the initial head change at the tunnel, versus time after tunnel penetration. Observation points in the horizontal fracture and 5 and 30 m from the fracture respectively. The model lay-out is given in Figure 10-4.

The role assigned to the fracture can be illustrated by a modelling where the tunnel does not intersect the horizontal fracture as illustrated in Figure 10-8. In this scenario, the fracture is located 5 meter below the tunnel floor. Groundwater head at the same observation points as in the previous model are taken to illustrate the transient behaviour of the head change, c.f. Figure 10-9. Primarily it can be observed by comparing the two figures (10-6 and 10-9) that the interception of a continuous transmissive fracture will cause a much more rapid decline of the head in the fracture and adjacent rock mass than without any interception. In the first case, a decrease of more than 60 % is recognized 5 m from the tunnel in the

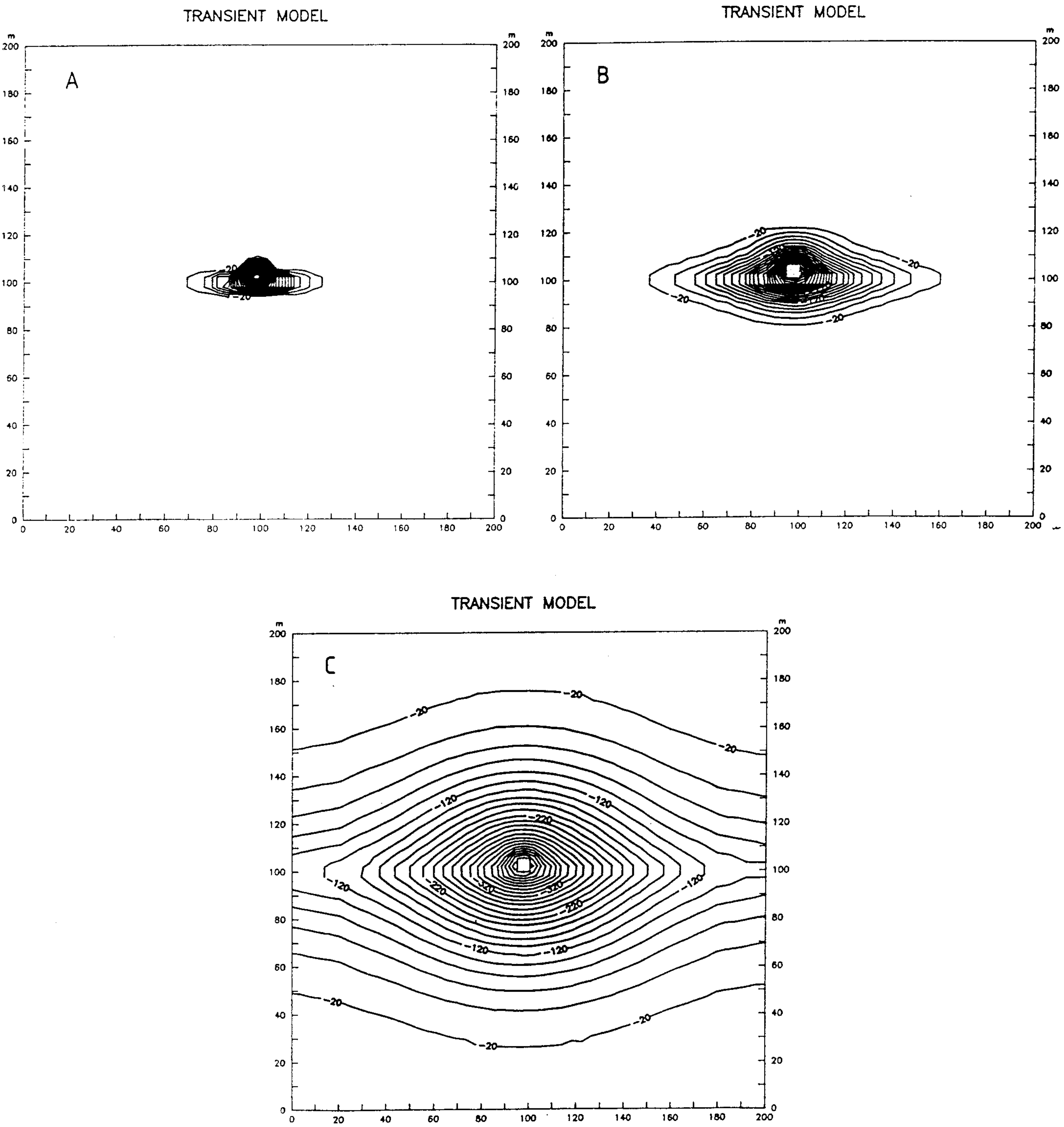


Figure 10-7. Groundwater flow situation illustrated by lines of equal groundwater head at various time intervals after tunnel penetration. A. Time = 1.2 hours. B. Time = 14 hours. C. Time = 220 hours. The model lay-out is given in Figure 10-4.

fracture after 1 hour. In the second case, a limited impact of only 2 % is found from the modelling results.

It could also be recognized from the results of the modelling of the second scenario, Figure 10-9, that a larger drawdown is established in the rock mass than in the fracture at the same distance from the tunnel, provided the distances are less than about 15 m. However, during the very early part an opposite situation is modelled.

The flow situations at various time intervals after tunnel penetration for the scenario illustrated in Figure 10-8 are presented in Figure 10-10. An impact on the equipotential lines due to the presence of the horizontal fracture can be recognized in the situations taken after more than about 10 hours.

The inflow to the tunnel will decrease versus time as illustrated by Figure 10-11. In the first scenario, Figure 10-4, a high inflow of over 10 l/s and km tunnel

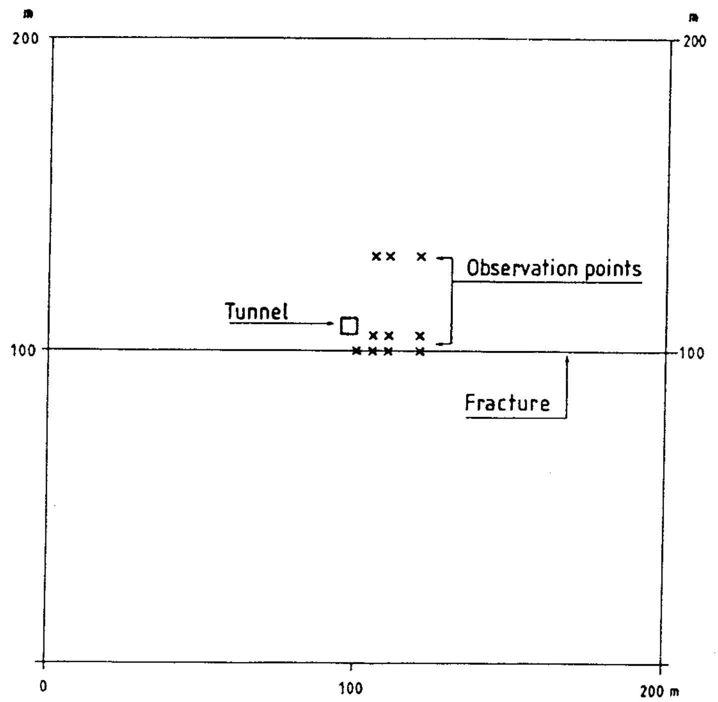


Figure 10-8. Geometry of a vertical plane including a horizontal fracture modelled to study the transient behaviour of groundwater head due to tunnel penetration where the fracture is located 5 meters below the tunnel floor.

is obtained during the first hour. After about 200 hours, the flow decreases to about 3 l/s km which might be an effect of the boundary conditions set. Of this inflow, about 90 % emanates from the horizontal fracture. In the second scenario, Figure 10-8, the flow decrease is less, from around 7.2 to 1.7 l/s.km. The modelled inflow values at time 223 hours can be used in Eq(10-1) in order to obtain an effective hydraulic conductivity of the bedrock. The values thus obtained correspond the a hydraulic conductivity of 5.10^{-9} m/s for the first scenario and 3.10^{-9} m/s for the second. These values should be compared with the ones given in Table 10-1.

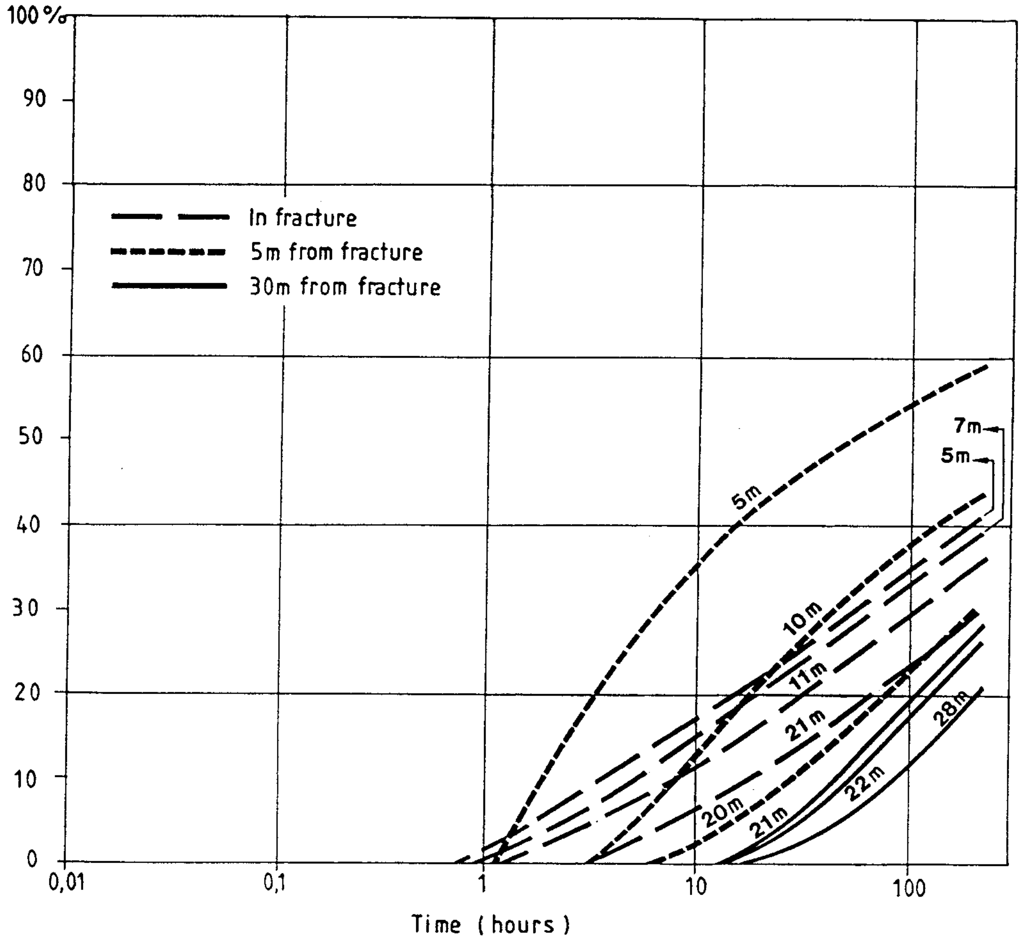


Figure 10-9. Changes in groundwater head as percentage of the initial head change at the tunnel, versus time after tunnel penetration. Observation points in the horizontal fracture and 5 and 30 m from the fracture respectively. The model lay-out is given in Figure 10-8.

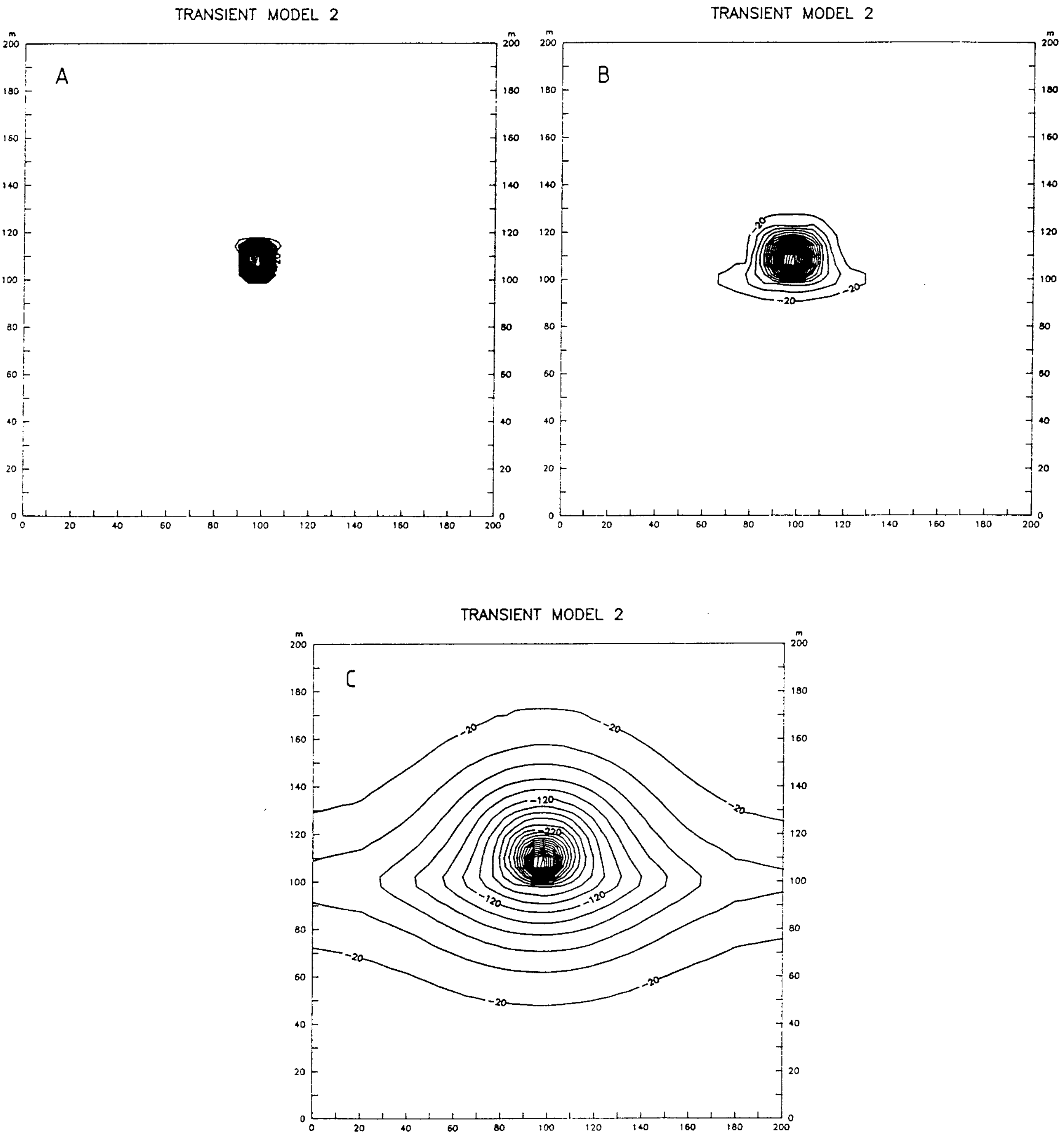


Figure 10-10. Groundwater flow situation illustrated by lines of equal groundwater head at various time intervals after tunnel penetration. A. Time = 1.2 hours. B. Time = 14 hours. C. Time = 220 hours. The model lay-out is given in Figure 10-8.

The results of the modelling show that head measurements can be utilized to indicate where a fracture, connected to a tunnel, can be found in the bedrock provided boreholes are available and sections sealed off in the holes. The very rapid response in the fracture compared to the rock mass is pronounced in the first hour after the tunnel penetration. This strongly stresses the importance of very early readings in order to evaluate the occurrence of fractures in a borehole which can have connections to tunnels.

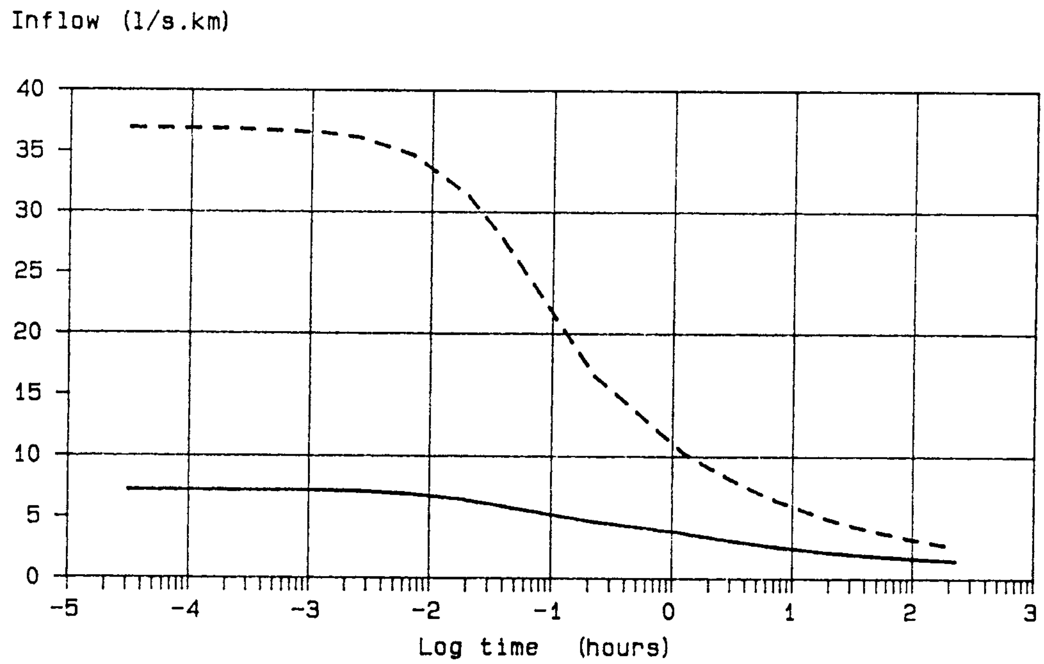


Figure 10-11. Groundwater inflow in l/s km versus time after tunnel penetration for the scenarios illustrated in Figure 10-4 (dashed line) and Figure 10-8 (full line).

10.3 MODELLING OF HYDRAULIC TESTING AT SFR

10.3.1 Background

A series of eleven interference tests were carried out during the construction of SFR. The tests and their results are summarized by Carlsson et al /1986/. Details and background data on the tests are given by Arnefors and Carlsson /1985/ and Andersson et al /1986/. In the current study, one of the tests is selected in order to illustrate the possibility of using test results and modelling to designate the rock quality. The test selected is referred to as test E by Carlsson et al /1986/ and it comprises the study of the extension and hydraulic properties of the fracture zone H2 at SFR.

10.3.2 Geometrical conditions of zone H2 at SFR

During the construction of the lower access tunnel to the silo at SFR, a fracture zone was penetrated. Due to high water inflow from the zone, measures as pre- and postgrouting had to be taken to accomplish further construction through the zone. The extent of the zone was interpreted from available core-logs, all together from 13 boreholes. Eight of these boreholes were drilled in a preinvestigation stage, later grouted and sealed and thus not available at the time of repository construction.

The results of the study of the core-logs identified the fracture zone, numbered as zone H2, as a sub-horizontal zone having a general dip of 18^{o} towards the SSE. The zone was found to be interconnected to a vertical zone, identified as zone 8, striking in N50W. A generalized picture of the upper surface of zone H2 interpreted from core-logs is given in Figure 10-12, and Figure 10-13 shows cross sections through zone H2 and zone 8.

10.3.3 Single-hole hydraulic testing in zone H2

Single-hole hydraulic tests were performed in the 8 boreholes (Kb1-Kb18) before the construction of the repository as well as in 5 additional holes (HK5-HK12) during the construction. The results from the tests are summarized in Table 10-2.

The width of the zone as given in Table 10-2 is taken from the core-logs. Plotted in a log-normal diagram, Figure 10-14, a mean value of $8x10^{-6}$ m^2/s is obtained for the transmissivity of the zone established from single-hole packer tests. As an average value of the thickness of the zone, the value 10 m is taken.

Table 10-2 Width and transmissivity of zone H2 obtained from core-logging and testing in boreholes penetrating the zone.

Borehole	Width (m)	Transmissivity (m^2/s)	Mean Hydraulic conductivity (m/s)
Kb1	5.7	$1.1x10^{-5}$	$2.0x10^{-6}$
Kb2	7.3	$5.1x10^{-6}$	$7.0x10^{-7}$
Kb4	5.2	$1.6x10^{-5}$	$3.0x10^{-6}$
Kb5	16.0	$8.3x10^{-6}$	$5.2x10^{-7}$
Kb11	2.4	$<1.2x10^{-7}$	$<5.0x10^{-8}$
Kb12	19.3	$3.2x10^{-5}$	$1.7x10^{-6}$
Kb17	8.7	$4.2x10^{-5}$	$4.9x10^{-6}$
Kb18	2.9	$2.5x10^{-5}$	$8.7x10^{-6}$
HK5	3.0	$4.4x10^{-7}$	$1.5x10^{-7}$
HK7A			$9.2x10^{-6}$
HK7B	6.5	$1.3x10^{-5}$	$2.0x10^{-6}$
HK7C	7.8	$1.9x10^{-7}$	$2.4x10^{-8}$
HK12	11.4	$2.3x10^{-6}$	$2.0x10^{-7}$

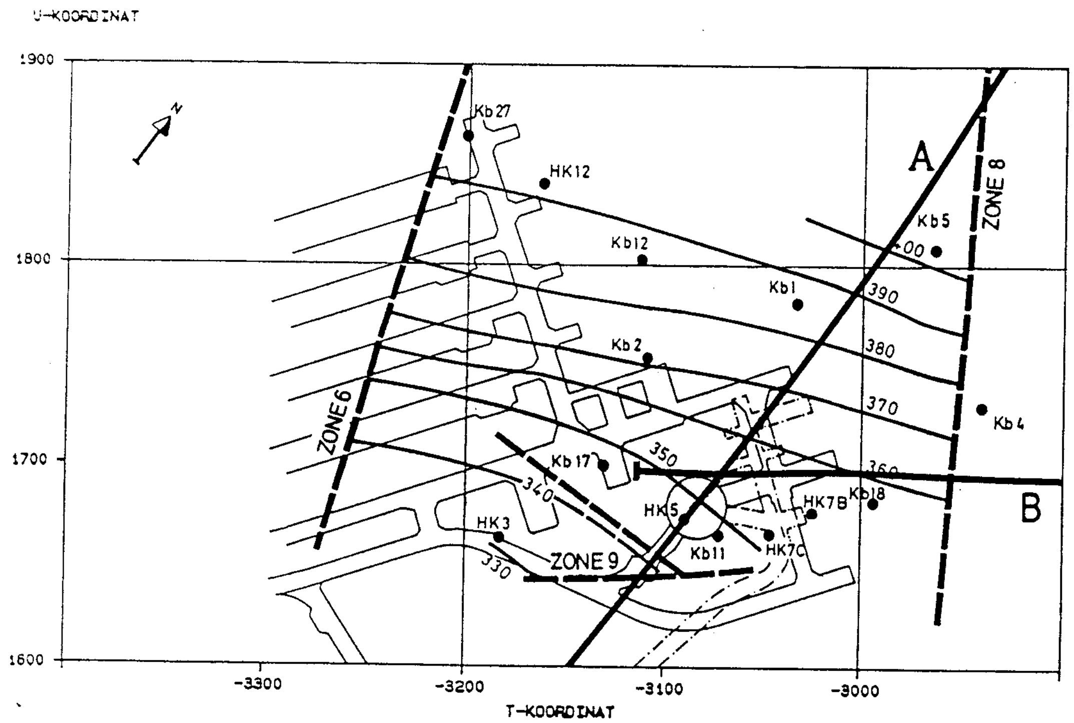


Figure 10-12 Equilines showing the upper surface of zone H2 within the area where the zone is encountered in the boreholes.

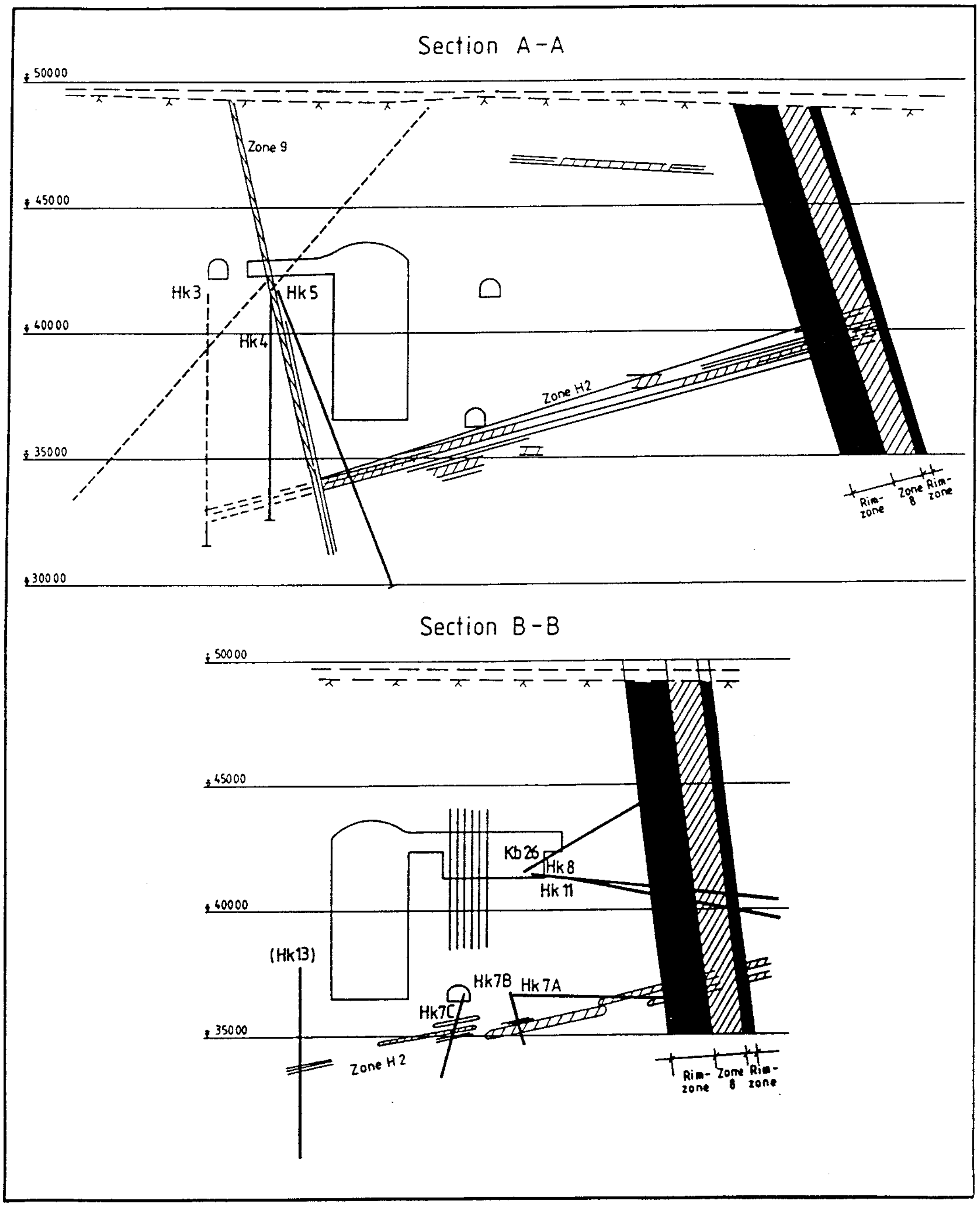


Figure 10-13 Cross section A-A and B-B laid out in Figure 10-12 illustrating fracture zones H2 and 8. After Christiansson /1986/.

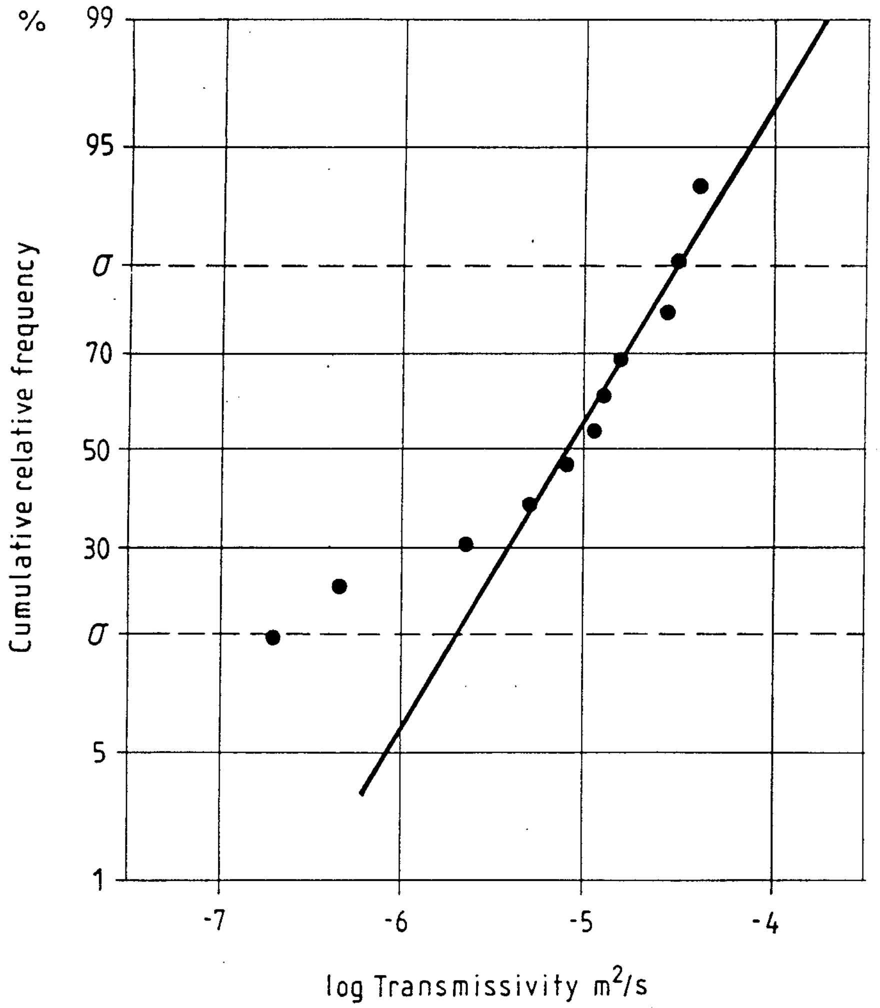


Figure 10-14 Relative cumulative percentage of log transmissivity values of zone H2 obtained from single-hole hydraulic testing.

10.3.4 Interference testing, Test E

Test E, conducted in the end of March 1986, aimed at describing the hydraulic conditions and properties along zone H2. In the test, borehole HK7B was the active borehole. Water was allowed to flow out from the zone at a discharging rate of 3.1×10^{-4} m^3/s (310 ml/s). In other boreholes within the repository, the head changes were monitored versus time. These observation boreholes were equipped with a multi-packer system, thus enabling the head change to be monitored both in the zone and in the surrounding rock mass. The test time comprised 111 hours.

The evaluation, reported by Andersson et al /1986/, focused on the head responses in those sections which were interpreted as penetrating zone H2. In accordance with theories reported by Streltsova /1983, 1984 and 1988/, the responses were divided into three groups; those sections with a rapid response (direct hydraulic contact along the fracture zone), those with delayed response (indirect contact) and those with no responses.

In Figure 10-15, the active borehole and the sections assumed to penetrate zone H2 are given in T- and U-coordinates. These coordinates are given in a local coordinate system especially set up for the repository, c.f. Carlsson et al /1986/.

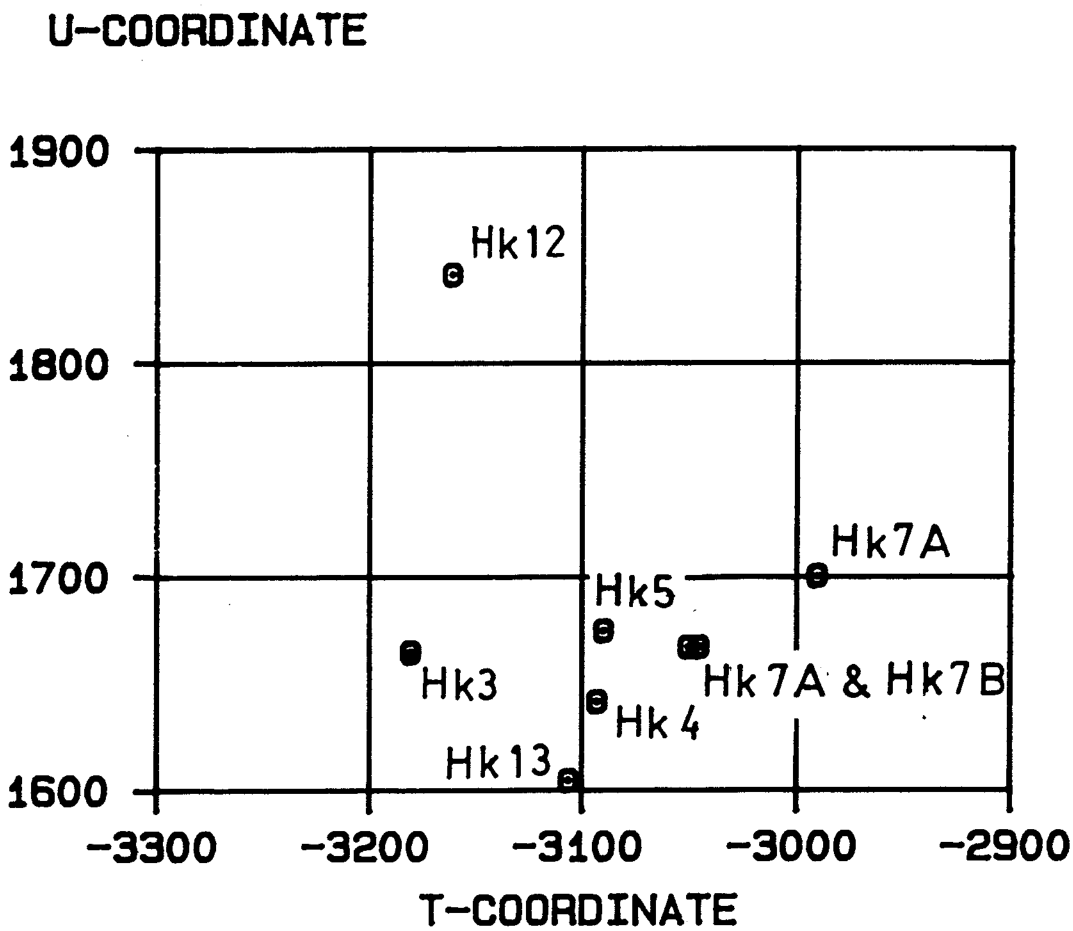


Figure 10-15 Location of boreholes penetrating zone H2. The coordinate system is the local system at SFR.

The two fracture zones H2 and 8 were interacting during the interference test. In the evaluation process and in the current modelling performance, zone 8 is taken as a continuation of zone H2 and the distance from the active borehole to borehole sections penetrating zone 8 is taken along the two zones. The distances between the active borehole and the sections in the zone, measured along the zone (or zones) are given in Table 10-3.

The vertical distance from zone H2 (and horizontal from zone 8) for the sections assumed to represent the rock mass are given in Table 10-4.

10.3.5 Transient modelling of Test E

The transient interference test, with borehole HK7B as the active borehole, has been simulated by the use of the numerical code TRAFRAP. This code is a finite element code described by Huyakorn et al (1987). The element-net laid out over the modelled area is shown in Figure 10-16. The model is axisymmetrical around the y-axis and the fracture zone H2 with continuation in zone 8 is illustrated as a 10 m thick layer (two elements thick), 150 m below the upper boundary of the model. This boundary is set as a Dirichlet boundary. Borehole HK7B discharging water at a constant rate of 3.1×10^{-4} m^3/s is simulated as three discharging node points in the horizontal fracture zone along the y-axis.

Table 10-3 Distance between the active borehole HK7B and sections in observation boreholes which penetrate zone H2 (or zone 8). The distances are calculated along zone H2 and zone 8. Hydraulic conductivity values evaluated from hydraulic test E.

Borehole Section	Distance (m)	Evaluated Hydraulic conductivity (m/s)
HK7C	22	5.4×10^{-6}
HK7A:1	40	
HK5:2	67	7.6×10^{-7}
HK4:1	80	
HKL11:1	100	5.0×10^{-7}
HK13:1	115	
HK8:1	125	6.8×10^{-7}
HK3:1	157	
HK12:2	215	1.5×10^{-6}

Table 10-4 Vertical distance between zone H2 (horizontal distance between zone 8) and sections representing the rock mass.

Borehole Section	Vertical distance (m)	Borehole Section	Vertical distance (m)
HK3:2	7-31	HK8:2	3-22
HK3:3	32-43	HK8:3	30-59
HK3:4	44-83		
		HK11:2	5-15
HK4:2	7-45	HK11:3	16-35
HK4:3	46-61		
HK4:4	61-83	HK12:1	2-19 *
		HK12:3	13-29
HK5:1	8-40 *		
HK5:3	5-26	HK13:2	2-21
HK5:4	27-68	HK13:3	22-51
HK7A:2	2	Kb25:2	40-41
HK7A:3	5	Kb25:3	42-45
		Kb25:4	45-48

* = located below zone H2

The various sections of the different boreholes are in the model represented by node-points. Since some of the sections are longer than the side of the elements, they could be represented by several node-points. However, in the process of comparing model results with the results from the hydraulic testing, the node-points illustrated in the element net in Figure 10-16 have been used.

The modelling procedure first covers a simulation focused on the transient head-change behaviour within the fracture zone. The obtained results are compared with the observed data. The comparison primarily aims at describing if the observed data can be referred to transient behaviour valid for the modelled fracture zone or if they are more representative for a transient behaviour of the surrounding rock mass. Thus, the comparison will give the first analysis of whether the geological interpretation of a continuous zone as illustrated in Figure 10-13 and 10-16 is a valid concept for further interpretation and modelling.

In the first phase, six different runs have been conducted where the hydraulic properties of the fracture zone have been varied according to the figures given in Table 10-5.

In total 30 observation sections represented by node-points are simulated within each run, representing node-points. Of these, 9 points are sited within the

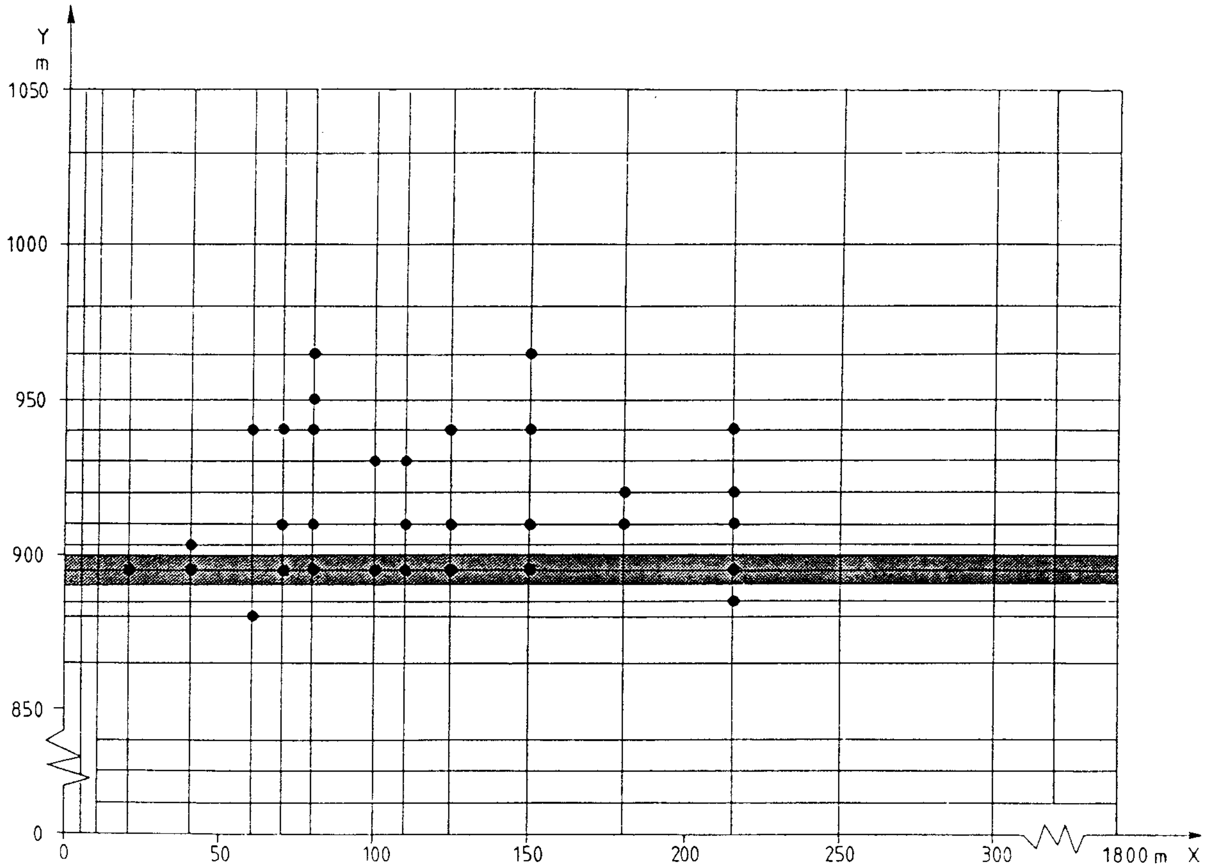


Figure 10-16 Element-net used in numerical modelling of the transient hydraulic test E. Element nodes representing various borehole sections during test E are illustrated by o.

fracture zone, c.f. Table 10-3 (including the two points in fracture zone 8). The calculated head-change versus time for the 6 runs given in Table 10-5 are illustrated in Figure 10-17 for node-points representing observation sections within zone H2. The observed values from the hydraulic test E are also illustrated in the figure.

The six runs comprising the first phase of the modelling exercise have all various hydraulic properties of the fracture zone, c.f. Table 10-5, whereas the rock mass properties are kept constant. However, the latter properties also affect the drawdown versus time in the fracture zone as a leakage of water to the zone from the surrounding rock mass. However, the initial stage of the drawdown curve in the fracture zone is mainly dependent on the hydraulic properties of the zone and the results of the first modelling phase should there-

Table 10-5 Hydraulic properties used in the first phase of the model simulation of test E at SFR.

	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6
Fracture zone						
K-value m/s	$2x10^{-6}$	$2x10^{-6}$	$2x10^{-6}$	$7x10^{-6}$	$7x10^{-6}$	$7x10^{-6}$
Ss-value 1/m	10^{-7}	10^{-6}	10^{-5}	10^{-7}	10^{-6}	10^{-5}
Rock mass						
K-value m/s	10^{-9}	10^{-9}	10^{-9}	10^{-9}	10^{-9}	10^{-9}
Ss-value 1/m	10^{-6}	10^{-6}	10^{-6}	10^{-6}	10^{-6}	10^{-6}

fore be compared to first parts of the observed drawdown curves. The observation point closest to the active well, section HK7C:1 (node-point 72), respond in a way that indicate a hydraulic conductivity of the same order as in Run 3. The respond starts later than that simulated in Run 3 which indicate an even higher specific storage coefficient than in this run.

Section HK7A:1 at a distance of 40 m from the active well indicates properties of fracture zone H2 close to the ones used in Run 6. However, a slightly lower specific storage coefficient is noted. As in the case for all observation sections in the fracture zone, the latter parts of the observed drawdown curves deviate from the simulated values as illustrated in Figure 10-17.

Section HK5:2 clearly represents the drawdown in the fracture zone. Run 3 is a good simulation of the observed drawdown values. The specific storage coefficient should however be somewhat smaller than in Run 3.

The observed drawdown in section HK4:1 (node-point 156) can be simulated with hydraulic conductivity values somewhere between the values used in Run 3 and 6. However, the specific storage coefficient has to be higher than the ones used in these runs.

Section HK11:1 and HK8:1 both represent fracture zone 8 (simulated in the modelling as a prolongation of zone H2) and are quite good simulated by Run 5. The first mentioned section might have a slightly lower hydraulic conductivity than used in Run 5.

The drawdown observed in section HK13:1 could be simulated as a respond valid for the fracture zone H2. Run 6, probably with a slightly lower hydraulic conduc

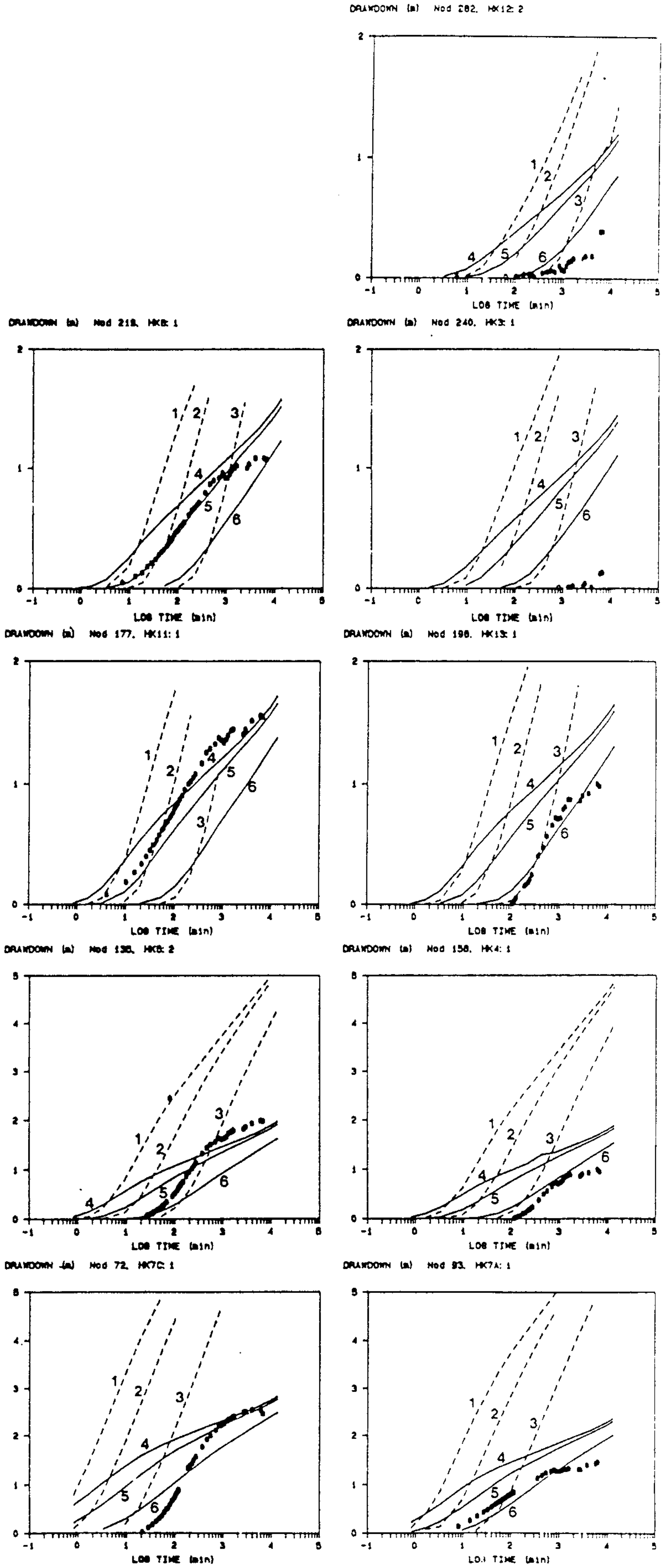


Figure 10-17 **Head-changes for observation sections in fracture zone H2 (and zone 8), observed and calculated. Hydraulic properties used in the different runs are summarized in Table 10-5.**

tivity, simulate the observed drawdown with a good agreement.

In order to simulate the observed drawdown in section HK3:1, hydraulic properties are required far different from the ones obtained for other sections in the zone and simulated in Run 1 through 6. Thus, the conclusion is that section HK3:1 is not representative for fracture zone H2. On the other hand, section HK12:3 probably represents zone H2 but with a slightly higher specific storage coefficient than in Run 6.

As a summary of the first phase of the model simulation of hydraulic interference test E, it can be concluded that the identified zone represented by various borehole sections can be verified by the simulation. The exception is the section HK3:1 which probably does not represent zone H2. The other sections in the fracture zone all show the behaviour of the drawdown versus time curves for the initial part of the curves as can be simulated by the model runs 1 through 6. The behaviour of the later part of the curves depend on the hydraulic properties of the surrounding rock mass.

In the second phase of the transient modelling simulation, the hydraulic properties of the bedrock (rock mass) have been varied whereas the fracture zone properties are kept constant. Table 10-6 summarizes the hydraulic properties used for the rock mass in the various runs in phase 2. The hydraulic conductivity and specific storage coefficient of the fracture zone are set at 7 10^{-6} m/s and 10^{-6} 1/m respectively.

The results given in Figure 10-18 through 10-20 illustrate the calculated head change versus time for the various values of the specific storage coefficient ascribed to the rock mass. Although various values on

Table 10-6. Hydraulic properties used in the second phase of the modelling simulation of test E at SFR.

Spec. stor. coefficient (1/m)	Rock mass hydraulic conductivity (m/s)		
	10^{-11}	10^{-10}	10^{-9}
(1) 10^{-8}	Run 206	Run 203	Run 212
(2) 10^{-7}	Run 205	Run 202	Run 211
(3) 10^{-6}	Run 208	Run 207	Run 210
(4) 10^{-5}	Run 204	Run 201	Run 209

the rock mass hydraulic conductivity are given in the runs, the slope of the curves in the figures are quite constant and mainly dependent on the value set on the properties of the fracture zone. The smaller the specific storage coefficient of the rock mass, the earlier the head changes are recognized. However, the differences in time is within one order of magnitude for orders of magnitude change of the specific storage coefficient when comparing the head changes in the fracture zone. In the surrounding rock mass, illustrated in Figure 10-20, larger time differences are encountered, up to two orders of magnitude.

The simulations have not been quite successful to illustrate the total behaviour of the observed head changes during test E at SFR. Effects caused by the boundaries as the SFR-excavations, additional but not modelled fracture zones and spatial variations in hydraulic properties might be some of the reasons for the unconformity obtained. However, the exercises show that major fracture zone continuity can be established by interference testing and briefly verified by model simulations.

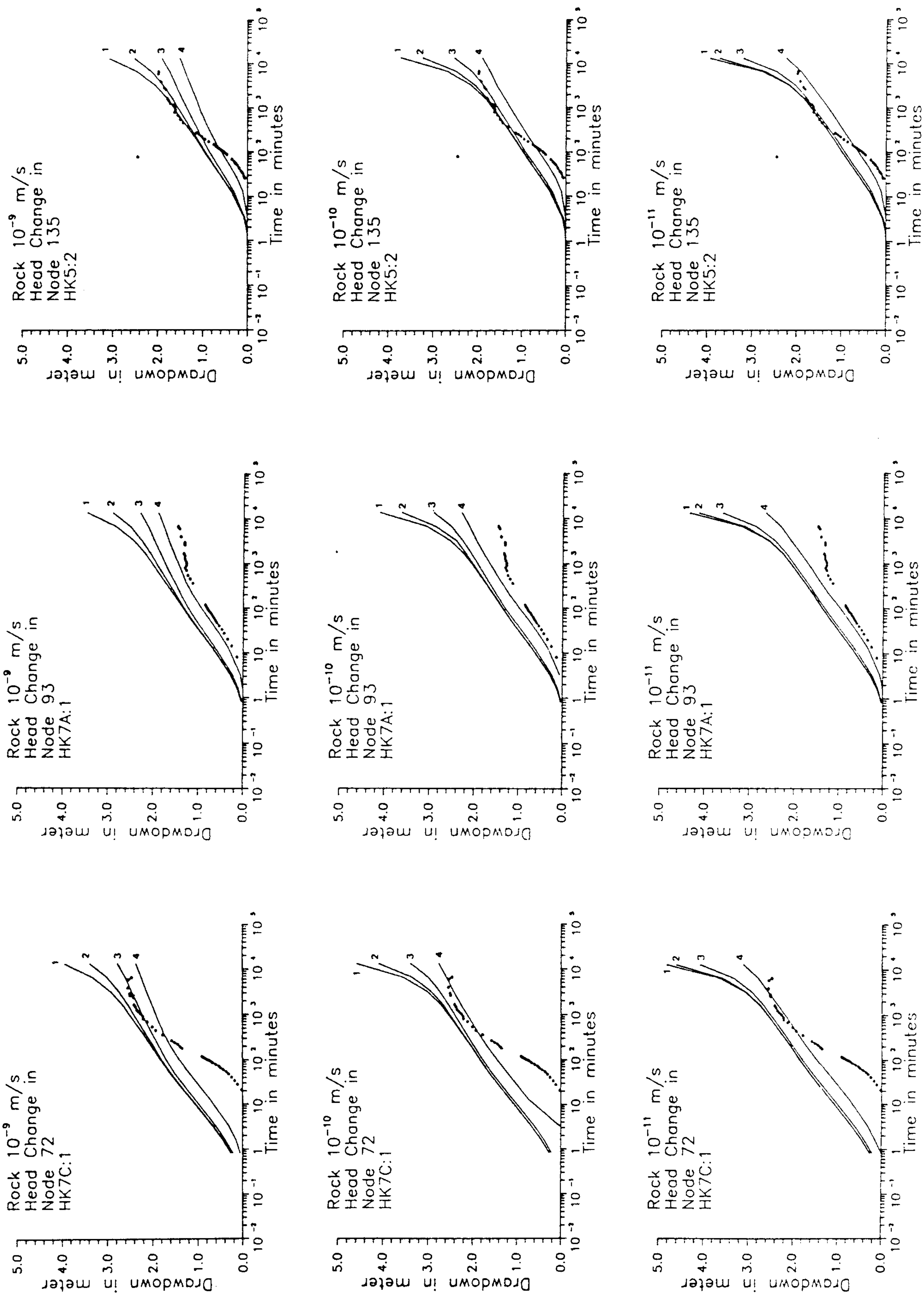


Figure 10-18 **Head-changes observed and calculated with various hydraulic properties ascribed to the rock mass. The hydraulic conductivity of the fracture zone is set at 7 10^{-6} and the specific storage coefficient at 10^{-6} 1/m. Values of the rock mass are given in Table 10-4. Observation points in the fracture zone.**

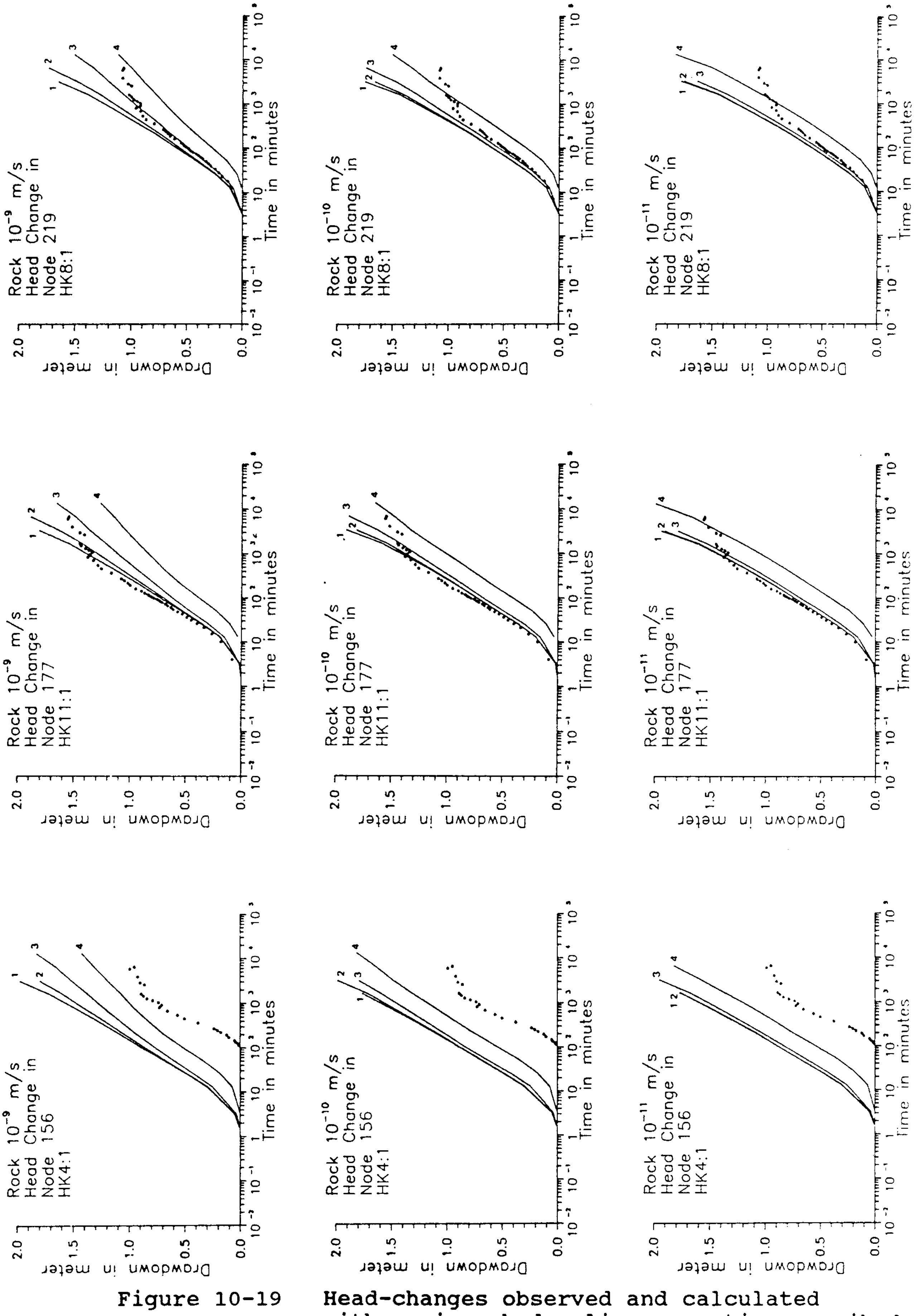


Figure 10-19 Head-changes observed and calculated with various hydraulic properties ascribed to the rock mass. The hydraulic conductivity of the fracture zone is set at 7 10^{-6} and the specific storage coefficient at 10^{-6} 1/m. Values of the rock mass are given in Table 10-4. Observation points in the fracture zone.

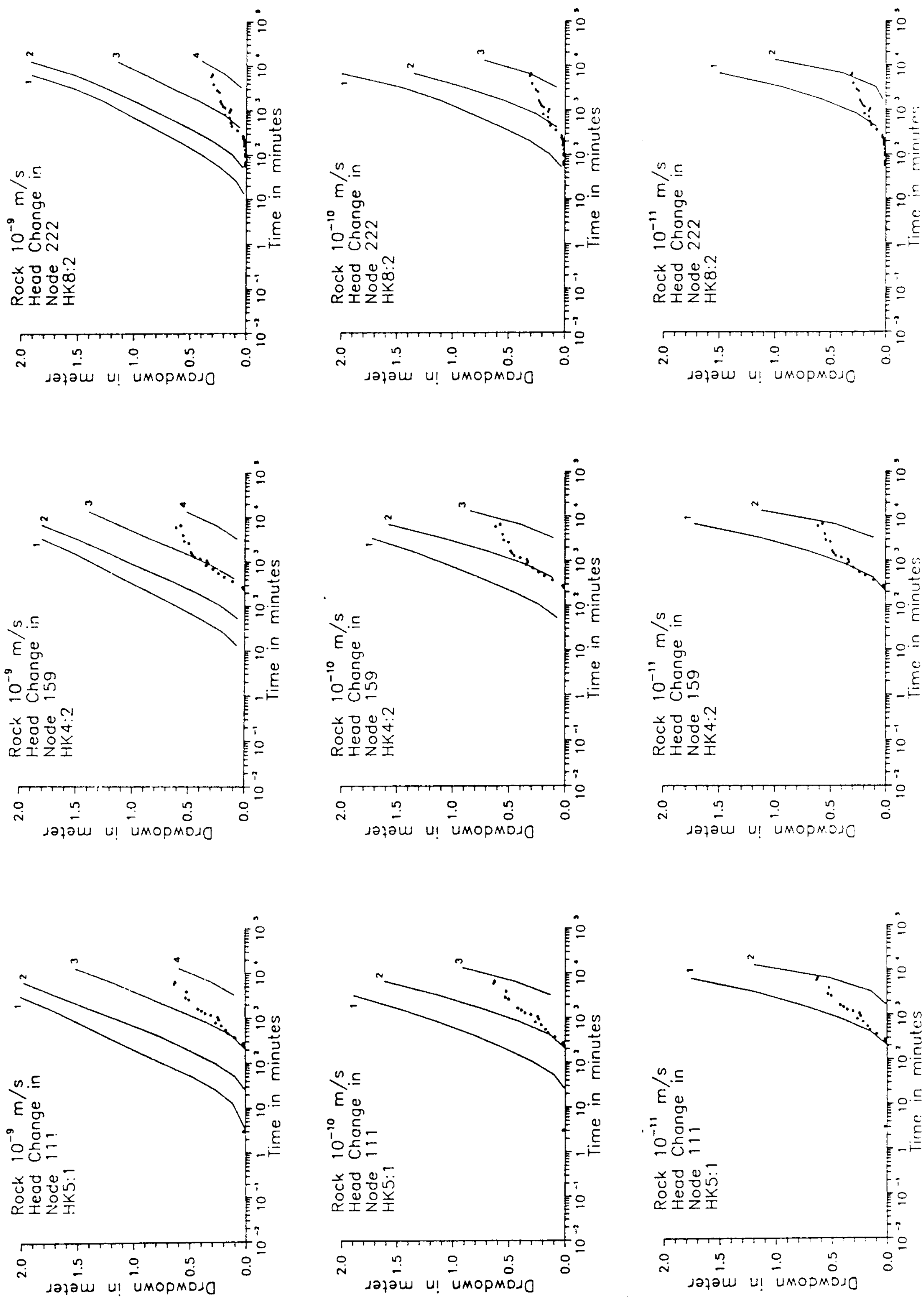


Figure 10-20 Head-changes observed and calculated with various hydraulic properties ascribed to the rock mass. The hydraulic conductivity of the fracture zone is set at 7 10^{-6} and the specific storage coefficient at 10^{-6} 1/m. Values of the rock mass are given in Table 10-4. Observation points in the rock mass.

11 CONCLUSIONS

The immediate surroundings of a nuclear waste repository can be described in various terms relating to the bedrock strength, mineralogy, fracture intensity, water conductivity etc. A quality designation of the near field of a repository is primariliy a delineation and description of the occurrence of adverse features for groundwater transport and rock stability such as faults, fracture zones, crushed rock etc. Techniques are today available for such a rock quality designation, or rock quality assurance program, to be carried out concurrently with the construction and licencing of a nuclear waste repository.

The two main repository concepts considered in the current report, the KBS-3 and WP-Cave concepts respectively, both allow rock quality assurance programs to be carried out simultaneously with the repository construction. Pilot boreholes combined with testing in those are the base techniques to be applied during a repository construction. Mapping of geology, structure and groundwater inflow in excavated constructions are other techniques to be used.

Application of any additional techniques not included in an "ordinary" rock blasting and construction scheme will most certainly have an impact of the time-schedule for a repository construction. However, it is of the uttermost importance that data are obtained early during construction and that monitoring stations are established for groundwater inflow and head measurements in the very early stage of a repository construction. Changes in groundwater head and flow monitored during the construction reflect the hydraulic system in the repository near field. In turn this system is a function of the geometry and properties of the occuring transmissive features in the bedrock. The early responses from impacts due to construction thus is a main technique in the rock quality designation process, whereas the responses in long term perspective are to a high degree dependent on the outer system, the groundwater boundary conditions etc.

Quality designation of the rock is a more straightforward issue for the "very near-field" comprising an induvidual deposition hole than for the larger parts of the rock that includes both shaft, deposition tunnels and holes. The larger portions of rock that comprise shafts and tunnels is also more difficult to quality designate due to the fact that disturbance by blasting and stress release influences the conductivity to an extent that is hard to visualize and evaluate at present. The current idea is that the axial conductivity of the rock adjacent to shafts and tunnels is much higher than that of the virgin rock, and the expected difficulty in quantifying this before and during the excavation of a repository suggests that "superconductors" operating over the entire repository may be regular features that remain undetected. This will not be critical if effective seals are arranged in the form of bentonite-filled slots combined with grouting of closely spaced boreholes since they will be effectively cut off. The proper placing of such local seals can partly be based on the knowledge of where major conductive zones are located, but it seems that a strategy basically involving systematic application of seals at the end of repository tunnels and at those levels in shafts where low-conductive rock is penetrated would yield the net effect that is asked for, i.e. large volumes of stagnant groundwater adjacent to the excavations.

Techniques as borehole and tunnel radar are very useful in the rock quality designation process. Adverse features can be identified in the bedrock from boreholes and tunnels without having to destroy the bedrock itself. Although tunnel radar technique is not yet fully developed, it is assumed to be a promising instrument to use in the rock quality designation process.

Currently criteria are not fully developed regarding the quality of the bedrock for various purposes in a repository. Such criteria would probably have to be very extensive and flexible and include variables which probably are very difficult to measure during a designation process. However, when coming to the selection process for deposition holes, some ideas on recommendations or criteria should be considered regarding for instance water in- or out-flow, distance to nearest larger fracture, number of fractures allowed in the borehole, length and interconnections of fractures etc.

The techniques available to day make it possible to delineate major adverse and water conductive features. In the next step techniques need to be improved in quantifying of the properties and geometries of these features. Hydraulic interference testing technique makes it possible to quantify the major features themselves. Properties of the surrounding rock mass are harder to evaluate from these types of testing. Figure 11-1 illustrates results of modelling of drawdown 20 m from an applied disturbance (constant flow) in the same geometry as given in Figure 10-16 for SFR. At various vertical distances from the fracture zone, the responses varies as a function of i.e. the contrast between the hydraulic conductivity of the fracture zone and the rock mass. In the calculations the same value of the specific storage coefficient is given in both the fracture zone and the rock mass (10^{-6} 1/m). The fracture zone has a hydraulic conductivity of 10^{-6} m/s (10 m thick).

The results in Figure 11-1 show that when the contrast in hydraulic conductivity between the fracture zone and the rock mass is about 100 or higher, the response curves from observations in the rock mass and in the fracture zone are highly separated. Thus, the response curves can for such situation be used to distinguish if the observed responses are to be regarded as coming from the fracture zone or the rock mass.

During each stage of the construction of a repository, a strategy is outlined regarding techniques possible for rock quality designation of the hydrogeological conditions and properties important for the long term safety of the repository. Geological, geophysical, hydrogeological and hydrogeochemical investigations are performed for collection of data for conceptual and predictive models. The predicted data are compared with field investigations in an iterative manner during the repository construction. In this way, the most favourable volumes of rock can be designated for the final locations of the deposition holes in the case of a rpository according to the KBS-3 concept. Adverse hydraulic features, intersecting the excavations or located in their vicinity, will be sealed by grout or bentonite injection, thus enhancing the quality of the rock by redirecting the groundwater flow away from the waste.

In a repository according to the WP-Cave concept, the term near field is considered primarily to be the volume encompassed by the hydraulic cage. The rock quality designation for a WP-Cave repository thus primarily aims at describing the rock volume inside the cage. In contrast to the KBS-3 concept, the flexibility of a WP-Cave is limited once the hydraulic cage is laid out and the annular tunnels constructed. To compensate for this, a thoroughly pre-investigation including a detailed conceptual model of the assigned rock block need to be conducted. Also a heavier use and confidence of engineering barriers is required compared to the KBS-3 concept.

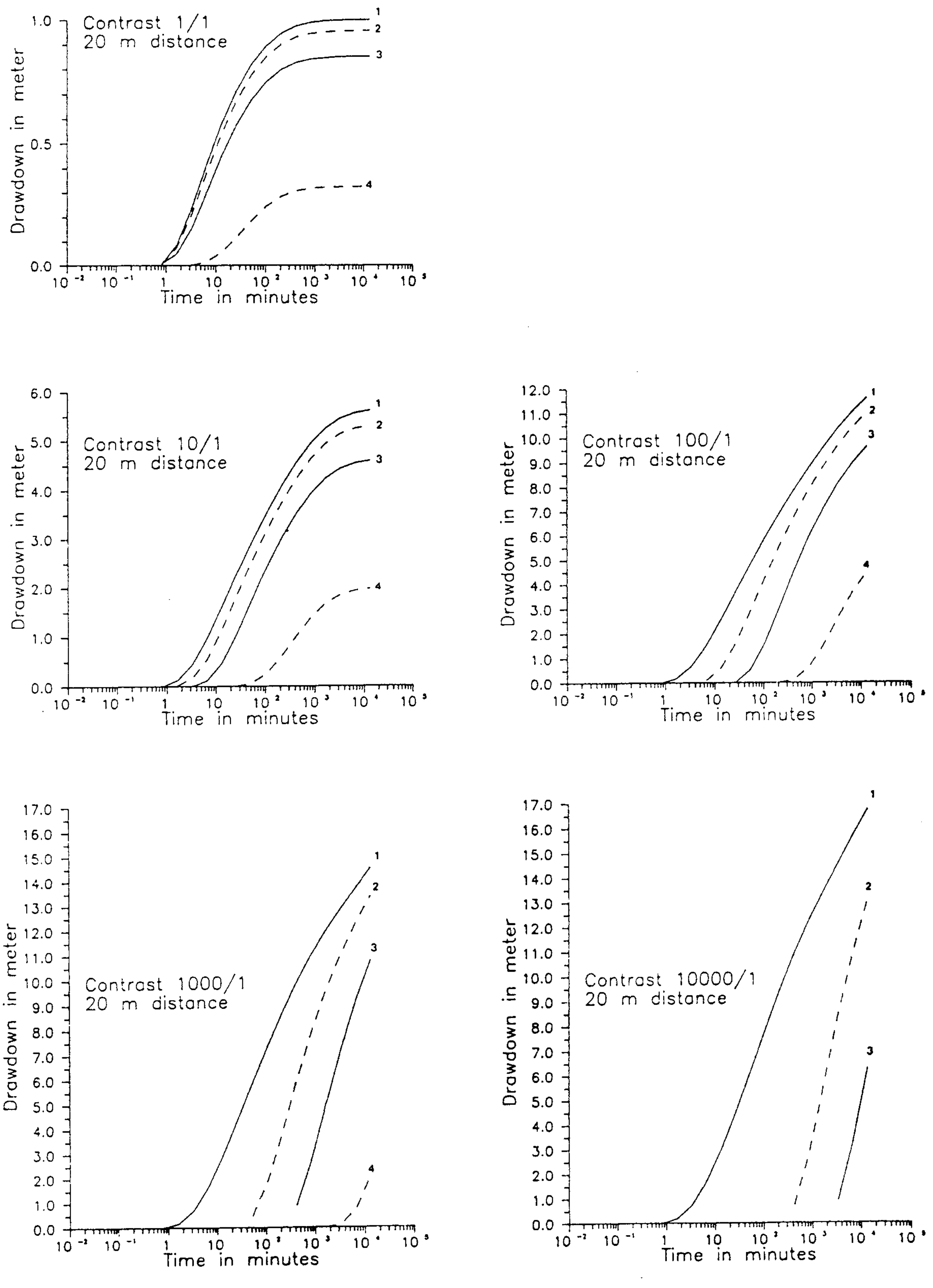


Figure 11-1 Drawdown in a fracture zone having the same geometry as illustrated in Figure 10-16. 1 = drawdown in fracture zone, 20 m from the disturbance. 2 = drawdown in the rock mass 3 m above the fracture zone. 3 = drawdown in the rock mass 10 m above the fracture zone. 4 = drawdown in the rock mass 50 m above the fracture zone

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13 REFERENCES

Abelin, H., Birgersson, L., Gidlund, J., Moreno, L., Neretnieks, I., Widén, H., Ågren, T. 1987. 3-D Migration Experiment - Report 3, Part 1. Performed Experiments, Results and Evaluation. Stripa Project, Technical Report TR 87-21. Stockholm 1987.

AECL, Atomic Energy of Canada Limited Research Company, 1988. Quality Assurance Manual. Geological and Environmental Science Division. Whiteshell Nuclear Research Development. Pinawa, Manitoba, Canada.

Ahlbom, K., Carlsson, L., Carlsten, L-E, Duran, O., Larsson, N-Å, Olsson, O., 1983. Evaluation of the Geological, Geophysical and Hydrogeological Conditions at Fjällveden. SKB Technical Report 83-52, Stockholm, Sweden.

Andersson, J-E., Arnefors, J., Carlsson, L., Danielsson, J., Hansson, K. 1986. Pressure Build-up and Interference testing at SFR, Part 2. SGAB IRAP 86403 (In Swedish).

Arnefors. J., Carlsson, L. 1985. Pressure Build-up and Interference testing at SFR, Part 1. SGAB IRAP 85409 (In Swedish).

Baecher G.B., Lanney N.A. 1978. Trace Length Biases in Joint Surveys. Proceedings of the 19th US Symposium on rock mechanics, pp 56-65. Ameriocan Institute of Mining Engineers, New York.

Black, J., Holmes, D., Brightman, M. 1987. Crosshole Investigations - Hydrogeological Results and Interpretations. Stripa Project Technical Report TR 87-18.

Carlsson, A. 1979. Characteristic Features of a Superficial Rock Mass in Southern Sweden (doct. thesis). Striae Vol 11, Uppsala 1979.

Carlsson, L., Olsson, T. 1985. Hydrogeological nad Hydrogeochemical Investiagtions in Boreholes. Final Report. Stripa Project., Technical Report 85-10. Stockholm 1985.

Carlsson, L., Winberg, A., Grundfeldt, B. 1983. Model Calculations of Groundwater Flow at Finnsjön, Fjällveden, Gideå and Kamlunge. SKB Technical Report KBS 83-45. Stockholm 1983.

Carlsson, L., Winberg, A., Rosander, B., 1984. Investigations of Hydraulic Properties in Crystalline Rock. In Scientific Basis for Nuclear Waste Management VII, Materials Research Society Proceedings. Vol 26, pp 255-267, Boston, MA.

Carlsson, L., Carlsten, S., Sigurdsson T., Winberg A. Hydraulic Modelling of the Final Repository for Reactor Waste (SFR). 1985. Swedish Nuclear Fuel and Waste Management Co. SFR 85-06. Stockholm, Sweden.

Carlsson L., Winberg A., Arnefors J. 1986. Hydraulic Modelling of the Final Repository for Reactor Waste (SFR). Swedish Nuclear Fuel and Waste Management Co. Progress report SFR 86-03.

Christiansson, R., 1986. Geological description of zones around the final repository. SKB Progress Report SFR 86-02 (In Swedish).

de Marsily, G., 1985. Flow and Transport in Fractured Rocks: Connectivity and Scale Effect. Mem. Int. Assoc. Hydrogeol. 17, pp 267-277.

Earlougher R. C. 1977. Advances in Well Test Analysis. Monograph Volume 5, Henry L. Doherty Series, New York, USA

Gustafson, G., Stanfors, R., Wikberg, P., 1988: Swedish Hard Rock Laboratory - First Eevaluation of Preinvestigations 1986-87 and Target Area Characterization. SKB Technical Report 88-16, June 1988.

Goodman, R.F., Moye, D.G., Van Schaikwyk, A., Javendel, I. 1965. Groundwater inflows during tunnel driving. Bulletin of the International Association of Engineering Geologists, Vol 2(1):39-56.

Huyakorn, P.O., White, Jr., H.O., Wadsworth, T.D. 1987: TRAFRAP-WT. - International Groundwater Modeling Centre IGWMC-FOS33, 1987.

IAEA, 1985. Techniques for Site Investigations for Underground Disposal of Radioactive Wastes. Technical report series No 256.

KBS-3, Final Storage of Spent Nuclear Fuel. Swedish Nuclear Fuel Waste Management Co., SKB, 1983

Neretnieks I., Abelin H., Birgersson L., Moreno L., Rasmuson A., Skagius K. 1985. Chemical Transport in Fractured Rock. Proceedings of advances in transport phenomena in porous media. NATO Advanced Study Istitute. Newark, Delaware.

Neretnieks, I., 1986. Stationary Transport of dissolved Species in the Backfill Surrounding a Waste Canister i Fissured Rock: Some Simple Analytical Solutions. Nuclear Technology, Vol 72, February 1986, pp 194-200.

Olsson O., Forslund O., Lundmark L., Sandberg E., Falk L. 1986. Borehole Radar - A New Technique for Investigation of Large Rock Volumes. Swedish Geological, Id-nr: IRAP 86292.

Olsson, O., Black, J., Cosma, C., Phil, J. 1987. Crosshole Investigations - Final Report. Stripa Project, Technical Report TR 87-16.

Priest S.D., Hudson J.A. 1981. Estimation of Discontinuity Spacing and Trace Length using Scanline Surveys. International journal of rock mechanics, Vol. 18, pp 183-197.

Rasmuson A., Skagius K. 1985. Migration in Strongly Fissured Zones - The Sensitivity to Some Assumptions and Parameters. Water Resources Research. Vol 22, pp 559-569.

Robinsson, P.C., 1982. Connectivity of Fracture Systems - a Percolation Theory Approach. J. Phys. A: Math. Gen. 16, pp 605-614.

SGAB. 1987. SBL - Rock laboratory. Proposal for investigations. Swedish Geological Co. Id-no: IRAP 87402.

Simmons, G.R. 1988. Geotechnical Research at Atomic Energy of Canada Limited's Underground Reasearch Laboratory. Proc. of Can. Nuclear Soc. Int. Symp. on Uranium and Electricity. Saskatoon, Saskatchewan 1988, Sept 18-21.

SKB 86-31. Swedish Nuclear Fuel and Waste Management Co., SKB, Annual report 1986, Stockholm, Sweden 1987

SKB. 1988. Swedish Nuclear Fuel and Waste Management Co., SKB. Annual report 1987. Stockholm, Sweden.

SKB 1988. Technology Needs for Selecting and Evaluating High-Level Waste Respostory Sites in Crystalline Rock. Technical Report BMI/OWTD-3. December 1988.

SKN, Statens Kärnbränslenämnd, 1985. NAK WP-Cave Project, Report on the Research and Development Stage, May 1984 to October 1985, Boliden WP-Contech AB. SKN Report 16, Stockholm, Sweden.

Stille, H., Fredriksson, A., Widing, E., Åhrling, G. 1985. Rock Mechanical Calculations. FEM-analysis of Silo and connecting tunnels. SKB Project SFR Report 85-09, Stockholm, Sept 1985 (In Swedish).

Streltsova, T.D. 1983. Wellpressure behaviour of a naturally fractured reservoir. Society of Petroleum Engineers, 1983, pp 769-780.

Streltsova, T.D. 1984. Buildup Analysis for Interference Tests in Stratified Formations. Journal of Petroleum Technology, Feb 1984, pp 301-310.

Streltsova, T.D. 1988. Well Testing in Heterogeneous Formations. An Exxon Monograph, John Wiley & Sons, Inc, New York.

Tirén, S.A., Beckholmen, M., 1988. Structural Analysis of Contoured Maps Kärrsvik-Bussvik, Lilla Laxemar and Glostad Areas, Simpevarp Area, Southeastern Sweden. SKB, Swedish Hard Rock Laboratory, Progress Report 25-87-27, Jan 1988.

Wilke, S., Guyon, E., de Marsily, G., 1985. Water Penetration Through Fractured Rocks: Test of a Tridimensional Percolation Description. Mathematical Geology. 17, pp 17-27.

Winberg, A., Carlsson, L. 1987. SFR. Calculation of conductive fracture frequency and description of skin effects around tunnels in the rock cavern area, SFR, Forsmark. Report in Swedish to SKB, June 1987.

Wyllie, M.R.J., et al. 1958. An Experimental Investigation of Factors affecting Elastic Wave Velocities in Porous Media, Geophysics 23 3 193.

Yoshida, M., Yoshimura, K. 1970. Deformation of Rock Mass and Stress in Concrete Lining around the Machine Hall of Kisenyama Underground Power Plant. Proc. from the 2nd Congress Int. Soc. Rock Mech., Volume 2, pp 593-607.

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