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First workshop on design and construction of deep repositories - Theme: Excavation through water-conducting major fracture zones Såstaholm Sweden, March 30-31 1993

Göran Bäckblom (ed.), Christer Svemar (ed.)

Swedish Nuclear Fuel & Waste Management Co, SKB

January 1994

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FIRST WORKSHOP ON DESIGN AND CONSTRUCTION OF DEEP REPOSITORIES - THEME: EXCAVATION THROUGH WATER-CONDUCTING MAJOR FRACTURE ZONES
SÅSTAHOLM SWEDEN, MARCH 30-31 1993

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Information on SKB technical reports from 1977-1978 (TR 121), 1979 (TR 79-28), 1980 (TR 80-26), 1981 (TR 81-17), 1982 (TR 82-28), 1983 (TR 83-77), 1984 (TR 85-01), 1985 (TR 85-20), 1986 (TR 86-31), 1987 (TR 87-33), 1988 (TR 88-32), 1989 (TR 89-40), 1990 (TR 90-46), 1991 (TR 91-64) and 1992 (TR 92-46) is available through SKB.

FIRST WORKSHOP ON DESIGN AND CONSTRUCTION OF DEEP REPOSITORIES – Theme: Excavation through Water-conducting Major Fracture Zones

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Göran Bäckblom (ed)

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Swedish Nuclear Fuel & Waste Management Co, SKB

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FOREWORD

The vast experience gained in underground construction of mines, hydropower stations, hydrocarbon storages, road tunnelling etc. under very different conditions is considered to provide the major basis for the construction of repositories for long-lived high-level nuclear waste. Although strategies, methods and approaches have been tested and experienced in the underground construction industries the results may not be directly transferable to the repository projects as there are differences in the basic, requirements and demands between the conventional underground projects and the repository projects.

With the Swedish Deep Repository Project moving into design and construction phases the questions of how to approach all the practical matters and how to handle all situations of underground construction are in progress of being addressed by SKB. In this planning process it was considered to be of great advantage if topics could be raised for discussions in a group of international designers and underground engineers with a mutual interest in nuclear waste repositories and with a developed know-how of the specific conditions that will guide the design and construction of a repository. SKB consequently launched the idea of starting a series of international workshops dealing with the different aspects of repository design and construction.

The meeting at Såstaholm is the first one in this series. The main topic was suggested to be experience from excavation through major waterconducting zones. In order to high-light also other issues involved in underground design and construction a session was also suggested for a more broad discussion.

The group of close to 40 persons proved to be convenient and effective.

During the meeting the discussions were recorded, but to our disappointment with some malfunction resulting in a few missing questions and comments. We have, however, in our editorial work excluded less relevant remarks and other parts which we judged to be of no interest to document for the future. The focus in the proceedings are on the presentations, which we have incorporated completely as they were presented to us in the written contributions.

Göran Bäckblom *Christer Svemar*

ABSTRACT

Final disposal of high-level nuclear waste has not yet been carried out in any country today. The concepts under development are all based on geological repositories, i.e., disposal at a sufficient depth below the surface to provide stable mechanical, hydrological and chemical conditions during the period the waste needs to be isolated from man. In the cases where crystalline bedrock is considered the proposed repository depths vary between 300 – 1000 m.

The construction, operation and sealing of a deep geological repository must meet various criteria that in many respects are more detailed and more demanding than usual in underground construction projects today. The work shall be done so that occupational safety is ensured. The work also shall conform to whatever restrictions are necessary for ensuring pre-closure operational safety and post-closure long-term safety.

March 1993 SKB arranged a two-day international workshop to discuss design and construction of repositories. Close to 40 participants from eight countries shared experiences regarding passage of major water-conducting fracture zones and other matters. This report summarizes the contributions to the workshop.

SAMMANFATTNING

Slutlig deponering av högaktivt kärnavfall har ännu inte påbörjats i något land. De förvarskoncept som är under utveckling baseras på geologiska förvar, dvs förvaring långt ner i berget (300 – 1000 m). Berget ger då mekaniskt skydd, låg grundvattenströmning och stabil kemisk miljö under den tid som avfallet är farligt för människan.

Anläggande, drift och försegling av ett djupförvar ska uppfylla många krav. Dessa är i många fall mer detaljerade och mer krävande än vad som vanligtvis gäller för undermarksbyggnad i dag. Arbetet ska genomföras under fullt betryggande personsäkerhet. Det ska också följa de krav och restriktioner som är nödvändiga för att försäkra sig om en säker drift av förvaret och för en säker slutförvaring.

SKB arrangerade i mars 1993 ett två-dagars internationellt arbetsmöte för att diskutera projektering och utförande av djupförvar. Närmare 40 deltagare från åtta länder diskuterade erfarenheter från tunneldrivning genom stora svaghetszoner m m. Denna rapport summerar bidragen till arbetsmötet.

Keywords: Design, construction, excavation, grouting, sealing, fracture zones, emplacement, repository operation.

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1 INTRODUCTION

Göran Bäckblom – Christer Svemar,
Swedish Nuclear Fuel and Waste Management Co, Stockholm, Sweden

Final disposal of high-level nuclear waste has not yet been carried out in any country today. The concepts under development are all based on geological repositories, i.e. disposal at a sufficient depth below the surface to provide stable mechanical, hydrological and chemical conditions during the period the waste needs to be isolated from man. In the cases where crystalline bedrock is considered, the proposed repository depths vary between 500 – 1000 m.

A recent study of the Swedish repository concept /SKB 1992/, Figure 1-1, concludes "... that a repository constructed deep in Swedish crystalline basement rock with engineered barriers possessing long-term stability fulfils the safety requirements proposed by the authorities with ample margin. The safety of such a repository is only slightly dependent on the ability of the surrounding rock to retard and sorb leaking radioactive materials. The primary function of the rock is to provide stable mechanical and chemical conditions over a long period of time so that the long-term performance of the engineered barriers is not jeopardized".

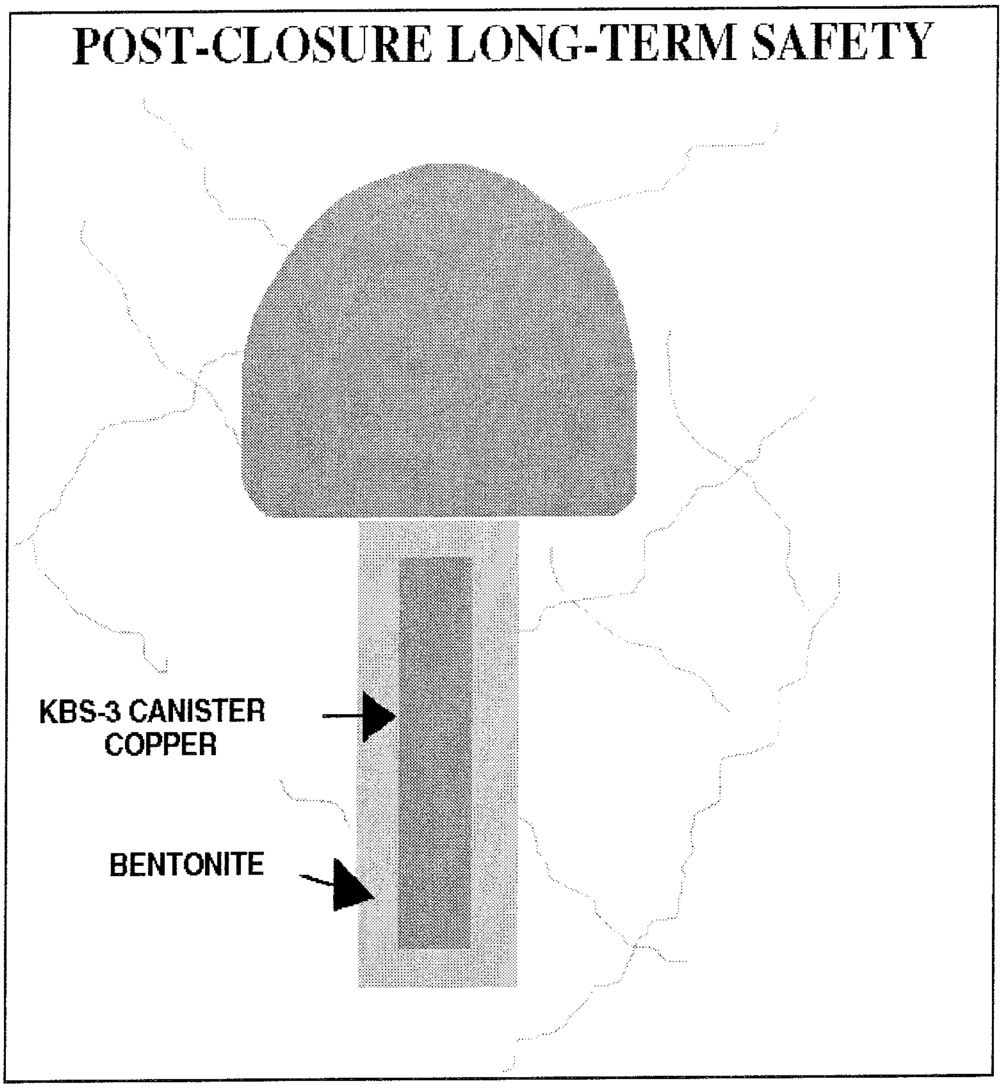
The construction, operation and sealing of a deep geological repository must meet various criteria that in many respects are more detailed and more demanding than usual in underground construction projects today. The work shall be done so that occupational safety is ensured. The work also shall conform to whatever restrictions are necessary for ensuring pre-closure operational safety and post-closure long-term safety.

One question of special concern in this workshop is the possible existence of discontinuities with high conductivity and high hydraulic pressure, Figure 1-2. These factors have an impact both on site characterization and constructability. One measure often taken in underground mining, for instance, is to lower the groundwater level considerably by pumping out the water. This facilitates both excavation and operation of the mine. This approach, however, might not be acceptable in a repository, because the pumping might cause oxidizing water to enter the repository level, which could be a disadvantage from the long-term safety point of view.

Instead the long-term safety aspect might call for more or less unchanged groundwater table, which can only be achieved by careful sealing of the excavations against leakage of water. The primary method for this would be pre-grouting of the rock followed by post-grouting after excavation.

Grouting of the rock is also being considered for sealing of less complex discontinuities, which to a large extent is common practice today. The aim is both to prevent drainage of the far field rock and to provide an acceptable working environment for handling of the water-sorbing bentonite clay. Three main objectives for development of grouting techniques have been identified in the Swedish R&D programme:

- Excavation through major water-bearing fracture zone.
- Sealing of canister holes in a KBS-3 type repository. An inflow per hole on the order of 2 l/h is assumed during deposition for preventing bentonite from swelling before the canister has been emplaced. This means that very fine fractures should also be attended to.



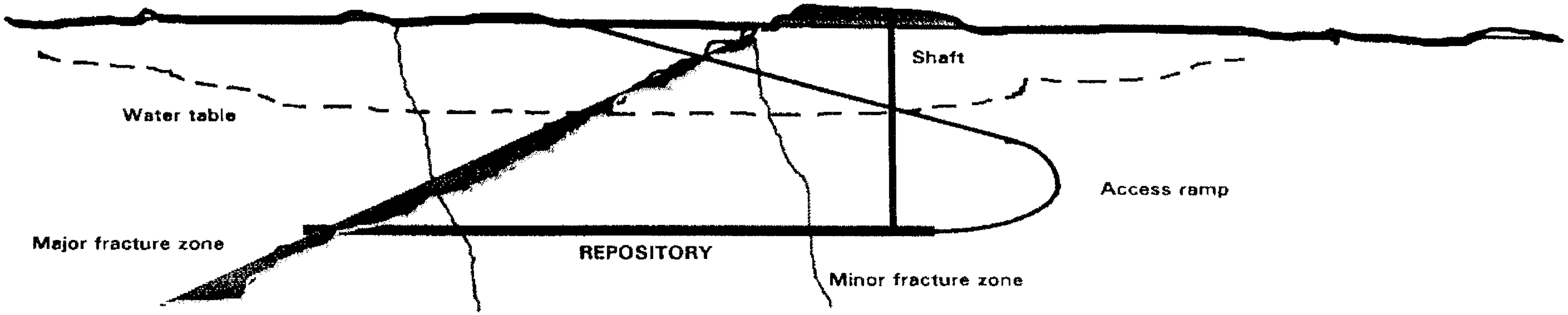
SKB 91 ... “shows that a repository deep in Swedish crystalline basement rock with engineered barriers possessing long-term stability fulfils the safety requirements proposed by the Swedish authorities with ample margin.

The safety of such a repository is only slightly dependent on the ability of the surrounding rock to retard and sorb leaking radioactive materials. The primary function of the rock is to provide stable mechanical and chemical conditions over a long period of time so that the long-term performance of the engineered barriers is not jeopardized.”

Figure 1-1. *Conclusion of the SKB 91 study.*

REPOSITORY DESIGN AND CONSTRUCTION

Workshop March 30-31 1993



QUESTIONS:

What water pressures can be handled ?

What inflow can be handled ?

What are the restrictions during construction ?
- **post-closure safety**
- **occupational safety**
- **pre-closure operational safety**

Figure 1-2. *Passage of major fracture zones.*

- Sealing of discontinuities in different excavations and the disposal drifts to prevent the groundwater table from sinking. A low water inflow into the disposal drifts also facilitates the handling of bentonite-sand mixtures during back-filling.

Naturally, grouting also will serve the purpose of providing occupational safety during repository construction, operation and sealing.

The time schedule in Sweden for siting and construction of the repository now calls for a start of assessments of the constructability of the underground parts, so that discussions can start with colleagues evaluating post-closure safety. These assessments shall be performed on site-specific data and plans for how the repository will be built. The engineers also need to know whether any restrictions must be considered with respect to long-term safety, for example acceptable amounts of concrete in different parts of the repository.

In the excavation of the Äspö HRL, a major water-bearing fracture zone was passed through in the spring of 1992. Fracture zone NE-1 (transmissivity 10^{-4} m^2/s, water pressure 1.7 MPa) is a complex fracture zone that required extensive grouting and reinforcement before excavation. Grouting theories with respect to amount of grout and its spread were set up and tested during 1991 – 1992. The evaluations are still under way. SKB has, however, defined a need to refine further methods to investigate and pass water-bearing fracture zones with high water pressure.

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2 PART I – EXCAVATION THROUGH WATER-CONDUCTING MAJOR FRACTURE ZONES

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2.1 CHARACTERIZATION OF A MAJOR WATER-BEARING FRACTURE ZONE TO OPTIMIZE THE LOCATION OF A VENTILATION SHAFT

Paul M Thompson and E T Kozak,
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2.1.1 INTRODUCTION

The excavation of tunnels and shafts through major water-bearing fracture zones is an issue of significant concern to engineers and scientists engaged in the study of deep geological repositories for the disposal of radioactive wastes. Occupational safety, constructability and operability of a repository, together with the long-term safety assessment, are key issues to be considered. Emphasis is usually placed on pre-treatment of the fracture zone using pressure-grouting techniques, followed by remedial grouting after passage through the zone. A different approach can be used that involves careful hydrogeologic characterization of the fracture zone to locate areas of low hydraulic conductivity in which to penetrate the zone with an excavation. In some cases, as we have shown at the Underground Research Laboratory (URL), the need for grouting can be entirely eliminated.

2.1.2 THE UNDERGROUND RESEARCH LABORATORY

AECL Research is conducting geotechnical research at the URL in southeastern Manitoba. The URL has been constructed to provide a representative environment in which to conduct large-scale multidisciplinary in situ experiments (Operating Phase experiments), designed to provide data required to assess the concept of disposal of nuclear fuel wastes deep within stable plutonic rock of the Canadian Shield */Simmons et al., 1992/*. The program is jointly funded by AECL and Ontario Hydro under the auspices of the CANDU Owners Group (COG).

The URL (Figure 2.1-1) has been developed in the granitic Lac du Bonnet Batholith. The site for the URL was previously undisturbed and was extensively characterized before and during construction of the URL. Construction began in 1983 after a comprehensive site characterization program. At the present time, the URL consists of a main shaft 443-m deep, and two main test levels at depths of 240 and 420 m. Upper and lower ventilation raises also connect the two levels to surface, and shaft stations have been excavated at depths of 130 and 300 m.

The geology and structural setting of the URL */Everitt et al., 1990/* are illustrated in Figure 2.1-2. The upper 200 m is essentially pink heterogeneous or xenolithic granite characterized by extensive vertical jointing. The low-dipping fracture zones (thrust faults) intersect the main shaft: Fracture Zone 3 immediately above the 130-m shaft station, and Fracture Zone 2 (FZ2), a major shear zone, below the 240 Level. A splay from FZ2 intersects the shaft immediately above the 240 Level. At the 240 Level the rock is essentially unfractured, homogeneous, grey granite with some pink alteration present at the north-west end because of the proximity of FZ2. Below FZ2 the rock is very sparsely fractured grey granite varying from homogeneous to heterogeneous to xenolithic. At the 420 Level the rock mass is composed of approximately 60% granodiorite, 30% grey granite, and 10% xenolithic granite */Everitt et al., 1990/*.

2.1.3 CHARACTERIZATION OF FRACTURE ZONE 2

One component of the research program at the URL has been the detailed characterization of FZ2. This type of geological discontinuity is common to the Lac du Bonnet Batholith and other similar plutonic rock bodies in the Canadian Shield. These fracture zones typically consist of a large zone of jointed rock and a highly sheared cataclastic zone. The extensive hydrogeological characterization of FZ2 is described by *Davison (1984)* and *Davison and Kozak (1988)*.

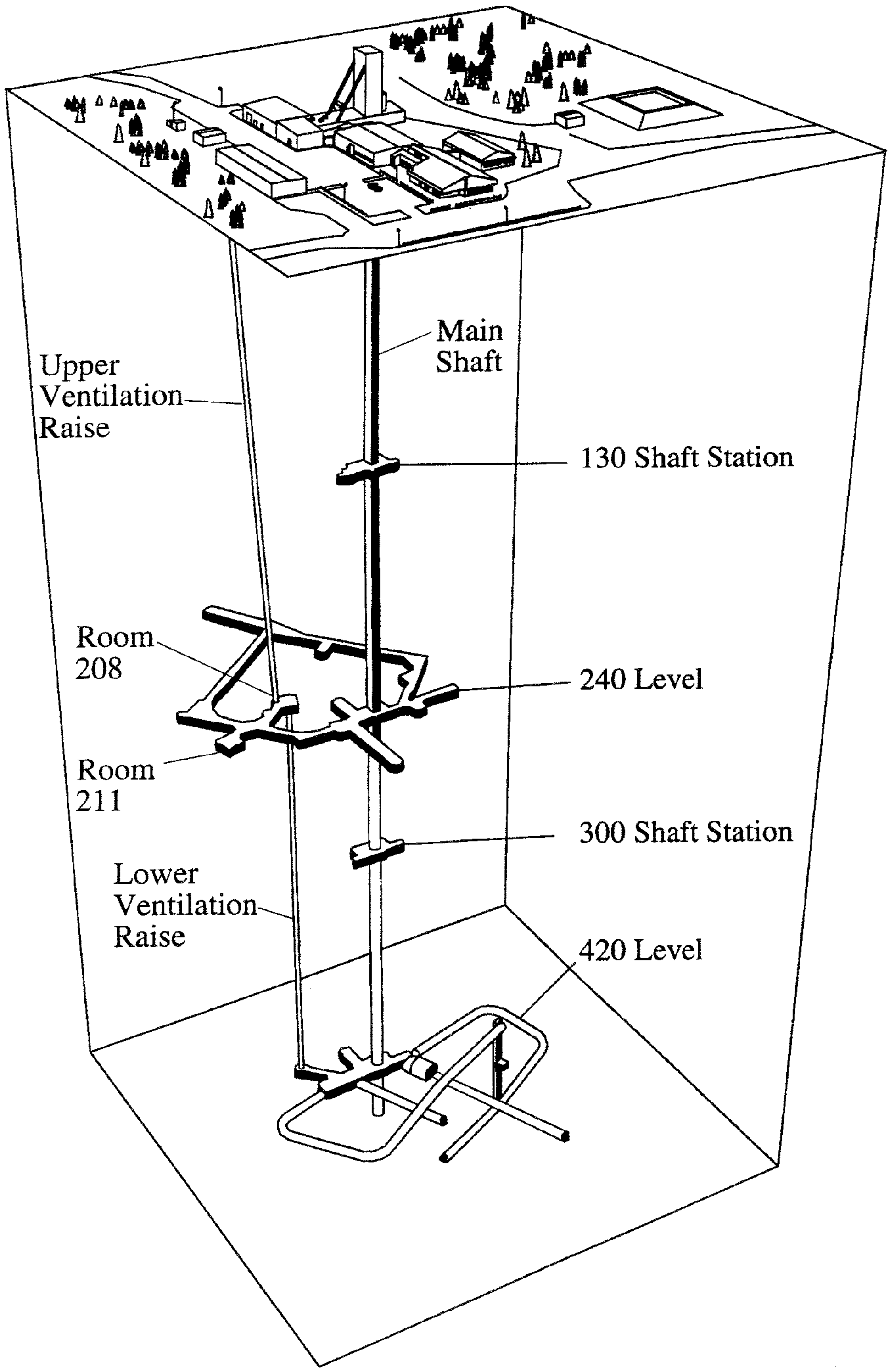

Figure 2.1-1. *Isometric view of the URL.*

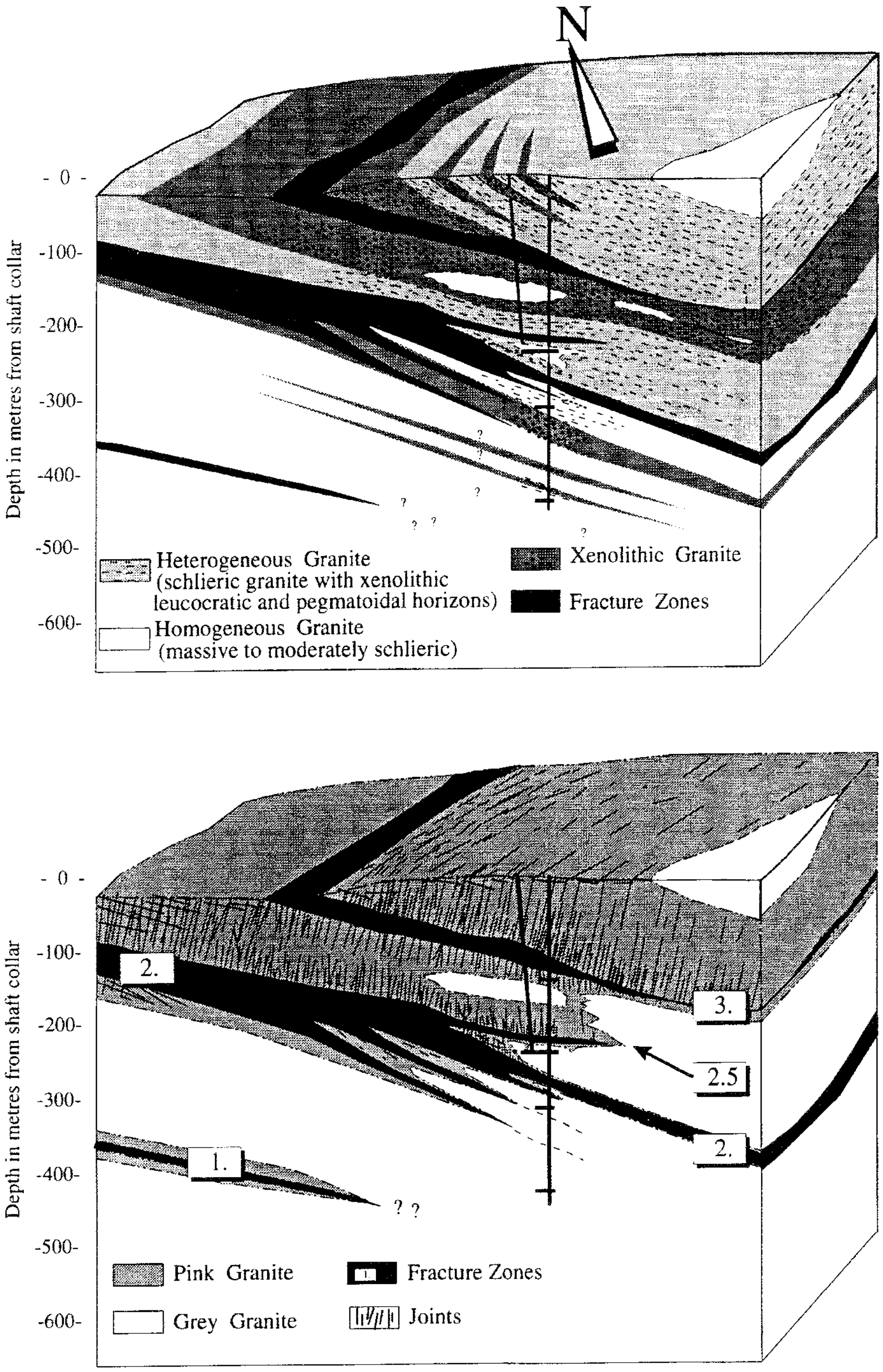

Figure 2.1-2. *Geological structure and lithology at the URL /after Everitt et al., 1990/.*

2.1.3.1 Surface characterization

Prior to any excavation at the URL site, over 180 boreholes were drilled from surface to characterize the subsurface geological and hydrogeological conditions on the 4 km^2 URL lease area. Of these, over 30 extended to depths greater than 150 m. The results of large-scale hydraulic pressure interference tests, single-borehole straddle packer tests, piezometric pressure distributions, and water chemistry were used to determine the permeability and hydraulic interconnectivity of the various major fracture zones at the site. An extensive network of multiple-interval piezometers was installed in the site charcterization boreholes to measure and record hydraulic pressure conditions in the rock mass prior to, during, and after excavation of the URL.

This information was used to plan the preliminary layout for the URL. It was decided to sink the main shaft to a depth of 255 m, about 15 m above FZ2. The main test level would be at 240-m depth allowing close access for further testing and characterization of this major hydraulic feature. Initially it was decided not to penetrate FZ2 because the resources were not available at the time to adequately characterize and carefully grout the fracture zone if it was required. If the penetration was not done carefully, there was a real possibility of permanently affecting groundwater flow and pressure regimes, which would compromise some of the hydrogeologic experiments planned for the URL. The URL shaft was sunk to a depth of 255 m between 1983 May and 1985 March, including excavation of the shaft station for the 240 Level.

2.1.3.2 Underground characterization

In 1986, Subsidiary Agreement No. 1 was signed by AECL and the United States Department of Energy (USDOE). The main focus of this cooperative program was to deepen the URL to provide experimental access to rock subjected to higher in situ stresses. In order to do this it would be necessary to extend the main shaft and bore a ventilation raise through FZ2. A thorough hydrogeological characterization of the zone below the 240 Level was undertaken because of concerns that the hydrogeology of FZ2 might be adversely affected. It was originally intended to locate the lower ventilation raise in a known high-transmissivity area of FZ2 in order to undertake a grouting experiment as part of the process. Room 211 (Figures 2.1-1, 2.1-3 and 2.1-5) was excavated for this purpose.

Thirty-eight continuously cored, 96-mm-diameter boreholes were diamond drilled from the 240 Level to intersect FZ2 at various locations for detailed geological, hydrogeological, geochemical, and geomechanical characterization. Figure 2.1-3 shows where the boreholes penetrated FZ2 relative to the 240 Level. Each of the hydrogeological boreholes from the 240 Level was drilled through the entire thickness of FZ2, and a multiple-packer monitoring system was installed in each hole for long-term piezometric monitoring, hydraulic testing and groundwater sampling */Thompson et al., 1990/*. A schematic diagram of the packer system is illustrated in Figure 2.1-4.

The spatial distribution of transmissivity within the 4 x 10^4 m^2 region of FZ2 beneath the 240 Level of the URL was determined by conducting short-duration hydraulic tests in all the boreholes and also performing several long-duration hydraulic pressure interference tests from selected boreholes. The single-hole tests were performed using multiple- and single-pressure-step fluid withdrawal methods and analysed using the methods outlined by Zeigler (1976). Two types of large-scale hydraulic tests were performed from these boreholes: constant pressure drawdown and constant rate fluid withdrawal. These hydraulic tests could be conducted without pumps because the static hydraulic pressure in FZ2 created pressures of about 2.0 MPa at the borehole

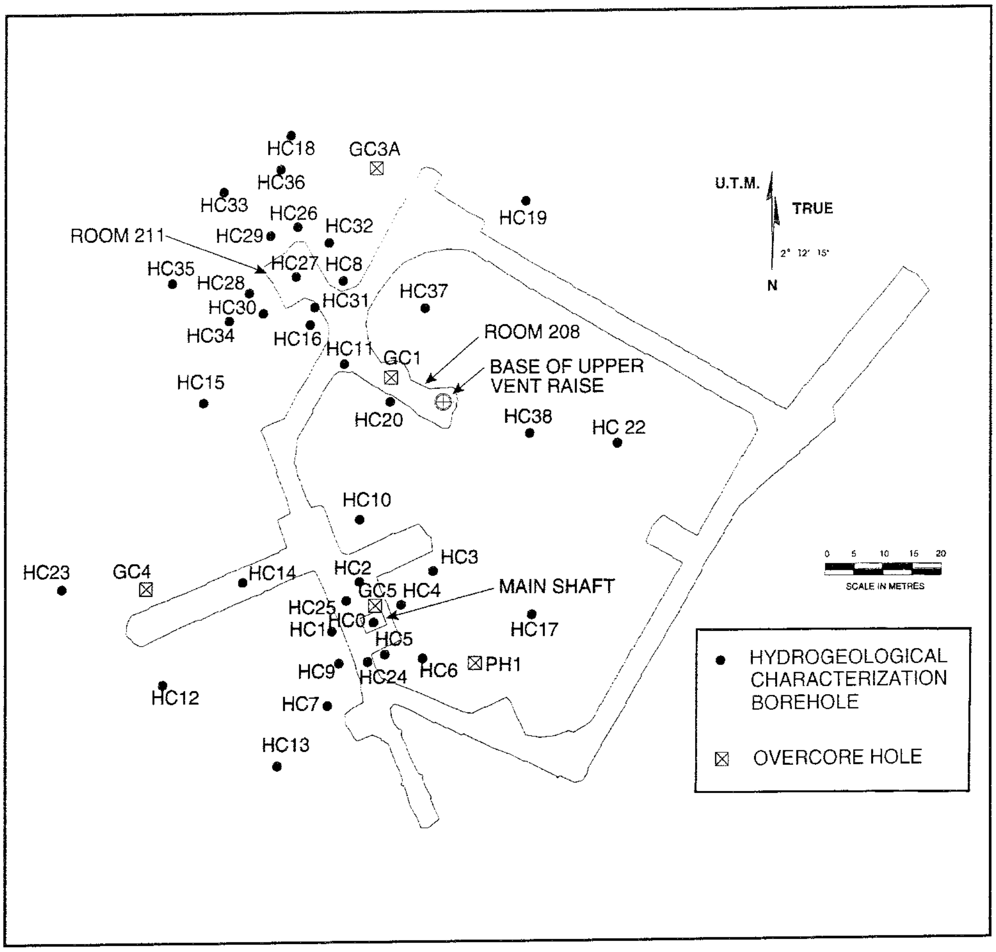

Figure 2.1-3. *Locations of intersections of hydrogeological characterization and overcore stress determination boreholes with Fracture Zone 2, superimposed on a plan view of the 240 m level.*

collars on the 240 Level. Indeed, much information could be obtained about the variability of hydraulic transmissivities across FZ2 by measuring the water flowing from the newly-drilled boreholes, and by monitoring the resulting hydraulic responses from nearby holes prior to installation of the packer systems. Some holes, drilled through high-transmissivity areas of FZ2, flowed at rates of up to 600 L/min, forcing a high velocity stream of water against the back of the room (5 m high), ejecting large volumes of sand and gravel-sized cataclastic material from the fracture zone in the process. Conversely, holes drilled through the low-transmissivity "tight" areas of FZ2 flowed at less than 10 mL/min.

In addition to the boreholes used for hydrogeologic characterization, five boreholes (Figure 2.1-3), collared from the 240 Level, were used for overcore in situ stress testing to determine the normal stresses acting across FZ2. Within each borehole between five and eleven overcore tests using the continuously monitored CSIR triaxial strain cell /*Thompson et al.*, 1986/ were carried out in the sound rock above and below the fracture zone.

In 1987, when the U.S. congress ended further studies related to granite as a potential disposal medium, the USDOE was forced to withdraw from Subsidiary Agreement No. 1. AECL completed development to the 420 Level. Because of limited resources, AECL decided to postpone the FZ2 grouting experiment and to relocate the lower

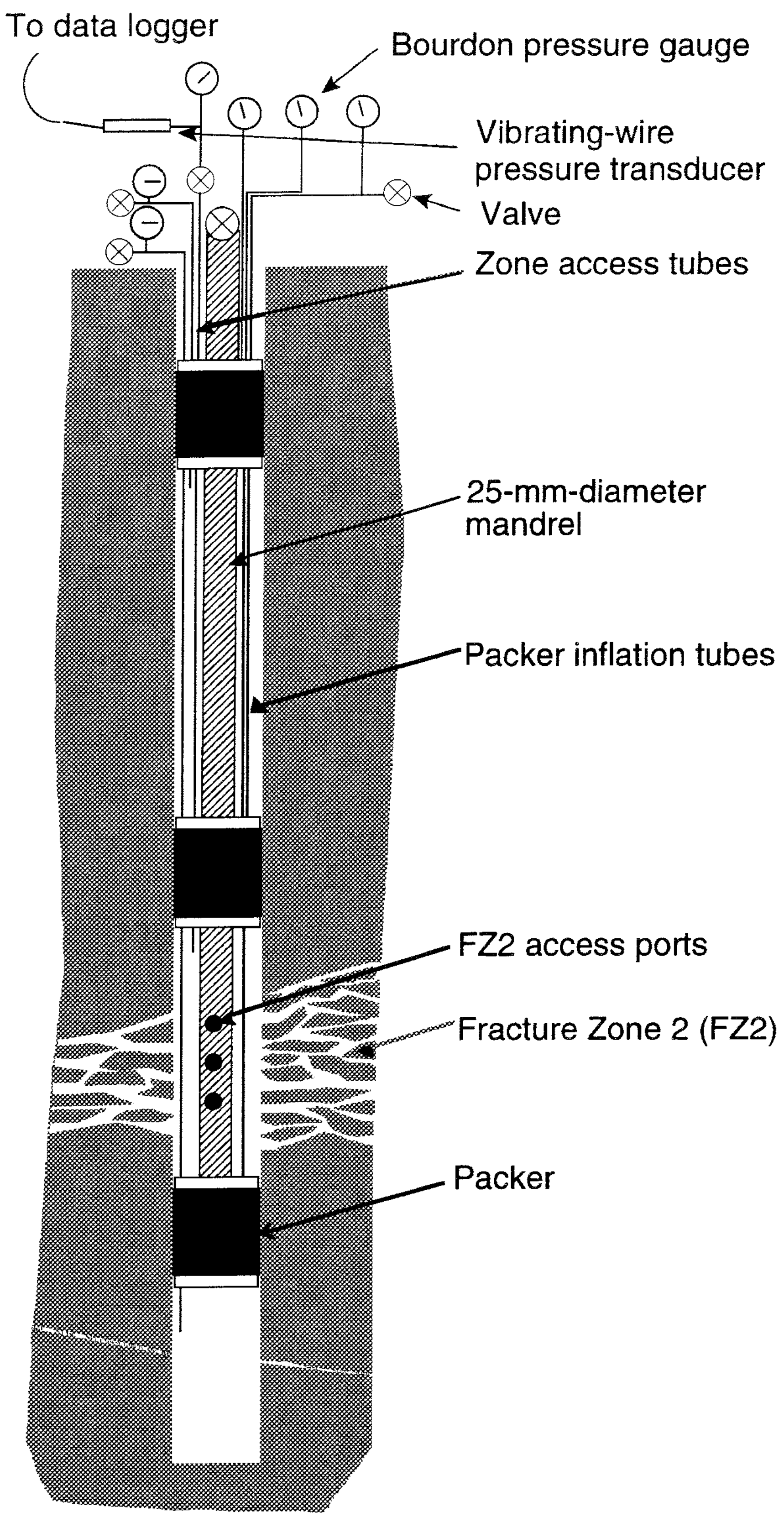

Figure 2.1-4. *Multiple-packer monitoring system used to characterise Fracture Zone 2.*

ventilation raise, so that it penetrated an area of lower hydraulic transmissivity in FZ2. The information obtained from the thorough characterization of the zone was used for this purpose.

2.1.4 CHARACTERIZATION RESULTS

The hydrogeologic testing revealed significant variations in hydraulic transmissivities across FZ2 as shown in Figure 2.1-5. The variations ranged up to six orders of magnitude for locations within 10 m of one another. Two distinctive high-transmissivity areas ($T \geq 10^{-5}$ to 10^{-7} m^2/s) were identified below the 240 Level, with limited hydraulic communication between one another. One is a high-permeability, low-storage basin with restricted recharge. The edge of this basin intersects the main shaft, but the detailed hydrogeologic characterization of FZ2 revealed that groundwater flow from this area into the main shaft would be minimal after an initial period of high flow. Nevertheless, a limited grouting program was undertaken prior to extending the main shaft */Gray and Keil, 1989/*, to test grouting materials and methods for future use

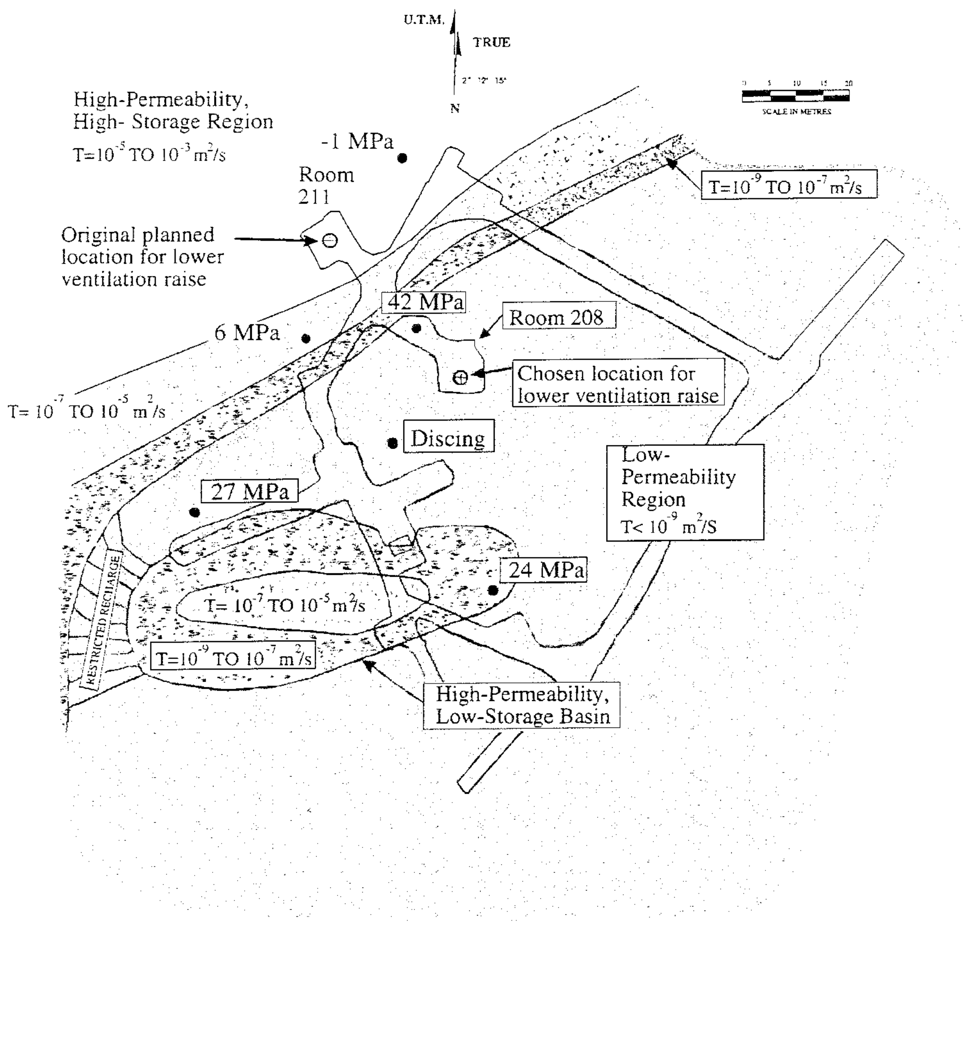

Figure 2.1-5. *Hydraulic transmissivity pattern and normal stress conditions in Fracture Zone 2 beneath 240 Level.*

at the URL. The other high-permeability region is located across the northwest portion of the study area underneath Room 211 where the ventilation raise had been planned originally. This part of the fracture zone is hydraulically connected to a large high-transmissivity region in FZ2, and therefore was capable of sustaining large inflows of groundwater if left ungrouted and penetrated by a shaft. Most other parts of FZ2 below the 240 Level were considered low-permeability regions with transmissivities ranging from 10^{-9} to 10^{-11} m^2/s.

Results from the normal stress measurements are also shown in Figure 2.1-5. The obvious correlation between the hydraulic tests and the stress measurements is that high normal stresses across FZ2 correspond to regions of low hydraulic transmissivities, while low normal stresses correspond to regions of high hydraulic transmissivities. In areas of high normal stresses, core discing was observed in the diamond drill core. The relationship between normal stress and the hydraulic properties of fracture zones is described in more detail by *Martin et al. (1990)*.

2.1.5 LOCATING THE VENTILATION RAISE

Based on the characterization program described, it was decided to penetrate FZ2 with the lower ventilation raise in a very low permeability region about 40 m southeast of its original planned location (Figure 2.1-5). Borehole HC20 drilled in this area produced less than 10 mL of water per minute. In choosing the new location, prime consideration was given to economics by minimizing the amount of new excavation required, to safety and health regulations which required at least 30 m separation between the main shaft and the ventilation raise, and to the operational aspects of the ventilation system. From strictly a groundwater control perspective, the ventilation raise could have penetrated FZ2 at any location in the low permeability region ($T<10^{-9}$ m^2/s) shown in Figure 2.1-5, and not have required any grouting.

The ventilation shaft was raise-bored from Room 208 to a diameter of 1.8 m with no pre-grouting (or post-grouting) required. After completion, water inflow from FZ2 to the raise was measured at 35 mL/min, and quickly evaporated into the ventilation air stream. Therefore the chosen location met the objective of allowing penetration of FZ2 without significantly affecting the hydraulic regime. If the original location in Room 211 had been used, a major grouting effort would have been required, and the hydraulic response of FZ2 would have affected groundwater pressures over the entire lease area, compromising the use of FZ2 for future testing and experimental purposes.

The only problems encountered during the raise-boring were stress-related. High normal stresses across FZ2 at the chosen location caused the formation of well-bore breakouts in the raise walls. Vibration and chattering of the raise bore reaming head and drill rods were the only symptoms apparent during passage of the reaming bit through the fracture zone.

2.1.6 DISCUSSION

Although information from a detailed hydrogeological, geological, and geomechanical characterization effort was used at the URL to optimally locate the ventilation raise, such a major investigation may not be needed in all situations. At the URL we required a thorough knowledge of the hydrogeologic conditions across FZ2 for planning major hydrogeologic experiments and for input to the site geotechnical model. However, on other projects it would still be necessary to drill and perform hydraulic tests on a large number of holes penetrating the zone at various locations, and to seal them either with packers or by grouting immediately after the drilling and testing.

2.1.7 CONCLUSIONS

A thorough characterization program was used at the URL to select the location for the ventilation shaft between the 420 and 240 Levels. The raise bored ventilation shaft penetrated FZ2 at a location where the hydraulic transmissivity was six orders of magnitude lower than nearby portions of the zone. The characterization program detected extreme variabilities in the hydraulic properties of the fracture zone over relatively short distances. The vent raise was excavated through the fracture zone with no need for grouting, and final measured water inflows were negligible. If the vent raise had been located in a high-transmissivity portion of the fracture zone, a significant grouting and sealing effort would certainly have been required.

Although it may not always be possible to choose the location where an excavation penetrates a water-bearing fracture zone depending on geological conditions or for logistical, operational, or schedule reasons, this method should be used where possible. Not only do construction costs and the need for extensive remedial work decline, but the final point of penetration is likely to improve occupational safety, and the constructability and operability of a repository, as well as enhancing long-term performance.

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2.2 FINNISH EXPERIENCES OF SEALING WATER-CONDUCTING ZONES IN ROCK

Reijo Riekkola, Saanio & Riekkola Consulting Engineers, Finland

ABSTRACT

Some case histories of sealing fractured zones in rock are described. Sealing methods used in the cases are pre- and postgrouting as well as freezing of the soil and rock.

2.2.1 CASE HISTORIES

2.2.1.1 Sea Sewer, Harjusuo

First case history describes the sealing work carried out in the Sea Sewer tunnel in the Harjusuo area */Ojanen et al., 1983/.* The tunnel penetrated a 60 m rock zone which was partly chemically weathered in a way that made the rock very porous. The zone also consisted partly of nearly horizontal clay-filled fractures. The zone conducted water considerably but the grout intake was poor. Groundwater pressure was about 0.3 MPa.

The zone was pregrouted with cement and resins, but the inflows in the tunnel remained at a level that made shotcreting impossible. The aim was to seal the tunnel by grouting the rock after shotcreting and casting the floor slab, which were considered necessary for use of high grouting pressures. Only after postgrouting was shotcreting possible. First, layers of shotcrete with accelerators were sprayed. A

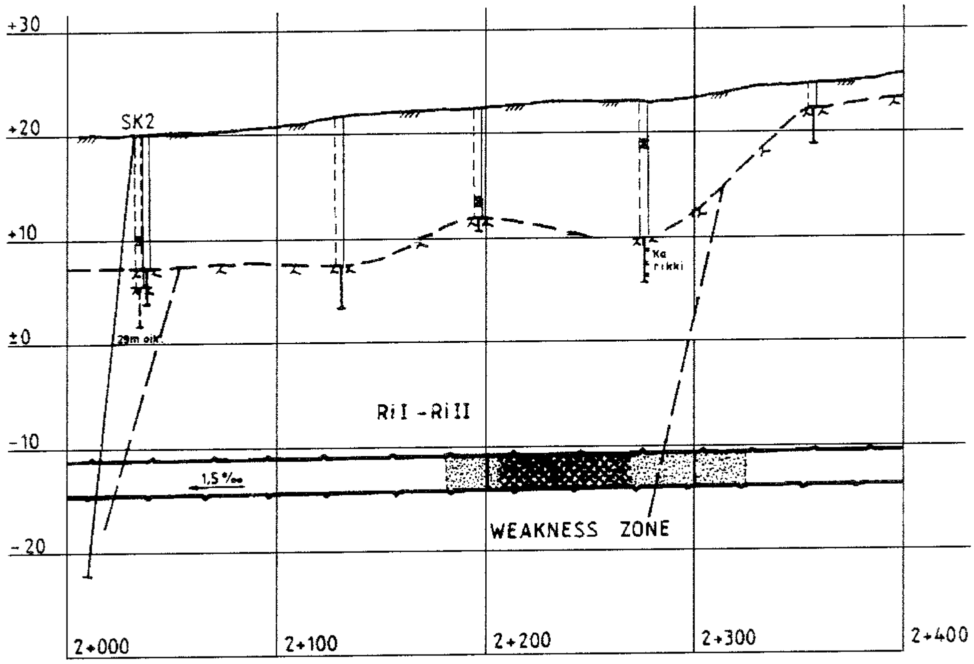

Figure 2.2-1. *Longitudinal section of the Sea Sewer sewage tunnel in the area of the Harjusuo zone of weakness.*

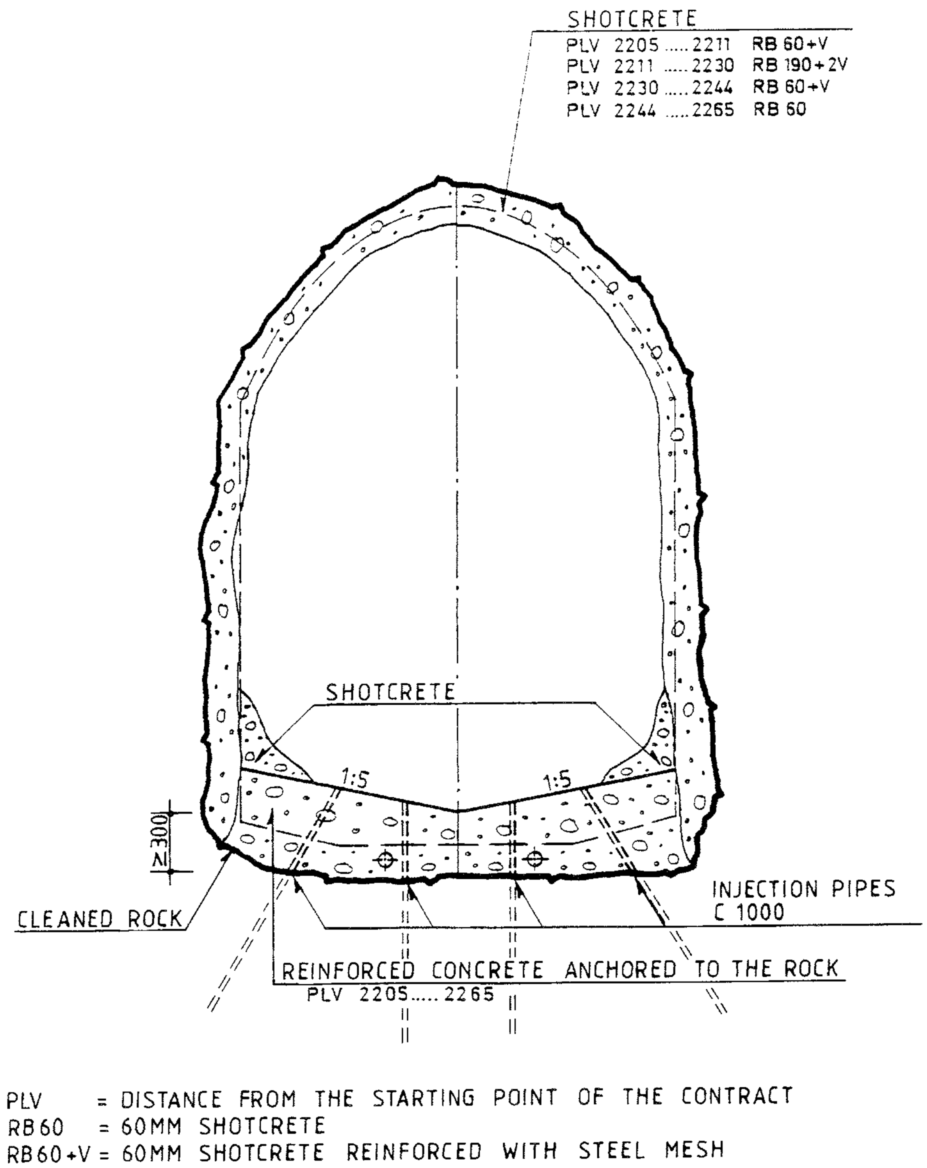

Figure 2.2-2. *Sea Sewer. Cross section of the sewage tunnel in the grouted area.*

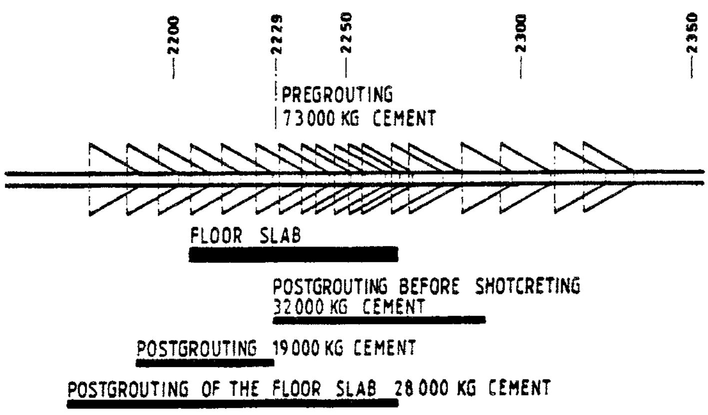

Figure 2.2-3. *Sea Sewer. Grouted area and number of pregrouting rings.*

reinforced concrete slab was cast on the cleaned rock floor and the floor was grouted through and against the slab. Postgrouting was completed after shotcreting.

Total cement intake was about 150 tons corresponding about 1 250 kg/grouted tunnel-m. Consumption of chemical grouts varied from 20 to 116 kg/grouted tunnel-m. The final inflows were reduced to about 100 m^3/day.

2.2.1.2 VLJ Repository

The second case history describes the sealing work carried out in the VLJ Repository in Olkiluoto. The repository is situated in tonalitic rock. The transportation tunnel penetrated an inclined fractured zone RC at two points. The lower part of the zone conducted water into the tunnel at a rate of 30-35 l/min.

The rate of inflow was not considered appropriate to transportation activities and a decision was made to seal the fractured zone by grouting so that the area could be shotcreted and any remaining inflows diverted to drainage systems.

The zone was postgrouted with cement. The w/c ratio was 0.5 and melamine based superplasticizer was used. The grouting program consisted of 18 holes with a length of 5-20 m.

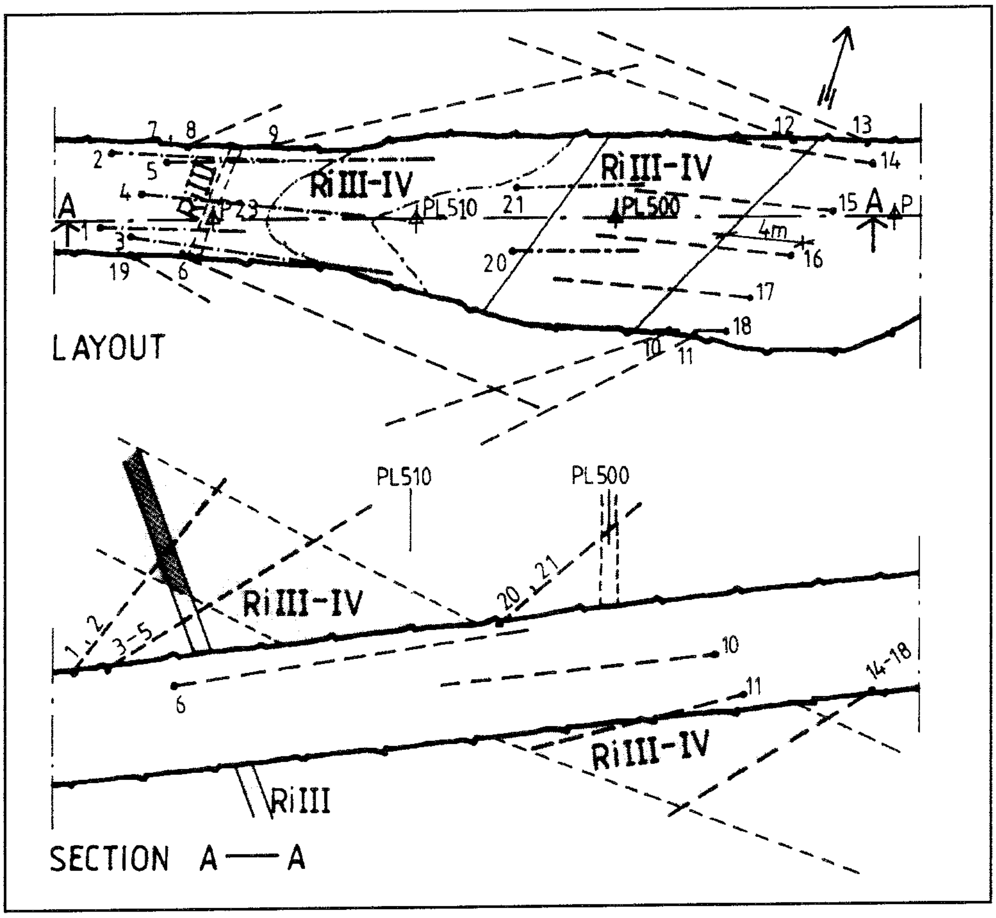

Figure 2.2-4. VLJ Repository. Grouting plan of the RC zone in the access tunnel.

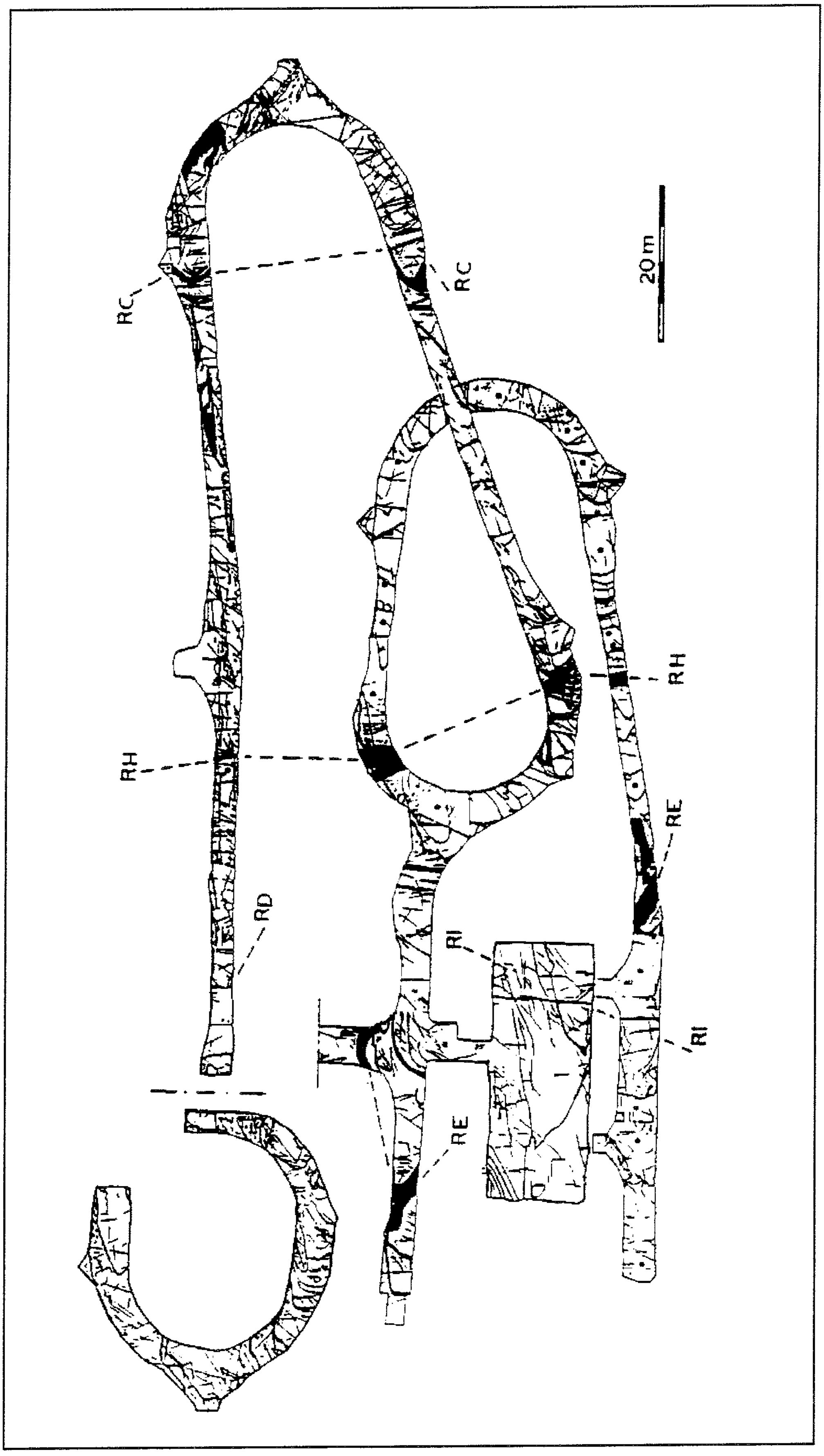

Figure 2.2-5. *VLJ Repository. Geological mapping, lay-out /Ikävalko et al., 1991/.*

Cement intake varied from 0 to 1120 kg/grouting hole. The inflows were first reduced to about 8 l/min. After redirection of the flow paths, the inflows returned to their original level.

2.2.1.3 Helsinki Metro/Kluuvi Cleft

In the Helsinki City Centre area the metro tunnels pierce the rock roof for a distance of 20 m and the tunnels at this location had to be constructed using the soil freezing method. A cast iron lining forms the final structure. This work site was located 30 m below the street surface under one of the busiest intersections. The section to be frozen was 50 m long in both track tunnels.

The most demanding phases of the construction work were the construction of the drilling chambers for the freezing pipes protected only by a very thin rock roof, the drilling of the casings for the freezing pipes from the rock into the soil horizontally, and the installation and use of the freezing system together with the air compression system which was used as a precautionary system. Blasting through the frozen soil and rock demanded an extremely cautious and slow work method, and it was done simultaneously with the installation of the cast iron cladding */Leppänen, 1978/.*

The ice cover was sized to withstand the soil and water pressure around it during tunnelling. The ice shell was designed in such a way that the compressive stresses arising in the ice shell did not exceed the ultimate compressive strength of the frozen soil within the loading time of the structure. The thickness of the ice shell was 2.5 m. Freezing was carried out with the aid of a direct freezing system. The refrigerant was Freon R 22. The volume that had to be frozen totalled 3 600 m^3 of soil and 7 200 m^3 of rock */Vuorela et al., 1978/.*

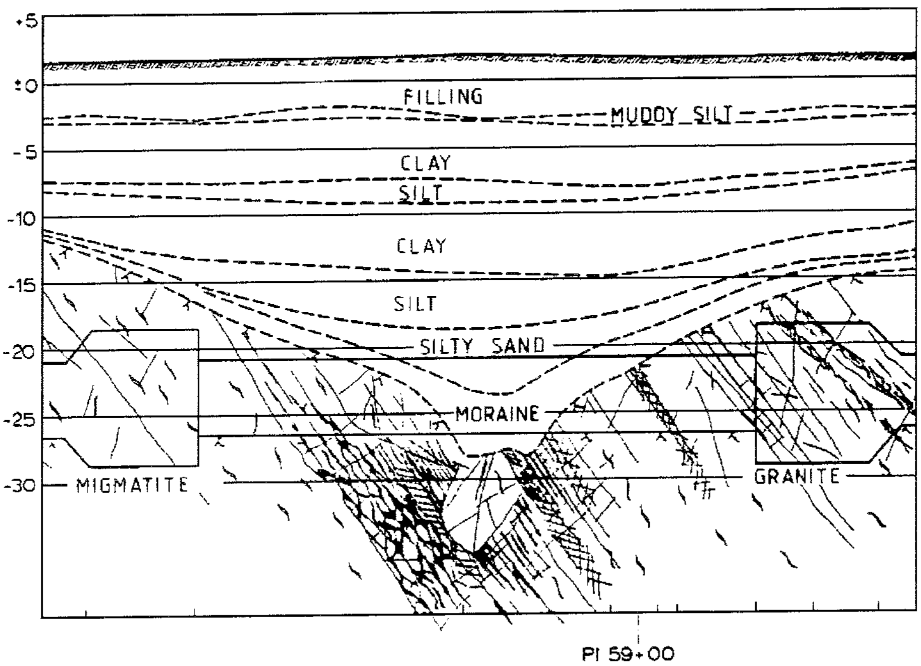

Figure 2.2-6. *Soil and rock conditions of the Kluuvi Cleft /Hartikainen et al., 1978/.*

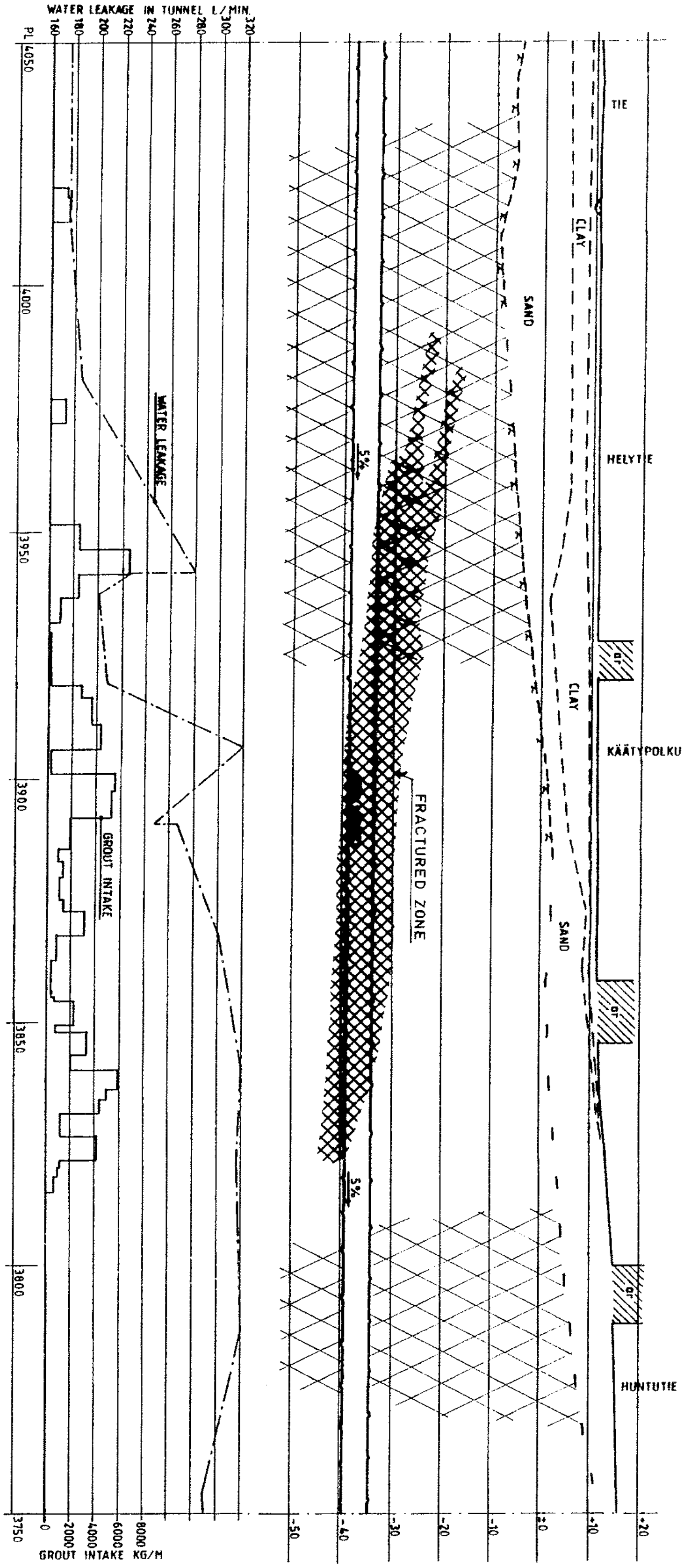

Figure 2.2-7. *Vuosaari-Pasila district heating tunnel. Water leakages and grout intake /Kähönen Y, 1993/.*

2.2.1.4 District heating tunnel Vuosaari-Pasila, Mustapuro area

Sealing measures in district heating tunnel Vuosaari-Pasila in the Mustapuro area entailed grouting of about 130 m tunnel. The rock in the area is granite and micagneiss. The water- conducting zone is a nearly horizontal fractured zone. The fractures are mainly open with apertures on the order of a few centimetres. In part of the tunnel, a 10-20 cm wide fracture with sandy filling can be seen */Raudasmaa, 1993/*. The highest inflows were several hundred litres per minute.

Grouting was carried out with cement and normally started with high w/c ratios. A lower w/c ratio (0.5) was used in connection with the open fractures. Pre- and postgrouting were done. The inflows after pregrouting were about 200 l/min. Groundwater pressure is about 0.4 MPa. Postgrouting work is still underway */Kähönen, 1993/*.

2.2.1.5 Viikinmäki central wastewater treatment plant

Grouting measures at the Viikinmäki central wastewater treatment plant consisted of grouting of the area around the digesters and grouting of the fractured zone in the aeration and presedimentation tunnels. Pregrouting of the fracture zones in the tunnels was planned according to field investigations, which showed some tight and some heavily water-conducting areas in the fractured zone. During construction, however, the intake of grout in the zone proved to be minimal and the grouting operations were reduced.

2.2.2 CONCLUSIONS

Development of investigation methods is needed to get a more realistic picture of zones to be grouted. Examples exist of considerable over- and underestimates of grouting measures needed.

Sealing of the floor in tunnels is the hard part of the job. Sealing of tunnels by grouting is usually expensive if no inflows are allowed. Alternative or supplementary techniques should also be considered in difficult cases.

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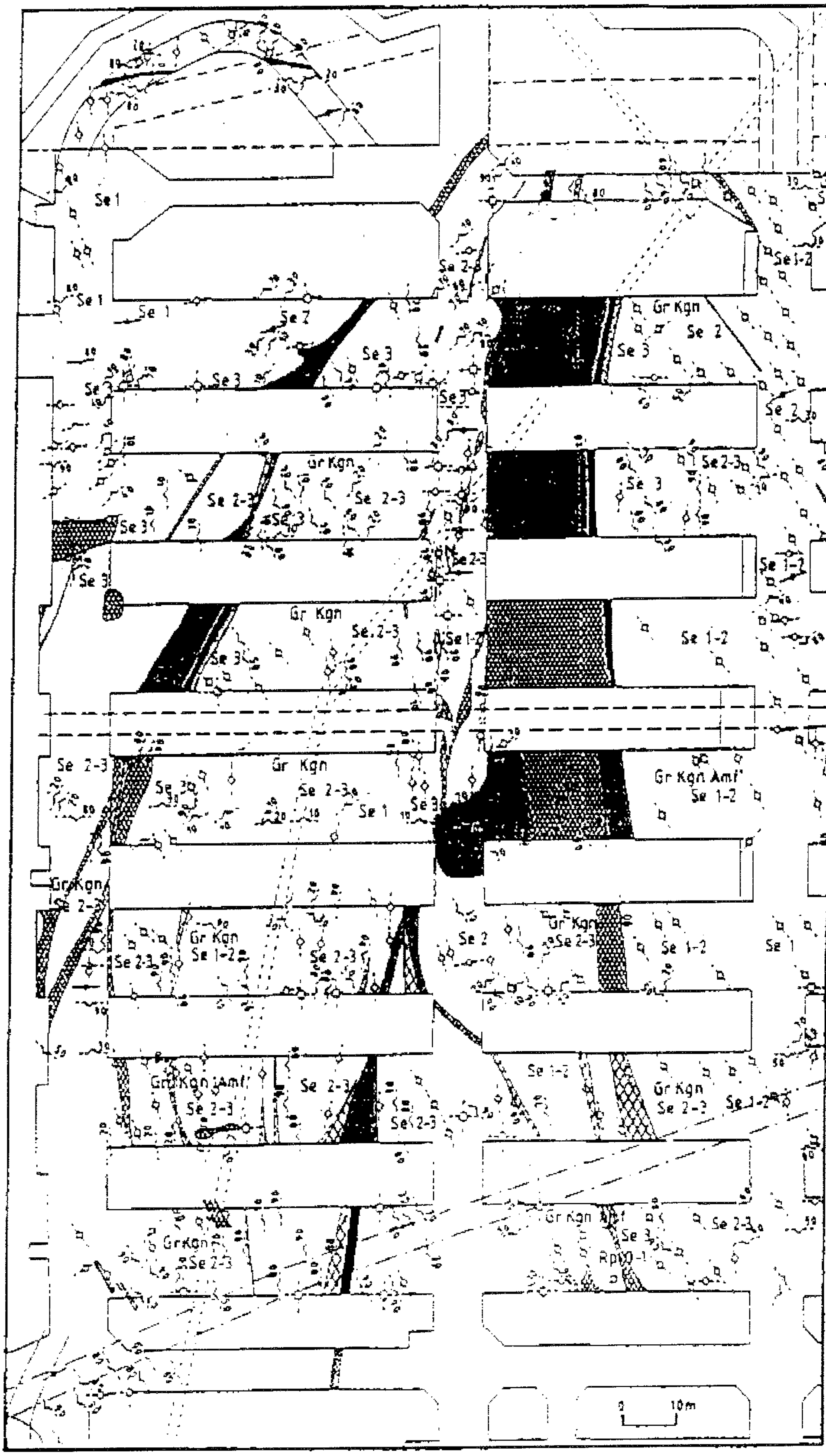

Figure 2.2-8. *Central wastewater treatment plant, Helsinki. Geological mapping results of the preliminary sedimentation and aeration tunnels /Niinimäki et al., 1992/.*

Ojanen H. and Riekkola R., Tunnel main sewer between Savio and Viikki. Rakennustekniikka 1983:1. Helsinki (in Finnish).

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Questions to Mr Reijo Riekkola

Mrs Maria Onofrei: Thinking of the state of hardening of a mixture. Do you think that could have been one of the reasons why you had some bad luck with the grouting?

Mr Reijo Riekkola: There was not really any hardening of the grouting material, as there was no time for that, but if you think about the range of materials which was used, it was all cement and chemical materials with very low viscosity and even that did not help too much.

Mrs Maria Onofrei: When you stopped the grouting at refusal you gave a lot of quantities of grout pumping. Did you stop because of the quantities or did you stop because you could not pump further?

Mr Reijo Riekkola: Usually the criteria, when you stop the grouting, is a kind of combination of the pressure and the consumption. At a certain pressure you must have a certain consumption.

Mr Brendan Breen: In looking at the sewer tunnel, did you at any stage consider the application or use of pressure and a secondary lining?

Mr Reijo Riekkola: It was discussed during the excavation, but when it actually happens you do not have any time to make a new design and when you want to say that you have already started something, it is easy to believe in what you are doing, and you just continue. It seems such a radical change to make a completely new design. But one conclusion from that special area is, that when we encountered the next fracture zone we had a design already available for the structure.

Mr Dwayne Chesnut: One of the ruptures showed that for the repository you had the water inflow measured along the tunnel floor as a function of distance and time.

Mr Reijo Riekkola: There were some other measurements at other points.

Mr Dwayne Chesnut: Basically you saw no long term effect on the grouting?

Mr Reijo Riekkola: Not on the total inflow rates. We did not have any effect.

Mr Göran Bäckblom: Can we classify zones according to groutability? It seems that the delay you had in the Helsinki sewer tunnel was 6 months. It is not uncommon that you get construction projects where you often hear that you have very severe problems with grouting of fracture zones. And often I get the impression that you get complex grouting problems when you are in complex zones. If you have a complex fracture zone you have different kinds of fractures, you have different kinds of gouge materials and the more complex the zone is the more complex the grouting will be. Do you have any idea – based on a previous job – how to figure out some kind of a rock classification system, to characterize which zones are easy to grout and which zones are not easy to grout?

Mr Reijo Riekkola: It is not easy to answer. It seems that you have to measure the water conductivity, but you cannot use any other material. You have to do your development mainly based on measurements of water conductivity and use some other complementary methods like geophysics.

Mr Dwayne Chesnut: Do you have rock problems where you do not have water problems?

Mr Reijo Riekkola: Yes.

Mr Gary Simmons: What particular parameters were used in selecting cases that involved water problems and ones that related to rock problems?

Mr Reijo Riekkola: The data are compilation of all the site investigations and mainly based on the quality of fractures and the number of fractures in the special region, and also the continuation of the special term.

2.3 PASSAGE OF THE MAJOR FRACTURE ZONE NE-1 AT THE ÄSPÖ HARD ROCK LABORATORY

Gunnar Gustafson, Chalmers Institute of Technology, Gothenburg, Sweden
Ingvar Rhén, VBB VIAK, Sweden
Roy Stanfors, Roy Stanfors Consulting AB, Lund, Sweden
Håkan Stille, Royal Institute of Technology, Stockholm, Sweden

2.3.1 INTRODUCTION

In order to test develop methods for passing through major water-bearing zones – especially with regard to location, characterization and controlled pre-grouting – a series of experiments has been performed on the Äspö-Hålö stretch of the access tunnel.

A pilot test was performed in the fracture zone EW-7. The results of this test showed a need for supplementary investigations, especially concerning the NNW trending water-bearing structures running approximately parallel to the tunnel and improvement of the grouting programme which was tested in fracture zone NE-3. In this zone the spread of grout was also investigated.

The fracture zone NE-l, which pre-investigations located at the southern end of Äspö, has during different tests revealed a high transmissivity (possible range $0.8 \cdot 10^{-4}$ to $8 \cdot 10^{-4}$ m^2/s). This zone was considered suitable for an extensive experiment concerning characterization and grouting.

2.3.2 OBJECTIVES

The main objectives of the experiments on passage through water-bearing fracture zones was to develop and evaluate methods for passing through water-bearing fracture zones under controlled conditions as regards the following activities:

- Exact location of the predicted fracture zone.
- Step-by-step characterization of the located fracture zone.
- Controlled pre-grouting – primarily as regards the spread of grout.
- Registration and analysis of the hydraulic pressure responses and changes in groundwater chemistry.
- Assessment of the course of events whenever the tunnel passes through the zone.
- Documentation of achieved sealing effect and spread of the grout.
- Initiation of long-term monitoring of the zone's groundwater pressure, chemistry, etc.

2.3.3 CHARACTERIZATION OF ZONE NE-1

Before NE-1 was entered, the zone was carefully characterized, both from the ground surface and from the tunnel. To complement boreholes drilled during the preinvestigations in the area around Äspö, four new percussion holes were drilled from the ground surface.

The first phase of the NE-1 experiment comprised drilling of two cored boreholes, from niches, approximately 200 m in front of the predicted position of NE-1, in order to locate the fracture zone more exactly. From each niche, two percussionholes were also drilled to detect possible N-S trending water-bearing fractures. The boreholes were investigated by use of spinner and borehole radar.

In a second phase, 8 percussion boreholes (26-40 m) were drilled from two niches, just in front of NE-1, in order to exactly locate the southern boundary of the fracture zone NE-1. Two percussion boreholes were also drilled from each niche to detect possible N-S trending water-bearing fractures.

The flow meter survey in the coreholes and the interference tests in the coreholes and the percussion holes drilled through the south part of NE-1 all indicated a very conductive structure with a transmissivity of around $4 \cdot 10^{-4}$ m^2/s. The responses

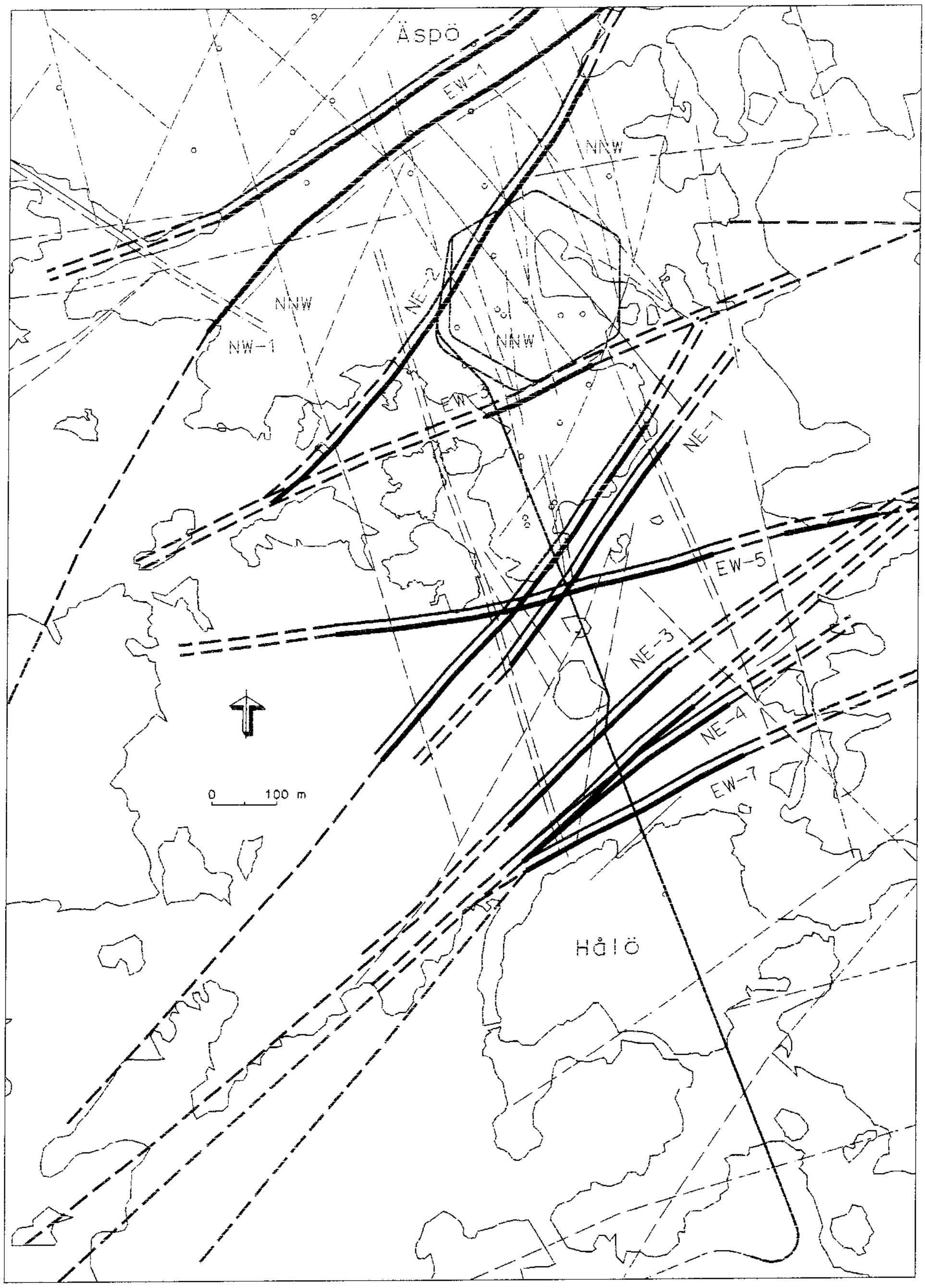

Figure 2.3-1. Plan of the Äspö area.

indicated that the major conductive structure should be NE-1 but it could also be seen that N-S conductive structures were probably in contact with NE-1.

After passage of the zone, NE-1 proved to be highly water-bearing and is assumed to trend N60° E and to dip 70° to the north.

The most intensive part of the zone, which intersects the tunnel at 1/300 m is approximately 5 m wide, highly fractured or crushed and in an approximately 1 m wide partly clay-altered section. The gouge material includes fragments of all sizes from cm scale down to <0.125 mm. The fragments are sharp angled, more or less tectonised granite and mylonite. Older fracture formations are also found as fragments, indicating that the gouge formation is a reactivation of a preexisting zone which developed under ductile, semiductile conditions. Some fragments are penetratively oxidised, probably before the fragmentation took place, and post-fragmentation precipitation of pyrite on the grain surfaces is visible, indicating that reducing conditions prevail. The main clay minerals present in the gouge material are mixed-layer illite/smectite.

The intensive part of the zone with open, centimetre-wide fractures and cavities is surrounded by 10-15 m wide sections of more or less fractured rock. The tunnel intersects the zone along a length of approximately 30 m. The main rock type in the zone is Småland granite with minor inclusions of greenstone and mylonite where the most intensive part of the zone is located in fine-grained granite.

2.3.4 GROUTING ACTIVITIES

Grouting in the access tunnel to the Äspö HRL has to a great extent followed normal practice in Swedish tunnelling. Based on pilot boreholes or inflow through blast-holes or the tunnel front, pre-grouting is performed when considered necessary. Typically, a fan of 10-25 grouting boreholes is drilled, tested by water loss measurements and grouted. The basis for the procedures is a grouting program which defines objectives, rock classification, guidelines and limitations. A special condition for this project is that a limited grout penetration is desired in order to avoid excessive groundwater contamination around the tunnel. High groundwater pressures and highly conductive fracture zones crossing the tunnel make grouting important in the Äspö tunnel.

Grout take and grout penetration depend on several factors. Several attempts have been made to correlate the grout take with the measured water loss from packer tests. All of them have given a very poor correlation. Grout take and penetration from theoretical point of view have been discussed by *Hässler (1991)*.

For a joint orientation at an oblique angle to the tunnel it is important that the holes are reorientated to intersect the conductive joints as much as possible. For this type of joint orientation it is important to have a greater overlap than normal or to seal the front by special methods. Steel fibre shotcrete supported by rock bolts was found to work quite well as such a front sealing method.

At high water inflows it has been found to be expedient to carry out a rough sealing first with a few boreholes before the rest of the fan is drilled and grouted. For NE-1, long boreholes that penetrate the whole water-bearing zone were very effective in sealing the zone. These holes also served the purpose of mapping the zone, which is also important in guiding the tunnel work.

To optimize the grouting work, it is desirable to be able to vary the yield stress of the grout, but also to speed up the hydration of the grout so that the grout take or pumping time can be reduced and the packer removed sooner. The first claim has been

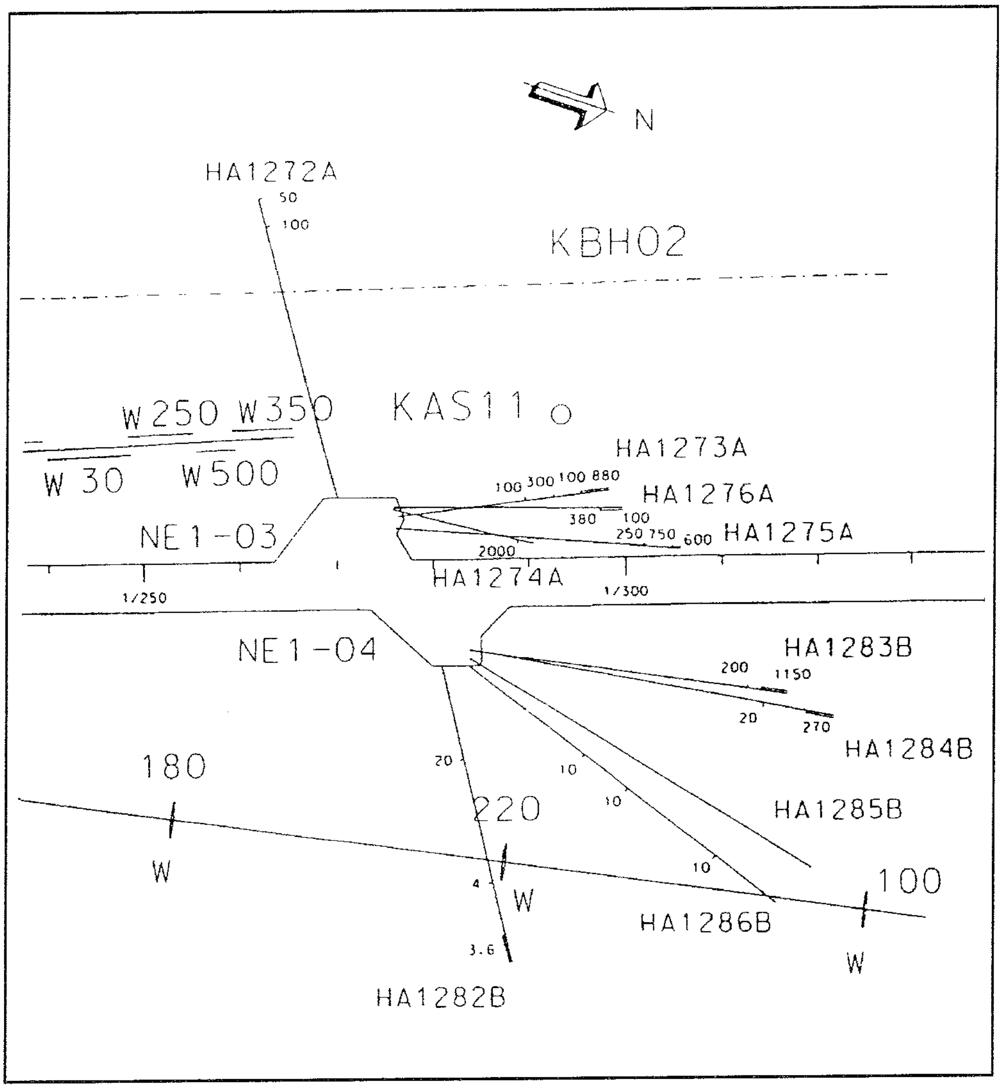

Figure 2.3-2. Positions of the percussion boreholes in niches NE1-03 and NE1-04. The figures on the boreholes are the inflow of water in l/min.

theoretically analyzed with data from the Äspö tunnel and found to be valid. The second claim was practically observed when the water pressure exceeded 1.5 MPa and required development of an accelerated cement grout with calcium chloride.

In order to get refusal and prevent outwash of the grout, the grouting pressure has gradually been increased. From the theoretical point of view it has been found that the grouting pressure must exceed twice the groundwater pressure to get a good refusal and prevent fingering of the grout. This was already confirmed by practical observations.

Some more rarely used grouts like polyurethane (TACCS) and Mauring have been used. These grouts were used in desperation when the sealing work seemed more or less hopeless. It is difficult to evaluate the use of these grouts since they were not systematically applied and tested. They did not, however, have the desired effect and were soon abandoned.

Some fans were grouted with a high water-cement ratio. It was found that this type of grout was not stable and also that no sealing effect was observed.

The flow properties and hardening time of the grouts have been tested both in the laboratory and in the field. The test results have been very important for a better understanding and development of suitable grouts for the Äspö tunnel.

2.3.4.1 The use of "Stabilo Grout"

With a mix of grouting cement, bentonite, plasticizer and silicate, Stabilodur F1 permits suitable flow properties to be obtained for achieving a limited penetration.

For high water pressures, the grout must be further accelerated in order to obtain a suitable strength, allowing the packers to be taken away sooner.

In order to obtain an accelerating effect, Stabilodur F1 must be added in larger amounts than was the case with the earlier grouts ("std, stiff, acc"). Also, the silicate should be added concentrated, not as a solution. With the use of a large amount of silicate, the time taken to achieve 100 kPa can be decreased to 6-10 hours, compared with 10-15 hours when silicate is not used.

The main feature of Stabilodur F1 is its ability to increase the initial shear strength (yield stress) when the grout is in a "fluid" state, see Figure 2.3-4. Although it has an effect on the strength increase when the grout is setting, it cannot be regarded as an accelerator.

2.3.4.2 The use of Calcium Chloride

It is clear from the study that by adding calcium chloride, the setting time can be altered as desired within a relatively wide range, see Figure 2.3-5. However, the grout will be very sensitive to disturbance during the first part of the hardening process. If the grout is disturbed during the initial phase, hardening will be delayed for several hours. The grout will achieve flow properties close to ordinary cement grout during mixing.

It must be noted that negative effects in the use of calcium chloride */Ramachandran, 1984/* include shrinkage of the hardened grout and the introduction of a corrosive environment. In the concrete industry the amount of calcium chloride is limited to 1.5% (of the total weight) in order to avoid corrosion. Also, the effect of a large amount of calcium chloride on the long-term durability of the grout is unknown and, the use of calcium chloride will make the grout less sulphur resistant.

The durability of the grout has been discussed and some tests have started, however, no special demands have been set up. Grout durability is a rather unknown field and more research has to be done. Grout with a high content of calcium chloride must be specially investigated regarding its durability.

Some preliminary classification systems were tested. They were not flexible and practical enough and had to be revised several times, but provided valuable information for development of such a system.

The grouting work has been followed up and analyzed regarding the possibility of predicting grout take and grout penetration. Theoretical models for prediction of grout take and penetration have been developed. The models give such a promising result, however, that further research work has to be carried out to study the different factors which influence the grouting results.

The theoretical models demonstrate quite clearly the very complex situation and the different factors that are of importance. The factors can be divided into three main groups:

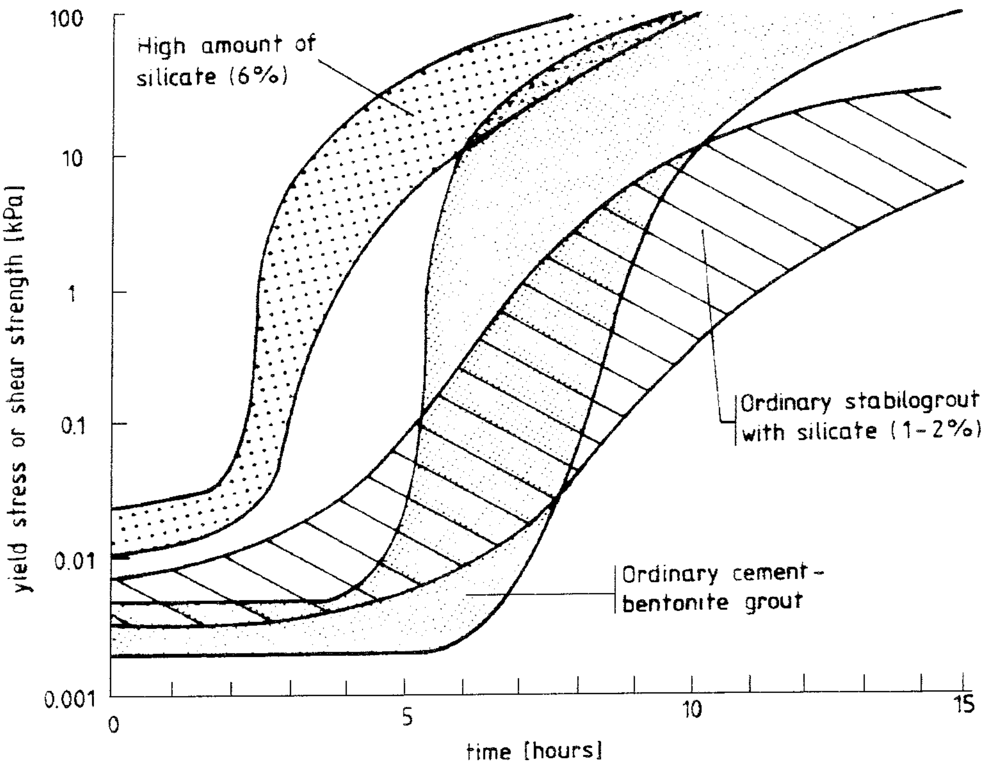

Figure 2.3-3. The effect of silicate.

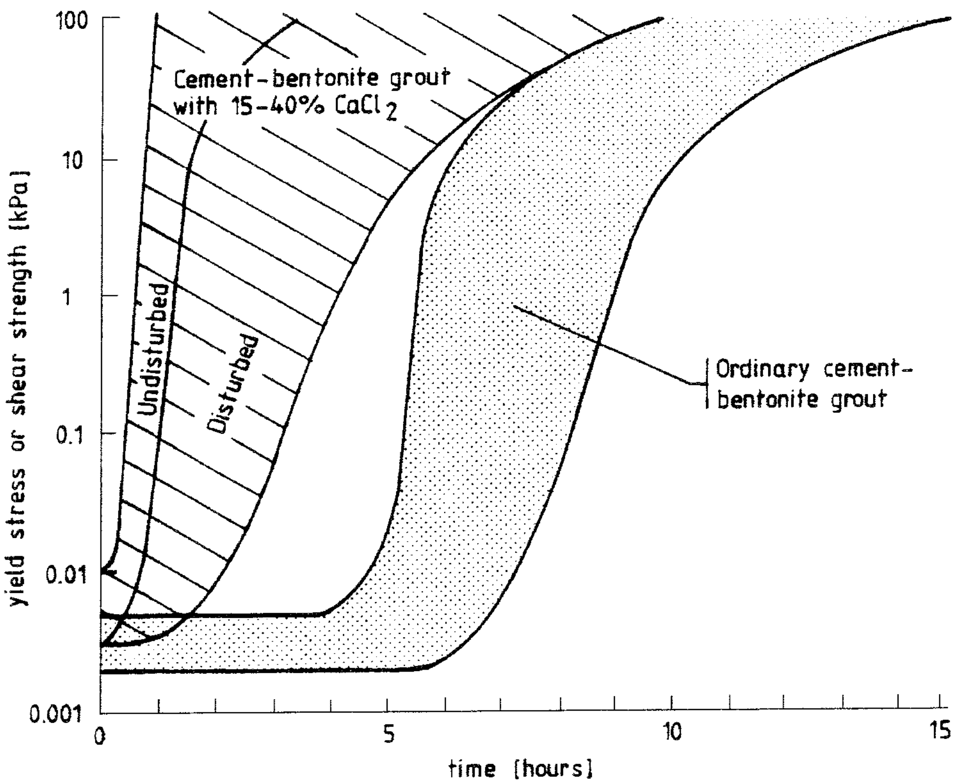

Figure 2.3-4. The effect on calcium chloride.

- grouting factors such as grouting pressure, yield stress and viscosity and grain size of the grout,
- geological factors such as number of joints, joint geometry, joint aperture,
- hydrogeological factors such as rock mass, transmissivity, groundwater pressure, porosity.

Development of a classification system for grouting must be compatible with the theory of grouting. A classification system must thus be divided into three parts that take in account the above-mentioned main factors.

A classification system must also be practical and simple so it can also be used at tunnel front during the grouting work.

The system must give advice about type of grout, grout take and penetration as well as grouting methodology.

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Questions to Mr Gunnar Gustafson and Mr Håkan Stille

Mr Tomas Franzén: It is a well-known experience that a lot of water comes from walldrops and my first question is: is this relevant before grouting or after grouting? The results after grouting also show the tendency, that you get the remaining water coming from the floor. The reasonable point could be that the water outside the tunnel is sinking and you collect it in the floor. There might also be a reason, that then again you have a large difference relevant to that difference. This touches the question of blast damage vis-à-vis other fracturing depending on stress release and so on. And the difference between blasting and TBM boring. This is a typical experience and I think that the figures are quite high.

Mr Gunnar Gustafson: I did some calculations on it some time ago and I am not quite sure that I remember the figures exactly, but if you have a tunnel without any blasting effect, I mean with the same rock all the way and no boundary effects, generally it will come from the floor. What you might add is blasting effects. If you increase the permeability close to the tunnel you do not increase the total leakage to the top, but you have a flow that goes within the open fractures. That gives an increased inflow at the floor if you increase the permeability locally close to the walls by one order of magnitude.

Mr Norbert Krauland: My question is a follow-up to this one. In every fracture zone the influence of blast damage could be very small. In a zone with low conductivity blast damage would be quite substantial. Has any attempt been made to check whether 65% of the water comes from the floor even in fracture zones?

Mr Håkan Stille: No.

Mr M H Raynal: You Mr Håkan Stille made a relation between gravity, the grain size and the jointed opening where you are able to grout. I would like to know some details about the minimum opening you can grout and the geometric characteristics of your cement.

Mr Håkan Stille: Well, the practice in Sweden, based on some testing, is that the minimum opening should be about 3 to 5 times bigger than the maximum grain size of the cement. I have not the grain size distribution with me, but it is ordinary grouting cement.

Question: ? About durability, did you make some laboratory tests, leaking tests for instance?

Mr Göran Bäckblom: We have started duration tests on a long term basis, let's say a few years, on how grouting material behaves in a perspective of 10, 20 or 30 years. It is the same for all types of grout. It is essential for good durability to have a low porosity in the grouting material. I know that Mr Mats Alestam has sometimes discussed that there has been grouting material where the porosity can be up to 50 %. I think it is very important to have grouting material with low porosity, that means you don't have so much flow through the grouting material.

Mrs Maria Onofrei: So it depends on the water content?

Mr Göran Bäckblom: Yes. It should be low like the grout you use.

Mr Håkan Stille: And the water quality? That is also important, not for grouting but for durability.

Mrs Maria Onofrei: You are referring to the phases reacting with the surfacing grout. But in your case I do not see that it is going to be a problem, because you are in a confined system. Usually this reaction is at the surface, due to the fact that it is an increase in volume. It has room to expand. The grout is going to crack. But in a confined system the pressure is going to push your grout against the rocks and give a better binding between the rock and the grout.

Mr Håkan Stille: But still there have been two cases in Sweden with grouting under a dam which has completely disappeared.

Mrs Maria Onofrei: It depends on the quality of the grout, of course, and the composition. With our grout we have seen in fact that the richer the grout water, the better. The phases with which the grout in supersaturated are going to precipitate.

Mr Reijo Riekkola: Could you identify the difficulties that are caused by the high water pressure and are there practical difficulties? Are there any points on the behaviour of the grouting material or setting behaviour depending on the pressure?

Mr Håkan Stille: We have found that you need a grouting pressure that would exceed twice the water pressure in order to avoid problems.

Mr Gunnar Gustafson: Well, one obvious thing is, that if the water pressure is greater than the grouting pressure, the grout begins to move backward when you stop grouting. In the Äspö ramp we found out that this was the case initially. We have also made some calculations on grout front stability and one finds that the grout front is really stable as long as you do not have to consider the mechanical hindrance of the grout front. If you drive grout with water, the front is very instable and you will have "fingering" and other things. That means, to be successful the grouting pressure must be substantially higher than the water pressure. Another thing that can help is of course the grout hardening and also gelling.

Mr Per-Eric Ahlström: What you are saying is, that you must have twice the water pressure?

Mr Gunnar Gustafson: Yes. And that creates another problem, because grouting packers tend to creep out.

Mr Dwayne Chesnut: How do you maintain the pressure during the time it takes to grout?

Mr Gunnar Gustafson: What we considered here was the situation when the grout flow stops. Before that, you may have a rather complicated flow situation. Mr Håkan Stille and I have referred to Mr Lars Hässlers thesis, which deals with this kind of thing. A lot of things happen during injection, but we have found that it is important for us in the "stop-situation".

Mr Dwayne Chesnut: You stop injecting and then what?

Mr Gunnar Gustafson: You inject as long as you can have any flow through the packer, that is good practice. Virtually all successful grouting in Sweden is based on that and I think that we explain that quite well with Mr Hässler's thesis.

Mr Dwayne Chesnut: You place some limit then on the injection process?

Mr Gunnar Gustafson: What we have said is, that we should be below the lowest rock stress. Safely below. That is the only restriction we have had on grouting pressure. But there are equipment limitations that we have to consider and I think that some of the problems were related to that. There is definitely a need for development of equipment for grouting at depth.

Mr Dale Wilder: Have you looked at any further relationship in volume?

Mr Gunnar Gustafson: In our case, if we look at the refusal situation we find that the velocities of the water are very low and, thus we have not really considered the movement of the water. It is an approximation, but it seems to be a reasonable one. What has bothered us more is grouted apertures. We can show that if it varies enough, you will have ungrouted areas within the fracture zone, even if you forget the mechanical restriction.

Mr Göran Bäckblom: I would like to give some supplementary information about the question of how grouting material behaves under high pressures. There have been studies at the Chalmers University of Technology on resin foams like TACSS, and they showed that we did not get any volume expansion if the pressure was higher than 1.9 MPa. That is some kind of limit for using of the resin foam.

Mr Gunnar Gustafson: I think we have experienced the same thing.

Mr Norbert Krauland: Being engaged in reinforcement in mining I would be interested in the question of how to design this support in such a zone.

Mr Håkan Stille: The tunnel was supported with shotcrete about 10 to 15 cm, reinforced with rockbolts in the roof and in the walls. The length was about 4 metres and the distance about 1.5. After some time there were some problems with the bottom, so concrete was cast. Then of course we had the grouted rock somewhere around the tunnel to a depth of about 3 to 10 metres. This short grid is not able to take the water pressure. It must be drained. Otherwise we will have problems with stability, so the idea was to get an arch in the rock around the opening with capacity to take the water pressure. It was presumed that the grout did not increase strength, but of course the grout will increase it, but we didn't know. The only way we have tested it, is the deformation here of the tunnel. It is perhaps a little smaller than we presumed or expected. It can give a rough idea of the improved properties of the grouted rock. Otherwise I do not know how to measure the improvement. You must put it into a testing machine to see the effect. It costs too much, I think. In addition some post-grouting has also been performed at the bottom. We have seen some increased inflow of water. But this post-grouting was not so successful.

Mr Reijo Riekkola: These 200 litres, did they come to the floor through the short grid?

Mr Håkan Stille: Yes, and through the boreholes. The swell exposed it. Then I think it was regrouted.

Mr Gary Simmons: Was there any attempt to try and flush out some of the gouge material in that zone to get a more comprehensive lot in the areas where you were drilling? Did you open the valves? And was 5 m^3 all you could get out?

Mr Roy Stanfors: Of course we could have pushed that out. We only let the material flow out.

Mr Gary Simmons: There was a question based on your experience really, trying to access with your opinion on cleaning things? Will there be any benefit in there?

Mr Håkan Stille: I think to some extent we tried to push out some material but we were not very successful. Sometimes during the drilling a lot of material was coming out but it is difficult to get out the grout material.

Mr Gary Simmons: We have this theory that jet washing can be used to clean infilling material out of fractures to improve the effectiveness of grouting but we have not yet confirmed this practice. We do appear to have had some success in jet washing during our grouting trials at the Underground Research Laboratory. Mrs Maria Onofrei will discuss these trials in her presentation tomorrow.

During the grouting trials jet washing appears to have removed a considerable amount of infilling material in portions of the fracture that were favourably oriented. However, the extent of effective washing is controlled by fracture geometry and the technique does not work around sharp corners.

This is a comment relating to your question about developments that might improve grouting. I am not sure that jet washing would improve grouting effectiveness so I'm asking for comments based on your experience. Would a grout plug in a fracture that had been cleaned by washing be more effective than a plug where the grout is mixed with infilling material?

Mr Göran Bäckblom: It is common practice for dam-foundations to wash out materials. Do you have any Swiss experiences to try to flush out some materials from zones?

Mr Gottlied Eppinger: Yes, but normally it is not interesting for us that the material comes out. Sometime it stops and then we push in grouting material, depending on where we have openings and then we start with very same cement composition and go on as long as the test drillings give a good result and then we go on with the tunnel.

Mr Håkan Stille: I mean, there are different techniques. You can use some kind of jet-grout techniques in order to wash out some material. The problem with the jet-grout techniques is that we will create something like this for our tunnel. In this material we will have a column about 0.4 to 0.8 metre. Here we will have the water pressure quite close to the tunnel. And this is the reason why we want to have penetration grouting in order to shed the water pressure from a long distance from the tunnel surface; several metres, so that the rock absorbs the water pressure and not the inner lining. We need an inner lining with the capacity to take a much higher pressure.

2.4 EXCAVATION THROUGH MAJOR WATER-CONDUCTING FRACTURE ZONES

Gottlieb Eppinger, Elektrowatt Engineering Services Ltd., Zürich, Switzerland

2.4.1 INTRODUCTION

Driving through water-conducting fracture zones – in simplified terms called faults – is one of the most critical tasks of tunnelling. In this brief lecture I will outline a few aspects of this subject. I would like to emphasize that I will present this information from the practical point of view.

2.4.1.1 General remarks

There is a big difference between driving through an expected, predicted fault and an unexpected fault.

In the case of predicted faults, it is a common procedure to approach the fault zone slowly, allowing for a sufficient safety distance. The necessary investigative measures can then be carried out, such as

- drilling for subsurface investigation, or
- seismic methods.

The following findings are of particular interest:

- width of the fault,
- pore pressure, water discharge,
- granulometric conditions of fault material,
- permeability and void ratio.

What are the decisive criteria for determining further procedures on encountering water-conducting faults ?

With unexpected, unpredicted or underestimated faults, serious consequences could occur, like

- collapse of the tunnel front,
- equipment and instruments could be buried under mud and debris causing damage to property,
- danger to personnel, who will have to abandon the tunnel front,
- material from the fault will invade the tunnel, creating dangerous caverns or voids.

All steps should be taken to avoid such events by proper geological investigation.

2.4.1.2 Investigation methods

The most common methods are:

- Detailed geological mapping based on surface exploration.
- Interpretation of aerial photos and development of photo-geology.
- Drilling for subsurface investigation.
- Exploratory drifts.
- Application of geophysical methods.

That means

- Seismic reflection method.This method could be very helpful in roughly identifying fault zones. However, the seismic reflection method is quite inaccurate in case of considerable overburden or steeply sloped faults.

- Tunnel seismic profiling (TSP). (Figure 2.4-1).
  TSP represents a newly developed and promising method to explore faults and disturbed zones from the tunnel front. There is one disadvantage of this method: Faults inclining with a small vertical dip or tapering to the tunnel axis can only be determined inaccurately and with great effort. This method is actually being tested in some drifts in Switzerland. It can be noted that this method has been basically developed by the engineers at NAGRA.

2.4.1.3 Consolidation and excavation of water-conducting faults

In most cases, water-conducting faults with a stable structure could be driven through without encountering major problems after draining and reducing the water pressure. In contrast, fault zones consisting of noncohesive material have to be consolidated prior to excavation.

The following methods are common:

- Grouting or injection.
- Freezing methods.
- Jetting methods.

2.4.2 GROUTING OR INJECTION

The most widespread methods are employed to injection. This procedure is based on improving the natural strength of the material and reducing its permeability by injecting "cementinous" or consolidation material under high pressure into the fault. The following grouting materials are available:

- Cement;
  - Types of cement with a grinding fineness of some 3000 cm^2/g.
  - Appropriate water / cement ratio approx. 0.7.

Adding bentonite stabilizes the suspension and improves flowability. Where voids of large volume are encountered in fault zones, some filling compound such as stone dust, ashes, slag or sand can be added in the first injection stage.

Apart from cement-water grout there are other materials, such as:

- Silicate gels.
- Resins.
- Foams.
- Silica gels (based on water glass).

Usually silica gels do not result in an essential increase of strength. However, they are very effective in reducing permeability to water. Therefore they are mainly applied in the second or third injection phase in order to reduce the permeability of the rock.

- Resins.

Resins are hard-elastic, noncombustible bonding agents. They have a reduced viscosity. Thus, they can penetrate into a non-cohesive ground with a grain size of 0.01 to 0.06 mm even in presence of flowing water. These materials are more expensive than silica gels.

- Foams – a relatively new injection material.

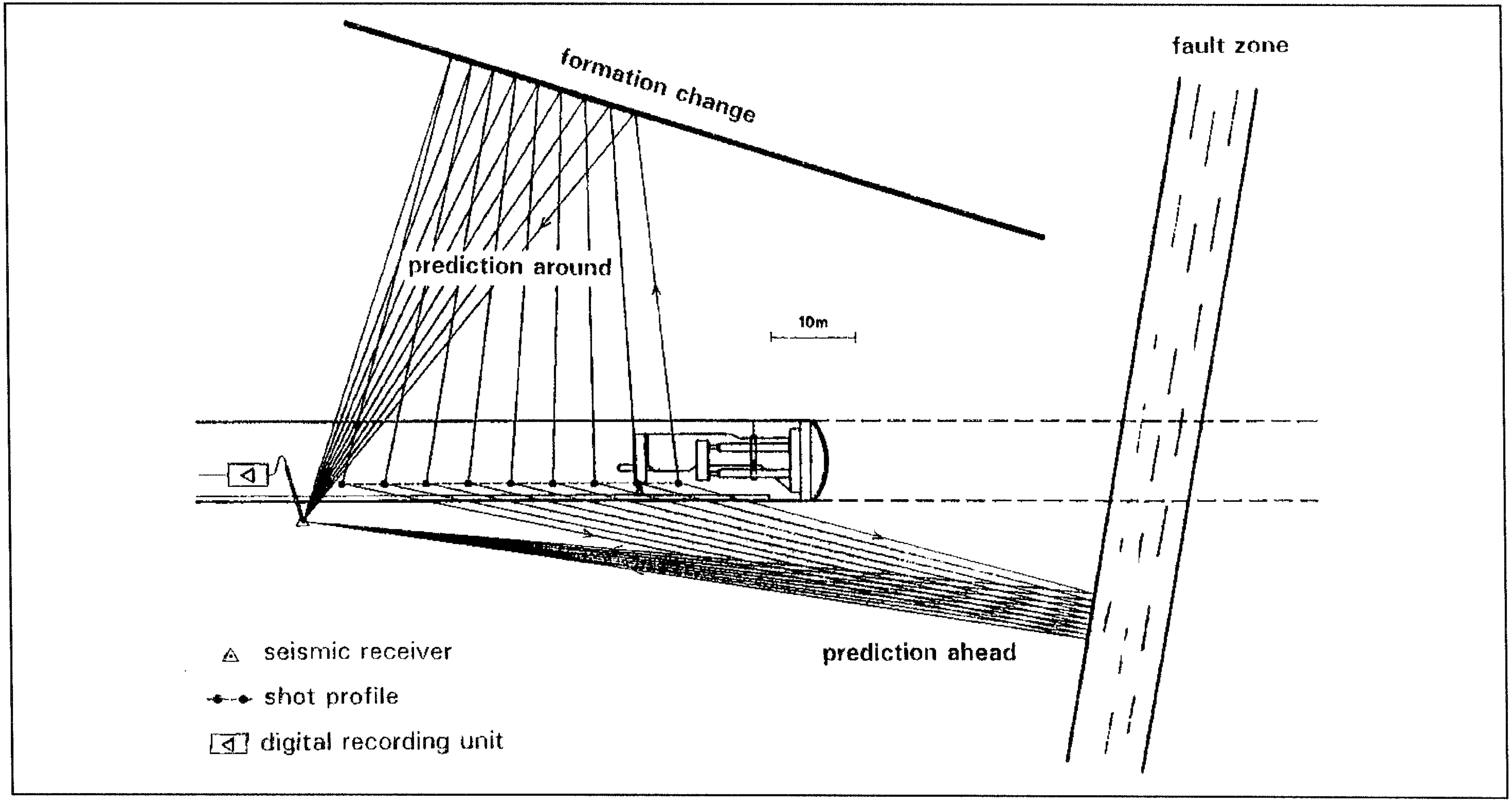


Figure 2.4-1. Measurement principle of seismic prediction ahead of the tunnel face.

A big advantage of polyurethane foams is their short reaction time of some 1 to 5 minutes, with a swelling factor up to 1:30. Foams are often injected into partially emptied or open faults to prevent further subsurface erosion quickly and effectively.

As far as environmental impact is concerned, not all questions have been answered. But it seems that the application of foams has a promising future.

In practice we are always confronted with the following questions: What is the condition of the material in the fault zone?

- Is the material coarse-grained? Is there any presence of clay material?
- Is there some illuviated sand or silt?
- How much pore water pressure is expected?
- What is the velocity of the water flow?

So many questions!

Unfortunately, undisturbed samples from coring do not contribute much to answering such questions. We must therefore proceed by trial and error, starting with a suspension and measuring the absorptive capacity of each injection hole.

In case of a large absorptive capacity the viscosity of the suspension must be decreased, and in case of a minimal absorption increased. The most appropriate mixture can be arrived at in this manner.

In most cases, several injections have to be executed. After each injection, the result and the extent of the injection has to be checked by means of test drillings.

Drilling holes for drainage play an important role. By this means the pore water pressure in the fault can be reduced, facilitating the injection work.

2.4.2.1 Cost comparison

A cost comparison for the basic material results in the following proportions: (Figure 2.4-2)

	Factor
– Cement	1.0
– Bentonite	1.5
– Silicate (soft-hard)	2.0 – 5.0
– Resins (hard-foamed)	50.0 – 55.0

2.4.2.2 Excavation of fracture zone

Usually a consolidated fracture zone is driven through by means of a small exploration tunnel that is subsequently enlarged to the full section. In most cases, the arch in fracture zones is provided with a double lining. For the outer lining, steelribs, lagging plates and shotcrete are used. The outer arch has to comply with static and design requirements in order to bear the full load of the ground. The consolidated fault material has only a temporary supporting function.

The presence of aggressive water affecting the concrete, it is necessary to provide plastic sheets for sealing. The inner lining has to be designed for the entire ground pressure. Due to expected corrosion the static effect of the outer lining can not be taken into consideration.

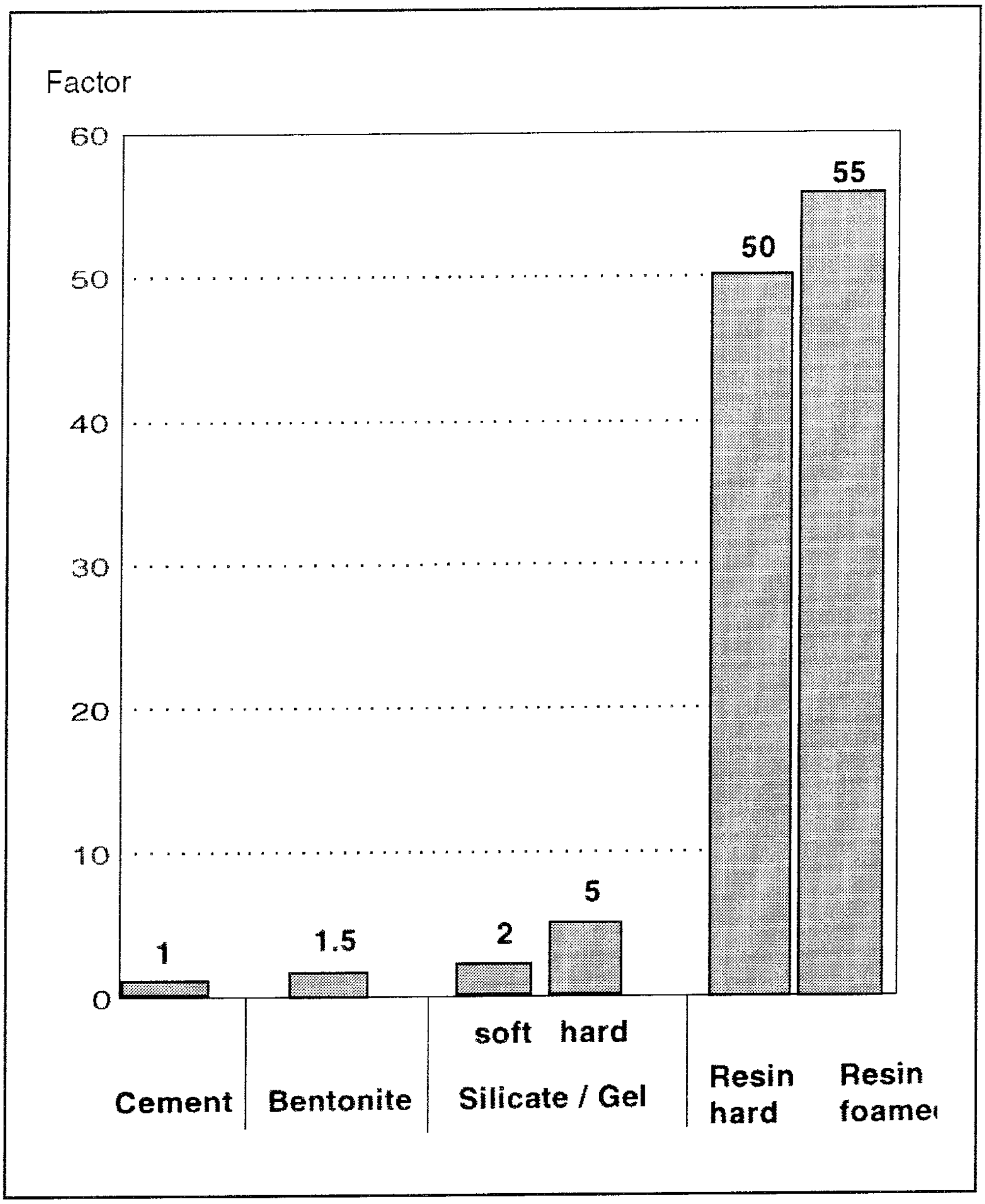


Figure 2.4-2. Basic costs per kg of grouting material.

2.4.2.3 Other consolidation methods

– Freezing.

Freezing is an alternative to injection. Depending on the size of the drift, the full section or only an arch of 2 – 4 m is frozen. The application of this method requires special installations and takes a lot of time, making it quite expensive. The freezing method is only to be recommended when the injection method is not applicable. In disturbed zones the freezing method is rarely used.

– Jetting.

With this procedure some 15 to 20 m long jetpillars are drilled forming a sort of a cone or shield around the excavation profile. The overlapping of the jetpillars is about 3 m, resulting in excavation stages of some 12 – 17 m. The working face at the end of a specific section also has to be stabilized.

This method is less time consuming than the freezing method and is slightly cheaper.

2.4.3 SUMMARY

As the outlined examples show, not only different injection materials but also different construction methods are available for the consolidation of water-conducting faults. Within the scope of this brief presentation it is impossible to enter into details of mixture composition, specifically in connection with injection materials, due to their dependence on different factors and the particular conditions of each site. The same is true for the complex chemical composition of silica gels and resins. This subject is a special field. When a fault is encountered, good collaboration is necessary between:

- Geologists.
- Design engineers.
- Contractor.
- Construction supervisors.
- Injection and grouting specialists.

Furthermore, it is desirable to solve the practical problems directly at site and not in an office on the 15th floor of a tower. An optimized work procedure with regard to cost and time can only be achieved knowing the local conditions and the latest findings from the ongoing work. In this way, the important aspects of cost, time and safety can be taken into consideration.

2.4.3.1 An example

At the end of my presentation I will give you an example from reality. It happened some years ago, but it nevertheless represents a classic case of a fault that was misjudged.

It involves a gallery in a hydro-power station in Switzerland with a diameter of some 6 m.

A fault at chainage 306: In the first 200 m of an inclined gallery a small fault was driven through with a full section. The ingress of water at these faults was 100 to 200 l/s each. From one of the faults some 100 to 200 m^3 of fines intruded into the gallery without causing major damages.

At chainage 306 (Figure 2.4-3a) the same signs appeared, indicating another harmless fault like before. However, this fault proved to be much bigger and more serious. On November 16, the first cave-in of 1 300 m^3 occurred and 9 days later the second cave-in with another 1 500 m^3 followed. The inflowing quantity of water from the fault rapidly increased to 180 l/s.

After constructing a concrete plug (Figure 2.4-3b) some 33 m behind the face the engineers tried to drive through the fault by means of a pilot heading without consolidation. At the end of January, when this pilot heading reached the fault, a third cut occurred with a quantity of 1 500 m^3 of material. Meanwhile the water flow decreased to 130 l/s, but with this new incident it increased again to 180 l/s. The

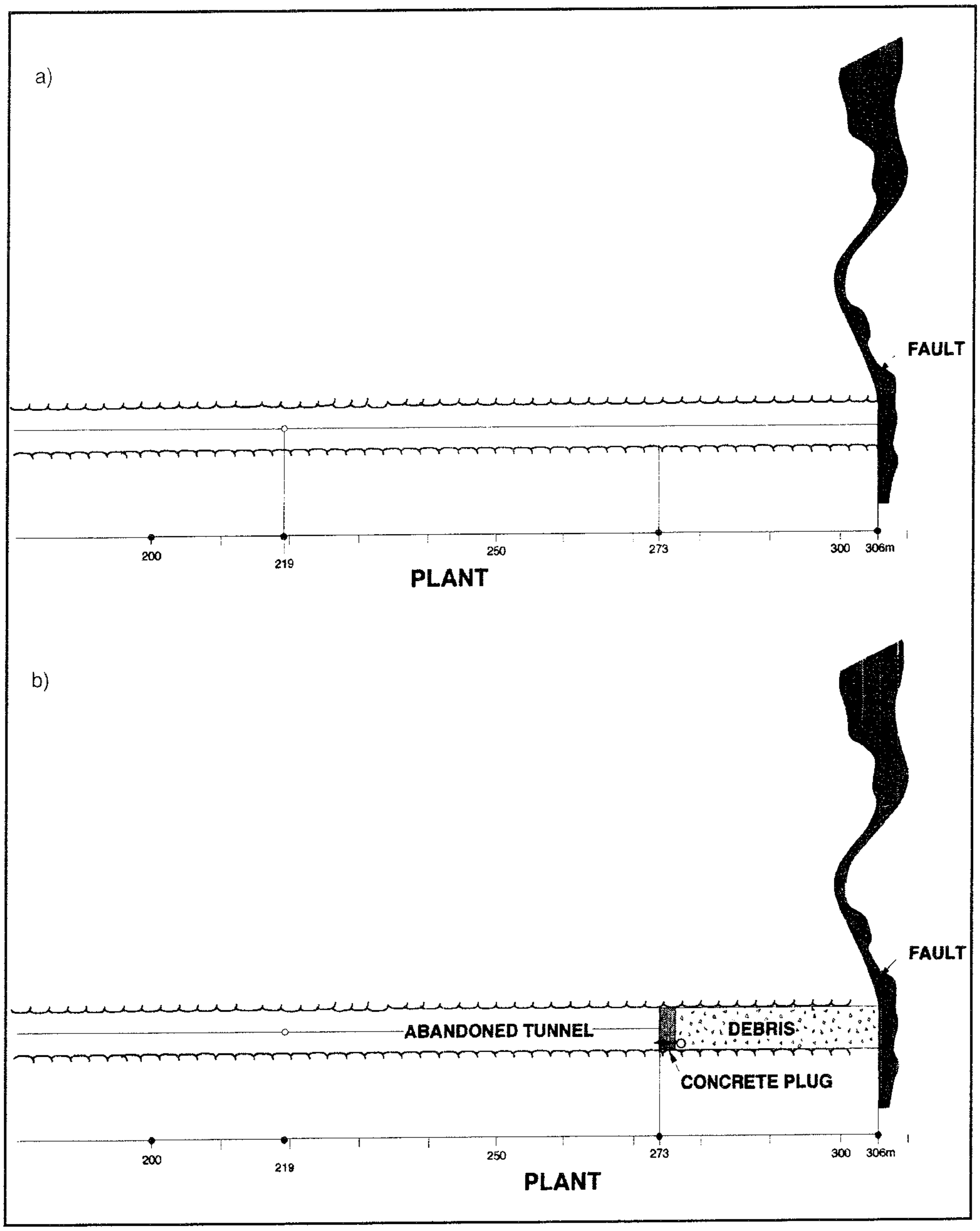


Figure 2.4-3a. *Normal tunnel excavation without knowledge of the incoming fault.*

Figure 2.4-3b. *Complete the plug and abandon the main tunnel.*

drillings for ground investigations showed quite an irregular width of the fault between 1 and 3 m. The decision was taken to bypass the fault with a new alignment.

2.4.3.2 Bypass (Figure 2.4-4a, b, c)

In the area of the fault the axis has been shifted by 30 m. In order to prevent injection from interference with the previously excavated gallery, a minimum distance of 30 m was chosen. The bypassing gallery approached the fault up to a distance of 6 m. Drillings into the fault zone showed a water pressure of 25 bar and a loose fill in the fault. The first phase of cement grouting thereby commenced.

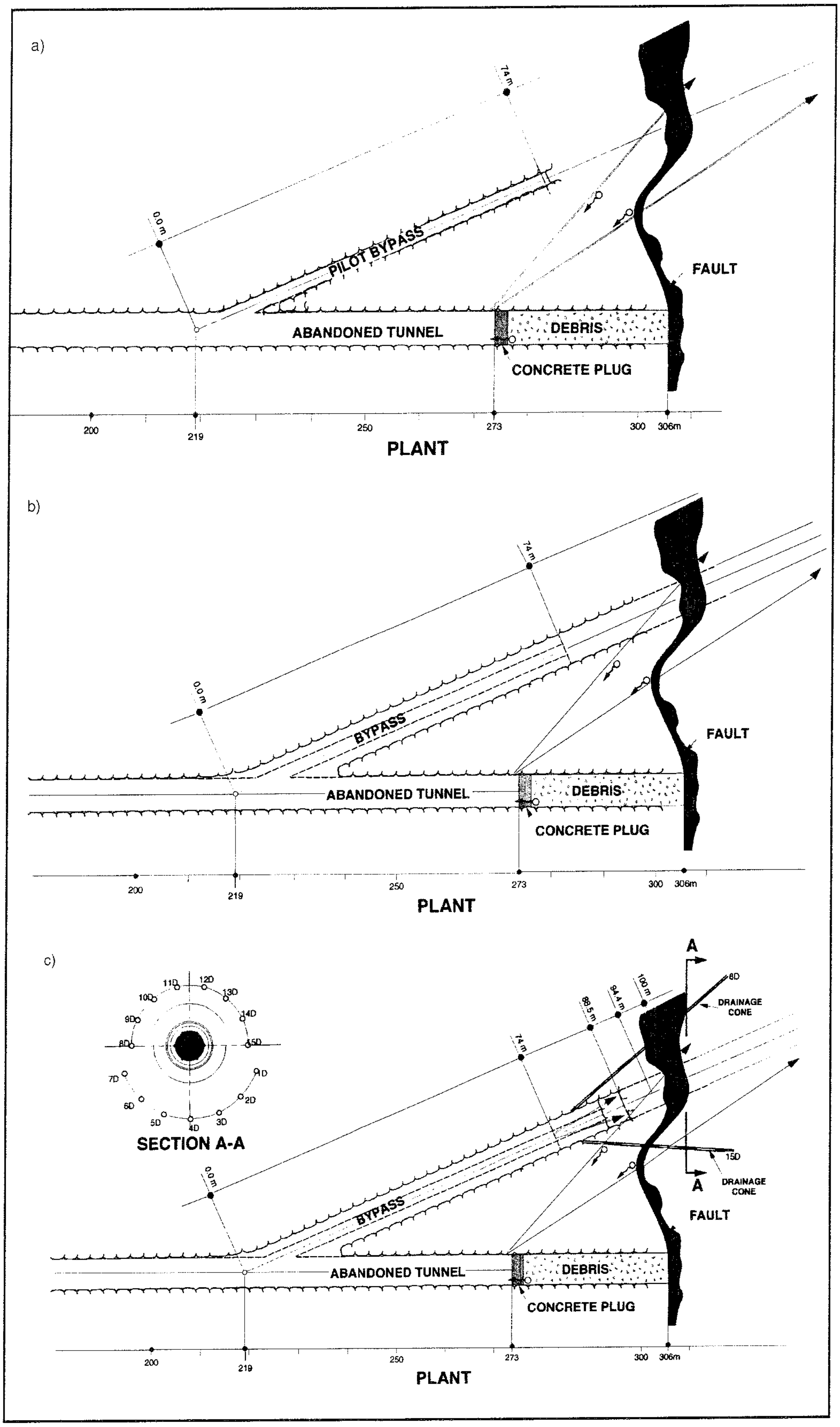


Figure 2.4-4a. *Start of the pilot bypass tunnel.*
Figure 24-4b. *Enlargement of the bypass tunnel.*
Figure 2.4-4c. *Complete drainage cone.*

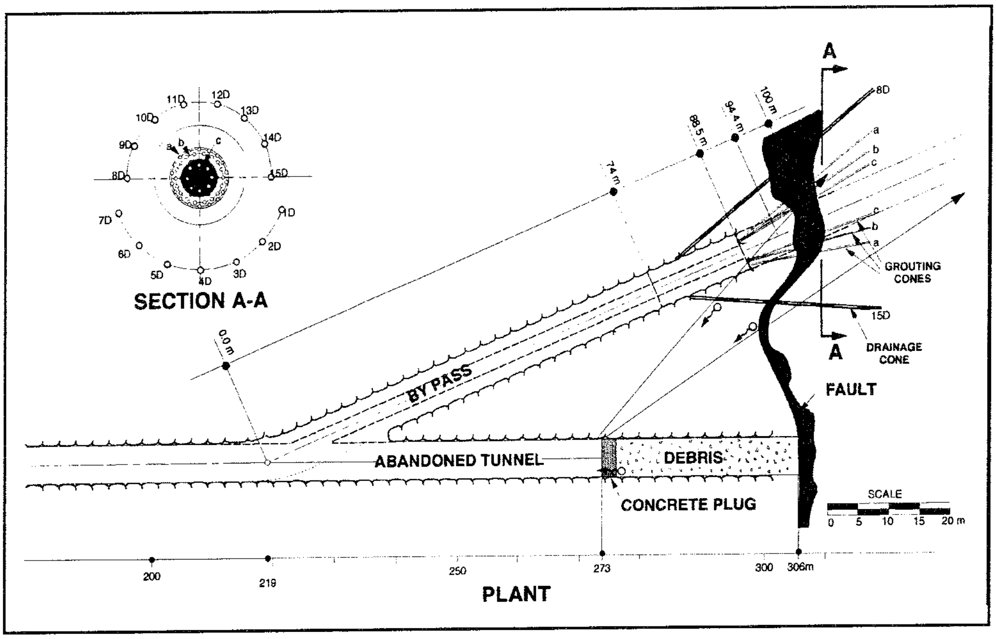


Figure 2.4-5. Complete first three cones of grouting drill holes.

2.4.3.3 Treatment of the fault (Figure 2.4-5)

In the course of this first phase 42 drillings of 10 m length each were executed and some 24 tons of grout injected with a pressure of 100 bar. Moreover, 9 drillings for drainage were performed. The work for this first phase lasted 2 weeks.

2.4.3.4 Final phase

In the exploratory gallery it was ascertained that the cement injection into the undisturbed fault had a minor effect only. The material was slightly consolidated but was not sealed. Hence, the water circulation in the ground brought about by the excavation destroyed the grouting cone. Another cut of 200 m^3 occurred in the pilot heading. Fortunately it could be closed with a great effort by concreting a plug (Figure 2.4-6a, b). Under these circumstances an extensive injection and drainage program had to be implemented.

The 9 cased drainage (Figure 2.4-6c) drill holes outside of the section to be consolidated reduced the water pressure from 25 to 5 bar. An arrangement of 8 concentric cones was emplaced (Figure 2.4-6d). Thus, the consolidated ground mass reached a diameter of some 18 m, representing three times the diameter of the planned excavation. During phase one and two of this campaign more than 100 drillings with a length of 2200 m were executed. As soon as a drill hole showed the intrusion of unstable material or water the injection procedure started. Each drilling was treated in this way until the desired length was achieved without influxes. Some drill holes had to be reamed out and injected again up to seven times. A total of 210 tons of cement grout was injected with a pressure of 110 bar and more than 4300 drill holes were reamed out.

Test drill holes with a diameter of 200 mm proved that the material could have been consolidated and stabilized. However, the sealing effect was insufficient. There was still an infiltration of water of about 1 to 2 l/s for each test drilling. By whatever means, infiltration had to be avoided for a further continuation of the work.

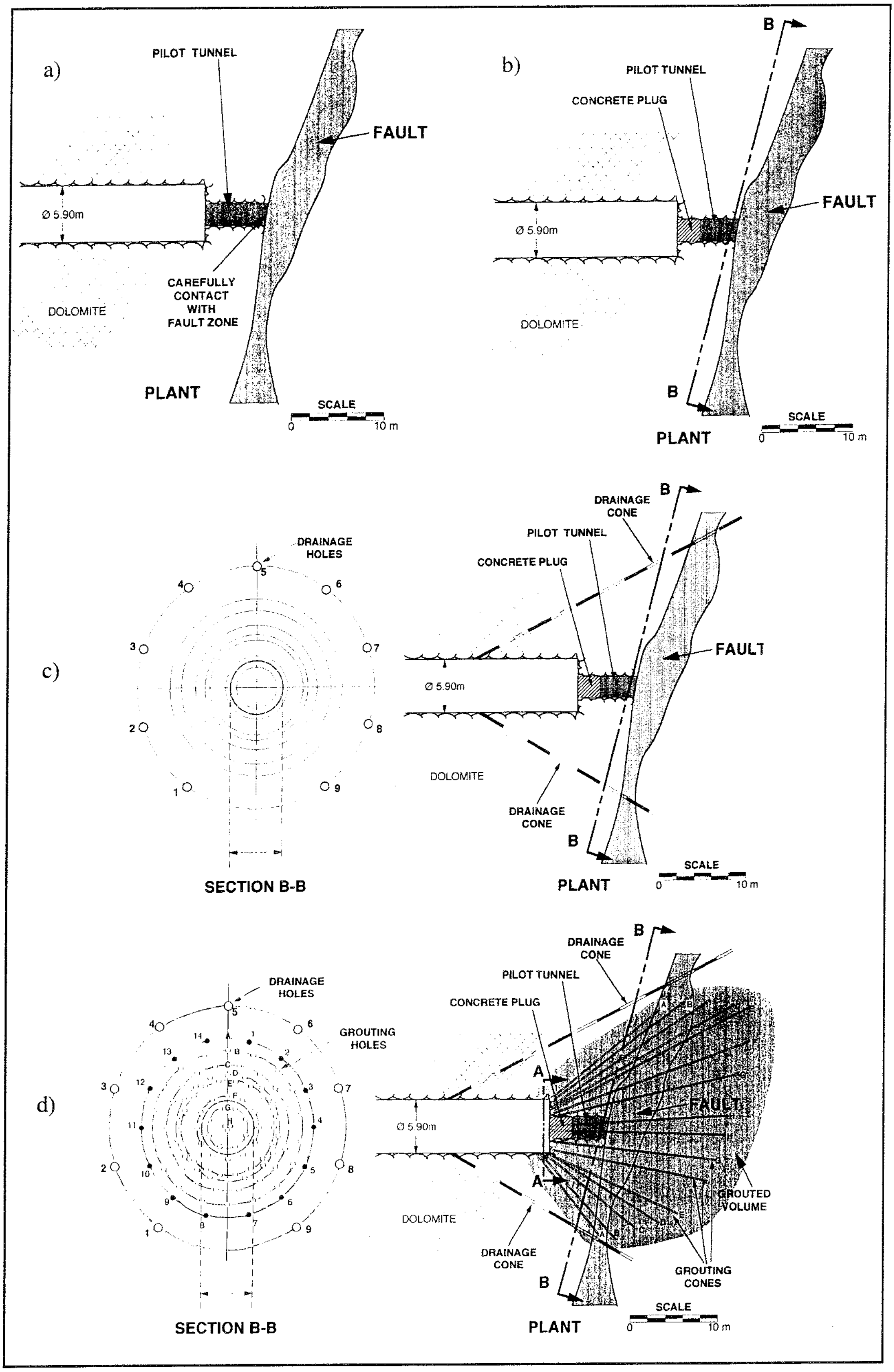


Figure 2.4-6a. *Final phase. First contact with fault zone.*
Figure 2.4-6b. *Final phase. Installation of concrete plug.*
Figure 2.4-6c. *Final phase. Installation of drainage cone.*
Figure 2.4-6d. *Final phase. Complete grouting volume.*

For the third injection campaign silicate injections had been chosen. The grouting material consisted of 700 l of water, 300 l of silicate and about 18 kg of sodium aluminate. With a water temperature of 6° Celsius, a setting time of 45 minutes resulted. With a total of 66 drillings, reaching a length of 1430 m and placed around 5 cones, 70 m^3 of gel were injected into the consolidated fault with a pressure of 40 to 65 bar.

Another 15 drainage holes were drilled. The result of this third phase was a complete success. Subsequently the gallery could be excavated without encountering any water infiltration.

2.4.3.5 Driving through the fault

The gallery was driven through by means of a top heading (Figure 2.4-7). The "rock", – i.e. the consolidated zone – was protected with a steel arch support and the immediate application of shotcrete lining and an external concrete ring of 25 cm. The face of the gallery did not have to be supported anymore.

The time required for the execution of all work was 13 months.

2.4.3.6 Technical data

– drill holes for injection (some of them had been reamed several times)	450
– diameter	42 to 45 mm
– total length of drill holes	> 6000 m
– drainage drill holes	35
– diameter	65 mm
– total length of drill holes	275 m

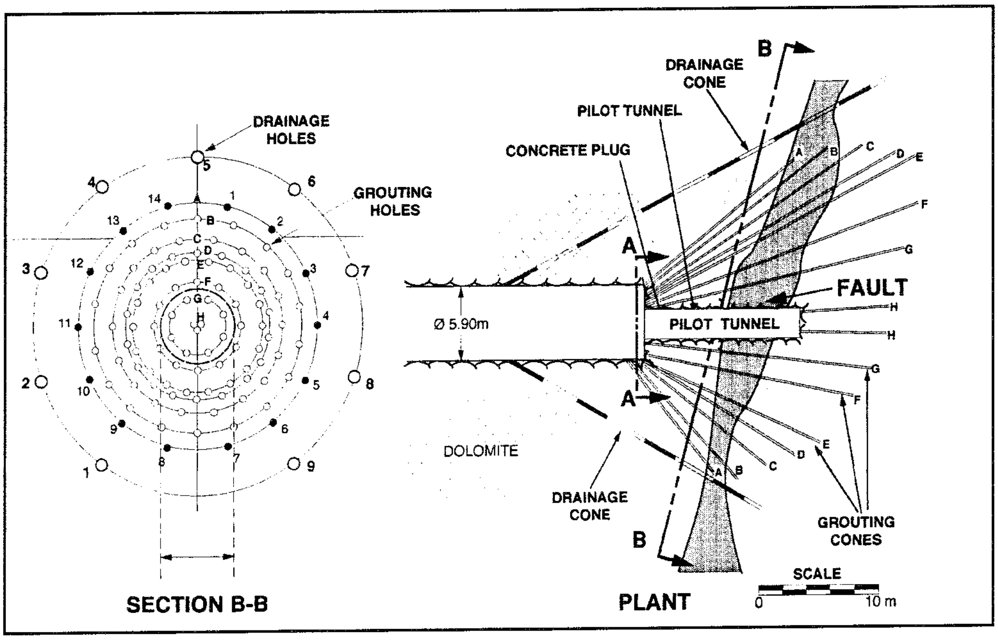


Figure 2.4-7. Final phase. Removal of plug and boring through fault.

- consumption
  - grouting material:

	cement	400 tons
	silicate	70 m^3

- injection pressure — 40 to 110 bar
- test drill holes, diameter — 200 mm

2.4.4 CONCLUSIONS

During construction of a total of 30 km of galleries for this hydropower scheme, some 30 to 40 water-conducting faults were encountered. Most of them were driven through by full- or partial-face tunnelling without consolidation.

With some particular faults a few problems arose. I deliberately described an extreme case of a fault zone where considerable problems occurred. Thus, the technical consequences and the effect on the work schedule was demonstrated.

Fortunately, grouting technology has made good progress in the last few years. Though the time required for consolidation measures could have been reduced.

However, it is still a question of essential interest for the engineer when and where a fault will be encountered and how extensive such a disturbed zone is. The availability and development of reliable prediction methods in tunnelling are of significant importance.

Notwithstanding, a certain risk will always be associated with construction of underground structures. Perhaps, this kind of risk contributes to the fascination of tunnelling!

Thank you for your kind attention.

Questions to Mr Gottlieb Eppinger

Mr Göran Bäckblom: You said in your presentation that you used very high grouting pressure, up to 110 bar. Do you have any problems setting packers?

Mr Gottlieb Eppinger: No.

Mr Håkan Stille: Why did you use such a high pressure? 110 bar.

Mr Gottlieb Eppinger: The water pressure which I used was in an open system. If you close the pipe, the water pressure will go up to about 30 – 40 bar.

Mr Håkan Stille: What was the conductivity of the zone?

Mr Gottlieb Eppinger: We do not know. We always start with a suspension measuring the quality and if it is too much, we change it.

2.5 DISCUSSION PART I

2.5.1 LAYOUT AND DESIGN

Mr Göran Bäckblom: We heard this morning Mr Paul Thompson say: "An ounce of prevention is worth a pound of cure". That is a good expression and that is maybe what it is all about. What can be done based on pre-investigations and what can be done during construction. I get the impression from Mr Roy Stanfors that you think that you can find the major fracture zones from the surface but maybe not all the minor fracture zones. How can that effect the layout and the design?

Mr Jukka-Pekka Salo: We already have some layouts but we have reorganised. They must go hand in hand, these site investigations and also the layout designs and other designs. If it is an iterative process, you have a basic layout and you have pictures from site investigations of the fractures. Then you can see where the most important features are and you can calculate the water-flow. Then you can investigate the most important places and then you can do a proper iteration. I think this layout should be taken into account in an early phase.

Mr Andy Nold: We are not yet in such a comfortable position as to have a final repository as the SKB have for low and intermediate level waste but nevertheless we are trying and working with first priority on the realisation of such a repository. We have now finished the first investigation phase, which contains about five wells with depth up to 1900 metres and we would like to go for the next step which consists of an exploration tunnel. All together: exploration tunnel, layout, layout design, safety design. I will give some information on this point. In a plan view we have the valley, here it goes up and the horizontal section shows a situation as follows: this is the formation and we have a hill, a so called "Wallenberg" which has a ridge about horizontal in this direction and then going up steep to higher mountains. Here is another small valley. This being the formation, we know from surface investigations so far on geology, hydrogeology and geophysics at these five wells, that major fracture zones are the really big ones. I'll just draw something to explain the idea. So, we also know that the fracture zones should be something like this, because the main stress-field is somehow like this. It is expected that we have no major fracture zones going on there. The project looks as follows: We intend to go in horizontally, more or less slightly 5 $^{0}/_{00}$ up, not having problems with water evacuation, and then we have a cavern system. Now the idea is to have here at the entrance facilities at the surface and we intend to realise something like this. Having the rock here, something like that. It was mentioned that these big zones may not give some problems. It may have some intermediate ones, which could cause some problems and this is really now the point. Our philosophy or strategy can be summarized as design-as-you-go. We know some major fracture zones and we know that we can exclude fracture zones by this. What we will do is draw an exploration concept and then look whether we find any other features. Then we come to the point we discussed before. Going along "design-as-you-go" means, – of course you need something to start with, so you know where to go – that you end up at a point. First you should know what to avoid, what type of zones you should avoid. And then you should have the tool to decide and to measure on site and to say if it is decisive or not. I think this is really the key question, because you may end up in a difficult situation if you say, that in case of not being sure, let us assume that it is a decisive fracture zone. If you do this too many times, I do not know with what pattern you may end up with. You have no possibility any more to really design a repository. May we summarize all we know in the following point. We know where to start, we have an idea how to go along and then the next point is to avoid major fracture zones. To know how to avoid them, you have to have some characteristics and you should have the means to really determine on site this one or not. This was just a short thing about repository design.

Mr Christian Sprecher: About designing and exploration concept, just for the case that has just been illustrated. I am not quite as optimistic as Mr Andy Nold is, but we all know of the positioning of all, I will call them layout-determining features. We have a number of vertical bore-holes in this area, but we expect the layout-determining fracture zones to be vertical so the chances are small that, if they exist, we would have hit them. With all indications we have from geological surface observations, which we more or less extrapolated. That is about the reliability of this information. Then we are going to realize the first stretch of our exploration tunnel. Then indeed one of the questions is: when we hit – and we probably will – a fracture zone, how can we determine whether the fracture zone we have hit is layout-determining or not? So, one of the questions I have for this auditorium is this. Does anybody have a good idea of how one could set up a list of criteria to decide during tunnel construction if a certain zone fulfils the list of criteria that we call a layout-determining zone and can we construct our repository cavern around them or do we have to keep a certain distance?

We are going to construct our repository caverns around zones which fulfil the list of criteria that we call layout-determining zone, or keep our repository caverns at a certain distance from such major features. These layout-determining features are a major exploration target for our next phase of investigations. If you set up an exploration target you should have a bit better idea of it than just vague things, like "it is big" or "it has a certain extent". You have to be more concrete. Maybe ultimately the performance accessment people have to answer this question about the layout-determining features. But even if they do that, you have to have the means so that during the tunnel construction you can quickly decide.

Mr Per-Eric Ahlström: I am sure that Mr Peter Zuidema and others who are doing safety assessments will say that there is not a clear cut answer from a safety assessment point of view. I think it is an interaction between design and safety assessment. Maybe that was what you were explaining. You need a dialogue. In some cases, even if safety considerations would allow you to be very close to a water-conducting zone, the people in place do not want to get drowned. There are other complications too. There is too much water coming in during the operational phase, and if you seal it off you may destroy your chemistry with too much concrete.

Mr Brendan Breen: If you are working in competent rock, you can normally detect major fracture zones by geophysical methods. They become obvious. When you look at that in three dimensions your opportunity for developing and avoiding major fracture zones are well proven, but standard techniques employed in most mining operations. With minor zones again, you do get early indications which have got to balance. The early indications can be signs of dampness, or there can be geophysical conductivity indications, that suggest that you have some water, which is often the first sign of a problematic minor fracture zone. In those situations you have at least an early indication that allows you, as was suggested, to turn away from the problem and design-as-you-go or abandon a site, until you have done further evaluation or, if you wish, to move away from the zone. So I think that, provided that your approaches are clear in relation to your development work, you should be able to cope with these issues and problems. What is important is that you consider in your design exactly what you can tolerate in relation to underground conditions before you establish a repository.

Mr Göran Bäckblom: I get an understanding from this, that you all are in favour of design-as-you-go.

Mr Norbert Krauland: I would like to address a question. I was a bit surprised this morning when I saw a picture of the tunnel between 20 m level and 200 m level. I see there very clear signs of pressure. I am not familiar with the problems of repositories but as far as I can understand you would not allow these zones around the storage holes where you want to have you canisters. The question is what kind of stresses we have near fracture zones. It is a common experience from mining that we have increased stresses nearby in these blocks. You might have very high stresses that might change within very short distances. So I think one should really have a close look at this in the design. We have experience from Scandinavia, from old mines, that in some places there are very high stresses in the rock at a depth of 50 to 100 metres. The changes can be very abrupt, within 100 metres it can change completely.

In the Äspö case we have a failure depth of 220 metres. In the design I have seen here, you are looking for 500 metres depth. It could be quite difficult to predict the stresses. This brings up the question of design-as-you-go. I think it is a necessity.

Mr Christer Svemar: About the layout I would like to add the options of ramps contra shafts when we are going down to the repository level. In Sweden we do have a very similar approach to the blocking as Mr Arnold Nold explained from the Swiss project. That is to try to characterize major fracture zones and leave a certain rock volume closest to them as a buffer zone. With a set of regularly repeated fractures there will be several blocks within which we assume there are only minor fractures, which we can accept. But one thing is whether the fractures in the blocks are vertical or sub-horizontal/horizontal. As Mr Göran Bäckblom explained we have seen one advantage from a long-term safety point of view, which is to have a major horizontal water-carring fracture zone above the repository. The question is how to pass such a zone. If you would like to make the problem as easy as possible, you would like to cross it perpendicularly. You do not want to cross it along its extension. But will we really have those major fracture zones? Will they be standing vertically and without any sub-horizontal or horizontal fractures? Or can we expect to have them horizontally with a similar pattern or some kind of pattern like the vertical fractures. On the Swedish sites representing different types of geological environments, drillings have indicated the existence of horizontal and sub-horizontal fractures. If we have to cross one of those we would like to do it as close to the surface as possible. In that case a ramp could be a suitable solution, as it can run beneath the fracture zone to the repository depth after the crossing. If it is located closer to the repository, a vertical shaft might be more appropriate. Still you would get the perpendicular crossing. So, from a layout point of view, we should not look only at the ramp solution, as long as we do not know the features of the site.

Mr Jukka-Pekka Salo: But it is also a question of the waterflow on the site.

Mr Christer Svemar: True. That is one feature determining the layout. If there is a major highly transmissive sub-horizontal fracture zone and a proper location of the repository in relation to this zone really would improve the repository function from a long term safety point of view, we should try to adjust the layout.

Mr Brendan Breen: The reference to the rocks and the problems of stressing is making the point, that it is the ratio of principal stress and minor stress that can be the consequence of failure, rather than necessarily one of extreme stress conditions.

Mr Norbert Krauland: The difficulty is to predict failure. Is it possible to predict failure? I do not think so. No one has so far been able to predict what kind of failures and stresses we have at a certain point.

Mr Gary Simmons: I think that there has been some progress made in laboratory studies on the strength and failure mechanisms in intact rock that is not influenced by fractures. Progress is also being made on estimating the extent of failures that occur in field tests. We have experience on the 420 Level of the URL where we are working in a 4:1 stress ratio across some excavated openings. We are finding that the failure mechanism is microcracking that results in rock spalling off the surface of the excavations. In the remaining rock of the excavation boundary there is no zone in which there is any significant fracturing that is hydraulically interesting. In this case it is reasonable to suggest that there are ways of engineering around the effects of high stresses and that it is possible to place a geological repository in this kind of environment.

Mr Göran Bäckblom: There is a proposal that we should discuss: excavation methodology through the major fracture zones and how to really cope with them. What are your views on blasting and mechanical mining, Brendan?

Mr Brendan Breen: As far as I have used the excavation methodologies, I think that principally economics drive the method of excavation. Much comparison has been made between the machine-mining and drill-and-blast. I do not really think that there are any significant differences. Blasting does give you more near-zone damage, but again it depends on the final extent of the damages as to whether or not you have any real concern. Certainly it is one of the most tested techniques that we have available and there are techniques of smooth-blasting as well as secondary blasting to give smoother walls. There are lots of techniques that allow you to go wherever you like in relation to excavation and I actually believe that the driving force is not the excavation methods. Economics are the key factors.

Mr Paul Thompson: A comment on the presentations of papers written about the advantages of for example mechanical excavation. I agree with what Mr Brendan Breen said, that economics plays a major role, but in our experience drill-and-blast can be achieved not only with minimum excavation damage, but it also has the advantage of allowing easy access and characterization and you can do a fairly economic characterization program as you advance in the tunnel. This is not achievable for example with TBM. You can obtain better information on convergence, you can obtain better information at the face on the geological conditions than you can with a machine excavation. I think that should be recognized.

Mr Andy Nold: I think that so much has been obtained about the conditions you have to fulfil and also about observations for information and so on. You can always find ways to design a parallel tunnel and drill from there and make measurements. We have come to the conclusion that tunnelling by TBM-excavation or blasting depends on many factors and you have to have an individual indication of what is good and optimum for one over the other. From a technical point of view we came to the point that drill-and-blast as well as TBM can be used in hard rock. It also depends on the sections and limitations. With TBM you cannot excavate large caverns, that is clear. Both possibilities depend on the conditions.

Mr Per-Eric Ahlström: You can avoid major fracture zones, if you have to pass them for some reason. What are then the implications of having a tunnel boring machine versus drill and blasting?

Mr Andy Nold: I started from the idea that disposal tunnels should not be placed in major fracture zones. Tunnelling machines are not made for such use; then you find yourself in trouble. This must be avoided.

Mr Håkan Stille: But in order to take really advantage of the design-as-you-go method you must have a flexible excavation method. Especially when you are going through a major fracture zone. And I think there is a great advantage in drill and blast compared to boring.

Mr Brendan Breen: I agree. There are advantages in both methods. If you are going for long distances and you know where you are going and you do not have to stop or be interrupted, then there is a lot of value in machine mining. There is no doubt about it. With machine mining you reduce the near zone damage and it can in certain situations save you on support costs. But taking that aside, I think that blasting techniques are quite sophisticated. They can be very controlled. If, as we may or may not have, subject to economic program pressures or whatever the issue, time to do the work like secondary removal or secondary zoning, in the case of large excavations it is normal principle of blasting to use secondary blasting. In those situations it can be fairly gentle with the surrounding rock. So you can do a lot of things to control what you do.

Question: The title of this session is "Need for Development of Technology". Maybe there is a need for improvement of technology in the field of TBM in order to make it more flexible and to make it competitive with drill and blasting.

Mr Brendan Breen: In Australia, one mine has just started the development of a machine that can excavate different tunnel shapes, and thus has this flexibility. It is called the Mobile Miner, MM130.

Mr Göran Bäckblom: It would be interesting to discuss how we can handle differential high water pressure.

Mr Gunnar Gustafson: To start with, there might be a limit to how much differential water pressure would be permissible because of groundwater chemistry. The basic idea behind what we do, is that we have an aerobic hydro-chemistry environment. Right now there is a redox experiment going on at Äspö and we do not know the results of it yet, but if we have a repository open for a very long period, which we have to anticipate, and if we draw down water oxygen we will have a change of environment. All your work may be in vain, because a lot of the safety is based on stable water chemistry. That means, that you should be very cautious with draw-downs, even if construction would be simpler.

Mr Per-Eric Ahlström: But what about the mines. Do you really get oxygenated water in mines that are 500 m deep?

Mr Gunnar Gustafson: Yes, I have seen that in Kiruna. That is an extreme situation, when you have an inflow of about 200 liters/sec. going on for 25 years almost. It was not that bad, but they moved the water down as they took out the ore.

Mr Per-Eric Ahlström: What was the situation at Stripa?

Mr Gunnar Gustafson: Stripa is a small thing.

Mr Christer Svemar: In Kristineberg, a sulfide ore body, there is oxygen in the water, because the pH in the mine water is as low as 2.

Mr Norbert Krauland: It depends on which way the water comes in. It is not quite fair to compare with a mine.

Mr Göran Bäckblom: I just want to check with Mr Gunnar Gustafson. Maybe we can distinguish between a local drawdown around a repository and a drawdown of surface water?

Mr Gunnar Gustafson: What I was talking about was water table draw-down. But of course we will have a lowering of the water pressure, even if we anticipate a disturbed zone around the tunnel with a lower permeability. In most of the cases we have found that there is a permeability feature around the tunnels, which we have seen at Äspö too. We are not quite sure of the explanation and we are working on it.

Mr Brendan Breen: I think it is most important in water management. You will not necessarily have the flexibility to deal with it, I mean isolating it from your repository. So water management is probably more important than any sort of final requirement of total control of water table.

Mr Per-Eric Ahlström: Is the conclusion that whenever you pass a major fracture zone, where you might have a drainage, you have to seal it?

Mr Brendan Breen: I think we have to control the water by either isolating the zone to reduce the inflow or by just pumping the water inflow out.

Mr Göran Bäckblom: When we have problems with water pressure, of course there is an inflow, but it may be a matter of economy how much water you would like to pump. What we have learnt from fracture zones, and I suppose from Swiss experience, is that we have very unconsolidated fracture zones and it is very important to stabilize the material. If not, you will have an flow of material into the tunnels or through the investigation holes.

Mr Christer Svemar: We have primarily considered cement as grouting material. The question is how the chemistry will respond to the presence of that type of material, as it increases the pH. One impact of importance may be, if there is a defective canister and the water comes in contact with the fuel. Another is the chemical impact on the clay. How much concrete can we use close to the disposal holes and how far away can we use as much concrete as we like? The present situation is actually positive in that respect that we have not seen any restrictions, when we compare what should be required for grouting of minor fracture zones, which we can have close to the disposal hole. Nor have we seen any restrictions in predicting amount of concrete that we think we may have to use in order to stabilize drifts, when we are passing major fracture zones. We are also concerned about added organic substances like plasticizers.

Mrs Maria Onofrei: With new materials, that are coming out on the market, there are possibilities to develop cement grout which will not increase the pH very much. We have already developed some grout which will have a pH of 10 instead of 13. We know that the superplasticizer consists of huge organic molecules which – I do not have the evidence, it is just an educated guess – will form complexes that will be retarded in the clays.

Mr Per-Eric Ahlström: The problem is that those of us who have done safety assessment believe you, but in terms of safety the others calculate each atom of carbon and they presume that there are some bacteria, that will convert each atom of carbon to some complex, that will take the plutonium out of the fuel. Nobody knows what will happen in a thousand years. If you put down some carbon atoms in the repository, in whatever form, you could probably create some kind of complex. That is the concern, unless you can prove otherwise.

Mr Dale Wilder: One of the concerns we have had regarding organics is biological degradation of container materials and complexing of radionuclides. We do not have much information about that. Mrs Maria Onofrei mentioned larger molecules in superplasticizers. Our concern is introducing things that may help the biological process and that could be rather intense, especially if you use a kind of high performance material. I do not know if it is bad or good, but it is an issue that needs to be looked at.

Mr Paul Thompson: Just another comment on grouting, that should be mentioned. Essentially I do not see that there has been quite a bit of improvement in terms of material science and materials that we use, but in my mind I have not seen any major changes in grouting methodology for the 20 years I have been involved in this business. I describe it as a sort of black art. I do not see any real science. We sort of throw everything we can at the problem until we solve it.

Mr Håkan Stille: If we look at the sealing of this kind of conductive structures at the depth we are talking about, there is quite simply not much experience. That means, that it is something we have to learn. At least we are trying to make some science out of it. We have a theory which at least explains some of the things we have seen. We are trying to get a reasonable strategy from that experience. A strategy that is not contradictory to good practice in grouting. What I hope for, is that it will give the same answers as a clever grouter with experience and even explain something more when we get into a difficult situation. That is what we are aiming at.

Mr Reijo Riekkola: Some of what has been achieved in the Stripa project is a development of dynamic grouting technology. So far the development has not continued, but I would like to mention that in Finland we have a development project and we are trying to find a more practical tool for normal grouting. We have had two designs on how to deal with it. I believe that the last one is going to be successful.

Mr Håkan Stille: Do you think that a dynamic grouting method is applicable to a major fracture zone at great depth?

Mr Reijo Riekkola: That is not one of the major points for dynamic grouting. It is more aimed at how to get the grouting materiel to penetrate small fractures.

Mr Göran Bäckblom: We must realise that we cannot construct a repository if we do not comply with the working regulations. We can say and plan whatever we want, but people will not work outside the regulations. We are now entering new fields of development work and construction work.

Suppose that we have a map of fracture zones. Which will be difficult to grout and which will not be difficult to grout? It is quite important, because if we have zones that are very difficult to grout, we should really avoid just those problems when constructing a repository.

Mr Brendan Breen: I think it is important when you are grouting, that you must have a barrier or a medium which will allow or alternatively form a seal. If you try grouting a jointed material, the water will disperse and it will, as you quite rightly say, find another route. Unless, of course, that route is blocked by a rock of very low permeability. So you need natural barriers to be able to grout against, otherwise you just move the water on or you must create some form of resistance so that the water does not find a route through or follows an easier route away from the working location itself. It looks as if that is the case where you, Mr Riekkola for example, said the shotcrete provided a sufficient barrier to water inflow, that it must have been found an alternative low-resistance discharge. Otherwise, you would have had water through the shotcrete, because the shotcrete is not impermeable.

Mr Dwayne Chesnut: But then we have the question of how you know where you have the barriers. It seems like you almost have to live with what you get.

Mr Brendan Breen: You have to know, that you have got a highly impermeable barrier, because then you can not set a seal against it. Otherwise, you have got to be able to provide sufficient grout and grout pressure to allow a diversion of the water or containment of the water. But water will follow fissures.

Mr Dwayne Chesnut: I understand that the main reason for grouting is always going to be to control water and it is not going to be for mechanical stability.

Mr Roy Stanfors: I think you, Mr Bäckblom, gave the answer yourself. But which fracture zones are easy and which are difficult to grout?

It is normally always very difficult to grout complex zones because you have sub-zones and a mixture of different conditions, you have open fractures with crushed material. So we can say you have fracture zones with high transmissivity which are easy to grout and you have more complex zones.

Mr Dwayne Chesnut: You say more complex. You mean more variability?

Mr Roy Stanfors: Yes.

Mr Gunnar Gustafson: It takes a long time to grout a complex zone. It may not be caused by people doing the wrong thing. It is complicated and for theoretical reasons you find, that you cannot fill the hole around the grouting, which is of course what everyone wants to do. It means that you have to regrout in the same position and not have time to really fill the space. It takes time and when things do not go as we like them to go, even in the best controlled situations, you get a little panic because you do not get where you want to and then you try all kinds of grouts and things. My impression is that this work takes time and has to be done more or less. – There is another question of moving water. It is a matter of contrast between conductive structure and the surrounding rock. If you are within an order of magnitude in complex conductivity between the zone and the rock surrounding it, the total leakage will be about the same even if you do not observe it. Then the grouting may have a stabilizing effect, so that you do not have the problems associated with constant inflow. The amount will be roughly the same. But if you have a contrast of several orders of magnitude between the zone and the surrounding rock, then you will have a real effect on the total leakage amount. You can reduce it substantially. It is a matter of contrast to a great extent.

Mr Gary Simmons: That contrast is an important issue for the point that Mr Breen was raising as well. You are placing seals to provide long-term isolation for waste if you are relying on the rock in zone of low hydraulic conductivity as a barrier. However, during construction and operation of a repository you will very likely need to put in a plug of some sort in a highly conductive zone to control water inflowing during those periods. Based on some of our experience in a situation where we have a large range of transmissivities, you can identify the transmissivity distribution, at least in a fairly gross way, early in the characterization of a site and you can accommodate it in the arrangement of the repository.

When you are underground you can find the local variability in the transmissivity and select locations of competent rock where you know you will be placing seals for long-term isolation and locations where you will require seals or tunnel liners for operational safety and groundwater control during those periods.

I think there is some technology and experience can be used to answer your question. It is knowing what you have to look for during site characterization that will allow you to determine what areas may be more readily grouted than others and to place the repository in and near those areas that may be harder to grout (i.e. have competent rock).

We probably need to do some more development in methods of grout application. We require methods that will allow us to deal with various fracture infillings and other things that may lead to a poor grout plug and to eliminate these factors to some degree. We should work towards getting a better quality plug on the first application of grout even though it may not be the final quality that you desire. We have done a little work on using high pressure water jets to try and sweep gouge material and clay out of fractures where we want to install a grout plug.

Mr Andy Nold: I just like to say, that our thoughts are going in the same direction as you Mr Gary Simmons just pointed out.

We should distinguish between fractures, crossing access-tunnels and shafts, which are in fact just to be overcome by civil engineering means to get a structure made. Then we have to distinguish other zones which may intersect the repository tunnels. Our conclusions were, that within the disposal area we should not have any major fracture zones. We came to a point that it is impossible to prove that you can make a site which lasts forever, depending upon the time of repository we are taking about, a thousand or ten thousand years. It is hardly possible to prove, even if one could believe that it may last. But to have proof is so hard, that we say that we avoid this by having major disturbed zones in access-tunnels only, thereby bringing the problem of characterization down one or two orders of magnitudes in difficulty. The impact on long term safety is much less far away from the actual disposal region. This is roughly the philosophy that we are applying in our project.

Mr Reijo Riekkola: Another area of the so called siting exercise is, that we want to avoid the main zones of weakness and also certain kinds of main water-bearing fracture zones. What you want to do with the rest of the zones depends on different kinds of criteria. You may characterize these zones. If you penetrate them beforehand with one or two holes, can you be sure that the water conductivity or transmissivity in the zone is within the scale that you measured in these points? Or do you have to be prepared actually for worse conditions that you should have tried to avoid? Or do you have to delay the decision until you have seen the actual conditions and make the measurements at that time?

Mr Christian Sprecher: What kind of criteria are you Mr Reijo Riekkola using to classify the fracture zones? I saw that you distinguished major fracture zones, which you try to avoid, and to which you keep a certain distance. How do you classify them?

Mr Reijo Riekkola: It is a tough question and I have to say that it is not very systematically solved. Exercises have made the classifications according to transmissivity in three different classes. We have created some kind of guidelines. We definitively want to avoid zones that have higher transmissivities than the first class and we have other criteria for the second and third class. But this classification from a rock engineering point of view is still under development and these are the results that the site investigation group has given us so far. We use different guidelines depending on what how the repository looks like. It is possible to avoid any known fracture zone, providing the layout is adapted properly.

3 NEED FOR DEVELOPMENT OF TECHNOLOGY (Major Fracture Zones). GENERAL DISCUSSION

Discussion leaders: Mr Per-Eric Ahlström and Mr Göran Bäckblom

Mr Göran Bäckblom: We would like to construct a repository at great depth and there are many questions that could be addressed regarding practical problems. In order to have a good discussion we will try to look at different sub-topics:

- Do we have the investigation technology underground to characterize major fracture zones? Drillholes through major fracture zones?
- Do we have instruments to use at high water pressures or high water inflows?

What do you think about the need for development? In the Äspö laboratory we have learnt that when we drill a hole in a major fracture zone, we get lot of water. And we get high water pressure. We have found it very difficult to drill a hole through a fracture zone. We have, so far, had problems with limitations on what kind of equipment we can insert into fracture zones or bore-holes and make measurements. It would be interesting to hear your perspectives, ideas and experience on investigation technologies.

Mr Roy Stanfors: I think it is important to say that you need investigation technology at first to find your fracture zones and I think that the problem is not mainly to find the major fracture zones. I think we know how to locate them and also to characterize major fracture zones, but it is very important to have in mind, also what we call minor fracture zones. I mean fractures 1 or 2 metres wide, which can be very difficult, especially when we are using a TBM method. It is also important for us to really be able to characterize fracture zones in order to use the test technology designed for grouting. I think that many times we do not know so much about fracture zones. We have bad characterizing before we start grouting and especially for complex zones we have had problems. It is very important to know the variability of the zone before we start grouting. I think we have been talking about seismic methods but can somebody here really say that we know what we see from this geophysical indication? I think we have indications, but it is very hard to translate them into practical matters. It is easy to say that we can drill and investigate a zone by drilling but it is not easy in reality. We have tried through any zone about 10 metre wide. We could not do that with normal methods. I mean, we have methods and we use them, but I think we need improvement for the future to use these methods as bases for excavation.

Mr Gunnar Gustafson: In terms of major and minor zones I would like to comment that the problem is, that you are going to recognize that in most of the methods you will find the major zones and there are some ways that you can look at "flowfields". Until you go through a stage of graving the major zones, the minors are not going to respond very much, so you do not want to identify them. I think you have seen that in Stripa and I am sure you have seen it at Äspö, for the majority of your waterflow is in a major zone and until you get them sealed up it is pretty hard from a geophysical standpoint to recognize where those minors are.

It might be worthwhile to think a little about what we are looking for. Every major fracture zone consists of a number of fractures. If you try to cut through them, you find that they have an inner structure that is not so simple. My impression is, that there are a few fairly long but conductive fractures, that are linked with some others. That means, that if you take a bore-hole, you find that if the bore-hole is long enough, the local transmissivity is as a rule larger than the transmissivity of the whole zone. That means, that if you go with a tunnel here, you will have basically water from the whole zone every time you hit a fracture. Then there is the question of what we want to do, to see what we have in front of us. We have done it by regularly having these probe-holes. What can we find with these probe-holes? How dense must a system be for us to identify conductors in the zone? These grouting-fans in positions where there has not been too much grouting already are an excellent way of looking at sizes of conducting fractures. This is a variogram of conductivities or transmissivities between holes in fan No 45. We have done it for composite and some of them look like a mess, but here we have a reasonable number. What you see here is a variogram, that goes from close to zero up to a sill somewhere around 2 to 3 metres. That means that the size of the feature we have is in the range from 2 to 5 metres. We want to identify these. If we are really close to something that can give us trouble, the best probe-holes are really the grouting holes. Then there is another disadvantage. There is very little time to decide what to do, what to put in the holes to get the maximum effect. There is another prejudice that I have. The "penny-shaped" fracture. You have seen that in literature. I think they look something like this. This is the accumulative distribution of transmissivities. If you assume that these ones, which are almost identical, are in the same conductor and put these together, you find a picture like this. But I think that the fractures really look like this. It goes quite well together with the zone-picture you find in URL. So that is how we have come to look at it. To hit one of these you really need something like this to find it. Characterization underground, without really knowing that we did the right thing, but we did a reasonable thing by going closer and closer, and as we got closer made the observation system more and more dense and finally we had the grouting bore-holes. That was very simple as all the tests were made in every one of these.

Mr Dwayne Chesnut: When you draw your characterization holes, when do you get most of the troubles? When you first encounter the fracture zone or when you continue? What happens when you first start to get a major water inflow. You stop and grout that hole?

Mr Göran Bäckblom: I think that when you have major water in-flow you need grouting. You have of course the option of drainage.

Mr Norbert Krauland: I am still a bit confused about the nature of the material within the fracture zone. It is not like rock. What is the material? Do we know what it looks like?

Mr Gunnar Gustafson: We have all of it. It is not quite sure that sand and gravel and clay and whatever is the most conductive part. I think Mr Roy Stanfors has the best knowledge of that.

Mr Roy Stanfors: After passage we found that the most intense part and the core is almost all clay. The fine-grained granite was crushed with clay. It is important to say that about 5 m^3 material was washed out during drilling.

Mr Göran Bäckblom: By the way, Mr Thompson, do you have any comments on the need for development in investigation technologies?

Mr Paul Thompson: The only comment that I would make would be specific to the URL. The only suggestion is to characterize the fracture as well as you can. In this case the variability is very low in terms of transmissivity, so I do not see any big advantage to elaborate characterization programs and how far you want to go.

Mr Göran Bäckblom: Do you think that you have the tools for characterization, even if there are very high water pressures?

Mr Paul Thompson: We have not done any higher pressures than 2 MPa. So we have some instrumentation at the 420 metre level with higher water pressures but very low water flow. There are no open fractures there. I suspect they will work at higher pressures but what the limits are, I cannot say right now.

Mr Gary Simmons: We do have vacuum systems installed at 1200 metre depth in bore-holes and in addition at 600 to 900 m depth and we do not have any significant leakage problems with them. I do not expect that there will be a great deal of difficulties if you have unlimited money like the petroleum industry.

Mr Per-Eric Ahlström: The question was if we can characterize a location with high water pressure. I think Mr Göran Bäckblom's question was: if you are coming with a tunnel and you are 10 metres ahead of some fracture zones at 600 – 700 metres depth.

Mr Gary Simmons: It would be somewhat difficult. But it will largely be a structural problem. You would not necessarily expect your packer to withstand that difference of pressure. But you could anchor your package-string in the mechanical anchor in the tunnel so you can rock both the anchor back into the rockwall and use it to carry the load. The question is: can your packers handle that kind of situation and the answer is probably, except very near the tunnel.

4 PART II – CONSTRUCTION, OPERATION AND SEALING

TABLE OF CONTENTS PART II

4.1 DESIGN AND CONSTRUCTION OF REPOSITORY – REQUIREMENTS AND CONSIDERATIONS

Christer Svemar, Swedish Nuclear Fuel and Waste Management Co, Stockholm, Sweden

4.1.1 INTRODUCTION

The objective of underground construction and operation is to dispose of the nuclear waste in a way that fulfils the requirements on long-term isolation of the waste and allows for full monitoring of bedrock conditions during the operational phase to ensure occupational safety and provide reliable data for long-term safety assessment of the repository.

Requirements on occupational safety in underground activities are well established in the mining and underground construction industry and broad experience exists regarding how to plan and conduct the work in a safe manner.

A new problem, however, is to consider restrictions imposed by the requirements of long-term isolation of the disposed nuclear waste. The very long times involved mean that extra allowances for contingencies are necessary and this can be expected to introduce new considerations regarding the way work is conducted underground.

4.1.2 DESIGN AND CONSTRUCTION

The repository is designed in several phases, each phase aiming at a higher level of detail based on more detailed information and more detailed analysis. A widely applied approach is to separate the design activities into three phases: 1) Feasibility design, 2) Preliminary design and 3) Detailed design. These phases comprise an iterative process which for a bedrock repository may be illustrated as shown in Figure 4.1-1.

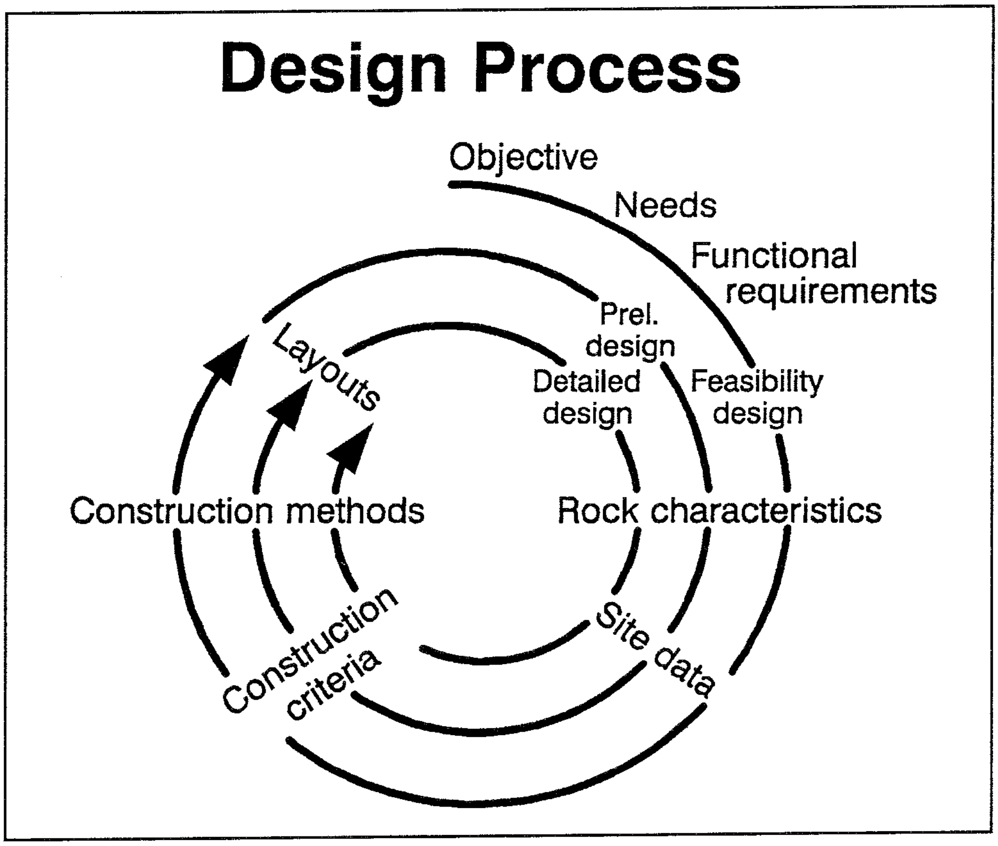


Figure 4.1-1. Illustration of the repository design phases.

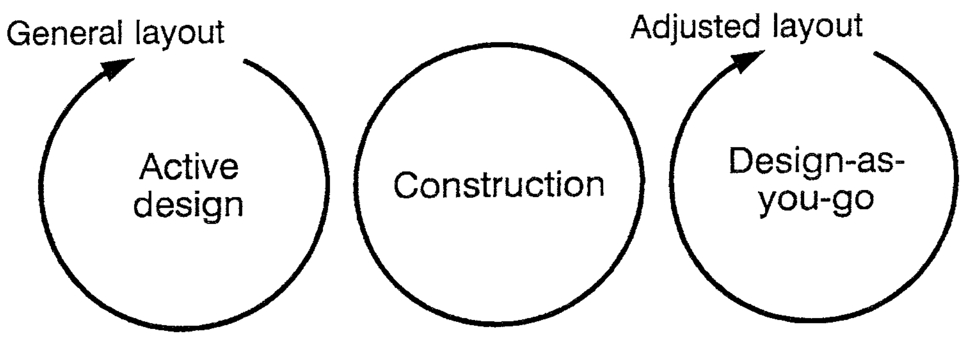


Figure 4.1-2. *Illustration of the interaction between design and construction.*

The actual design work has two components: Design theory and Design methodology. *Bieniawski (1992)* has presented the following definitions:

> *Design theory: Systematic statement of principles and experimentally verified relationships that explain the design process and provide the fundamental understanding necessary to create a useful methodology for design.*
> *Design methodology: Collection of procedures, tools and techniques that the designer can use in applying design theory to design.*

The design process has been fairly thoroughly analyzed with respect to the procedure for designing equipment, buildings and general surface structures. The process is somewhat different and much less thoroughly understood for underground facilities, one explanation being the necessity of adjustability to the events occurring during construction.

Aspects of the design can be adjusted during the course of construction as is illustrated in Figure 4.1-2. In the mining industry, for instance, this "Design-as-you-go" approach is well established. Provisions are made for this in the design phases, denoted here as the "Active design" phase as opposed to "passive" design). It is obvious that "Active design" and "Design-as-you-go" will have to be synchronized in order to guide the construction in an efficient way.

Figure 4.1-2 shows that the construction work underground involving rock excavation, rock support, grouting, installations etc. is based on the detailed design made prior to the start of the work. The guiding principles during the course of the work are occupational safety and the need to achieve and document long-term performance and repository safety. New and more detailed information on the rock is analyzed when it is received and, when appropriate, used to adjust the design. The construction loop is illustrated in Figure 4.1-3.

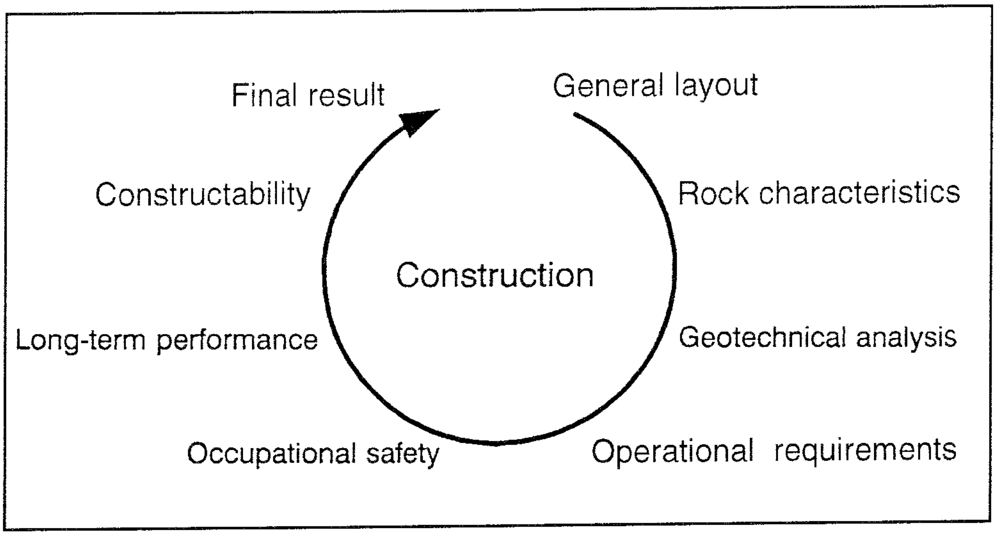


Figure 4.1-3. Factors affecting the construction work.

4.1.3 REQUIREMENTS AND CONSIDERATIONS

4.1.3.1 Requirements

The primary concerns of the design phases are the requirements and criteria dictated by the long-term performance of the repository, and by occupational safety underground. The requirements of long-term performance are in practice converted into a layout specified by design parameters such as dimensions, volumes, weights, engineering materials etc., which make up the "Goal". In order to achieve the "Goal", it is necessary to execute the construction and disposal processes as well as the backfilling and sealing operations in such a way that the prescribed design parameters are obtained.

4.1.3.2 Considerations

The "Goal" is established with due allowance for contingencies occurring in the course of construction. Plans and procedures are prepared for different activities and include reasonable allowances for flexibility. In practice, if surprises occur, adjustments to procedures may be deemed necessary if the basic requirements and criteria are to be met.

Occupational safety is also of major concern. Hazardous situations will have to be solved in a safe and controlled manner, maybe even with greater caution than is ordinary in underground operations due to the fact that construction and operation are taking place in a nuclear waste repository.

4.1.3.3 Examples of questions of concern

The requirements and considerations are primarily considered in the "Active design" phase, and most parts of the "Detailed design" are expected to be executed as planned. Certain parts of the "Detailed design", however, such as the exact position of deposition tunnels and deposition holes, most probably fall under "Design-as-you-go", as do other construction and operation activities where the basic design is questioned.

Examples of questions of concern are:

Long-term performance

- The possibility that radionuclides will dissolve and migrate.
- Canister failure (corrosion, pressure etc.).
- Bentonite alteration.
- Groundwater composition.
- Groundwater transport paths.

Construction of repository

- The use of drill and blast, TBM or disc cutters in relation to the disturbance of the rock wall.
- Rock supporting methods such as rockbolting, shotcreting, wire meshing in relation to the use of different engineering materials.
- Methods for crossing fracture zones, if the crossing is close to a disposal position or far away.
- Methods to control difficult rock conditions such as rock burst, breakout etc.

Disposal of canisters and emplacement of buffer materials

- Acceptable water inflow in different parts of the repository.
- Stability of rock close to disposal positions.
- Timing of disposal activities.
- Acceptance of engineering as well as stray materials.

Occupational safety

- Radiation protection.
- Fire fighting.
- Personnel rescue/evacuation systems.

4.1.4 SKB PLANS

The initial step for SKB will involve identification and listing of items that are judged to have an impact on design, construction and operation. It will also involve evaluation and proposal of methods, equipment, procedures etc. for providing suitable practical solutions.

In a second step existing methods, equipment etc. will be adjusted, or if necessary, new ones developed for the proposed solutions.

Finally, in a third step, methods and equipment which are not commercially available will be tested and demonstrated.

4.1.5 REFERENCE

Bieniawski Z T, 1992. Design methodology in rock engineering. A. A. Balkema Publishers, Old Post Road, Brookfield, VT 05036, USA.

Questions to Mr Christer Svemar

Mr Dwayne Chesnut: I understand the basic idea to avoid major fracture zones, which will be the final determinant of water inflow for any placement. Then you have minor water flow possibilities in individual boreholes for canister placement. Could you, e.g. grout a canister placement-hole to make the short term inflow acceptable for a placement or would you try to avoid that drillhole?

Mr Christer Svemar: I think it is an economical trade off. We do not, however, have any good method today for prioritization.

Mr Dale Wilder: As a point of reference regarding need to control hydraulic fluid leaks from TBMs, we did a test of thermal effects on hydraulic oil in the presence of water, using a goldbag autoclave. The hydraulic fluid broke down and it completely destroyed the goldbag. Therefore, loss of hydraulic fluid into the environment is not a minor issue for repositories.

Mr Michel Dardaine: Close to the major failure zone, will you build backfill with current techniques or do you reinforce the tightness of the backfill to limit the water flow?

Mr Christer Svemar: I think that in certain cases we need to take precautions like making a plug. The Swiss made a study many years ago on the long term effect of placing a canister at different distances from such a water-bearing fracture. They calculated numbers and make evaluations of the importance of each distance. That is a basic evaluation. In addition I think that some engineered protection is needed in order to be certain to have a sufficient barrier against water transport. In this context we are trying to find designs for plugs and how and where to put them.

4.2 THE PLANNING OF THE ROCK CHARACTERISATION FACILITY (RCF) AT SELLAFIELD

Brendan J Breen, Mining Manager, UK Nirex

4.2.1 INTRODUCTION

United Kingdom Nirex Limited is a Company jointly owned by British Nuclear Fuels plc (BNFL), Nuclear Electric plc, The United Kingdom Atomic Energy Authority and Scottish Nuclear Ltd. The Secretary of State for Trade and Industry holds a special share. Nirex are charged with the responsibility to provide and manage a national disposal facility for solid intermediate-level (ILW) and low-level (LLW) radioactive waste.

Following earlier assessment work on suitable sites and an exploration programme at Dounreay in Caithness and Sellafield in Cumbria, the Company announced (July 1991) its intention to concentrate further investigations at Sellafield.

Since then continuing site investigations have provided more geological and hydrogeological information particularly on the basement rocks of the Borrowdale Volcanic Group (BVG).

In the assessment of long-term safety a continuing programme of surface-based investigations will proceed, but in October 1992 the Company announced /in *Nirex Report No 327 – Sellafield Repository Project, A Rock Characterisation Facility Consultative Document*/ its intent to pursue a Planning Application for a free-standing phase of underground exploration by gaining access to the basement rock via vertical shafts to develop a fuller understanding of the surrounding geology and hydrogeology. This would complement the surface investigations and provide a basis for refining analysis of long-term safety. Additionally more precise geotechnical information would be gained to enhance the subsequent detailed design and more accurately determine the construction methods.

4.2.2 OBJECTIVES

The primary objective of the RCF will be to provide rock characterisation information (and scope for model validation) to permit firmer assessment of long-term safety, to enable decisions on the detailed location, design and orientation of the repository and the most suitable construction.

The results of borehole drilling and geophysical surveys to date have indicated that a suitable block of BVG rock exists at circa 650 m bOD (below Ordnance Datum) and that this can be investigated by establishing exploratory underground roadways at that level with the potential to extend investigations to depths down to 900 m bOD.

Throughout the feasibility study for the RCF the environmental impacts were identified and assessed and appropriate mitigating measures incorporated into the design. The design is evolving and will be modified in the light of scientific, engineering and environmental considerations. This formed a starting point for consultations with local authorities and other related bodies with the aim of identifying and discussing all impacts to enable any appropriate refinements to achieve an environmentally acceptable design.

Following a full process of consultation, the Company propose to submit a planning application for the RCF by mid-1993. Assuming planning approval by the end of 1993, site establishment could commence early in 1994.

A planning application for development of the repository would follow once the long-term safety potential for the site could be assessed with sufficient confidence, taking account of the investigation programme as a whole. If early results from the RCF are essentially confirmatory of the geological and hydrogeological assessments derived from surface-based exploration then a planning application for repository development could be submitted by late 1996 or early 1997.

In planning the underground access to establish the RCF the over-riding need is to ensure that the construction of the facility meets the primary objective of ensuring effective characterisation work.

4.2.3 GEOLOGY AND AREA DESCRIPTION

Figure 4.2-1 shows the general area around the Sellafield site with the proposed site of the RCF and the area for further investigation delineated. Access will be by vertical shafts sited on land currently owned by BNFL, west of Longlands Farm. The shaft will be located in the north-eastern part of the potential repository zone, which, based on current indications of deep water flows should minimise any risk of compromising the long-term integrity of the block of rock identified as a potentially suitable location for a repository.

A planning application has already been submitted for a programme of boreholes adjacent to the shaft site, principally to confirm the position of the shaft site, to provide baseline monitoring for the hydrogeological regime prior to disturbance by construction and to provide construction-specific geotechnical information to support detailed excavation design.

Figure 4.2-2 shows a general geological section through the proposed site as established from regional boreholes drilled since 1989. Figure 4.2-3 shows the location of regional boreholes.

The strong, low-permeability basement Borrowdale Volcanic Group is directly overlain at the RCF location by a low-permeability layer of muddy matrix breccia (Brockam) which is in turn overlain by approximately 400 m of sandstones of the Sherwood Sandstone Group.

The BVG outcrops near Gosforth to the east of the proposed site but descends steeply to the west being some 1600 m deep under the Sellafield Works. To the west and north of the RCF area limestones immediately overlie the BVG while to the west the Brockram is succeeded by the St Bees Evaporites and the St Bees Shales.

The sandstones are overlain by glacial deposits of varying thickness. The sandstone beds are known to be water-bearing, particularly in the upper 100 m and will require ground treatment and sealing during shaft sinking. Until a more detailed site-specific investigation is pursued within the proposed investigatory drilling programme, it has been assumed that the full section to circa 400 m will require ground treatment.

4.2.4 CONSTRUCTION DESIGN

4.2.4.1 Site preparation

The site area is on a moderately sloping ground and excavation of unconsolidated ground will be progressed on a cut-and-fill basis giving an overall reduction in surface base level at the shaft sites. Throughout the site planning due consideration has been given to mitigating the environmental impacts, details of which are given in Nirex Report No 327 */Sellafield Repository Project/*. Figure 4.2-4 shows a general site layout during the development phase of the RCF.

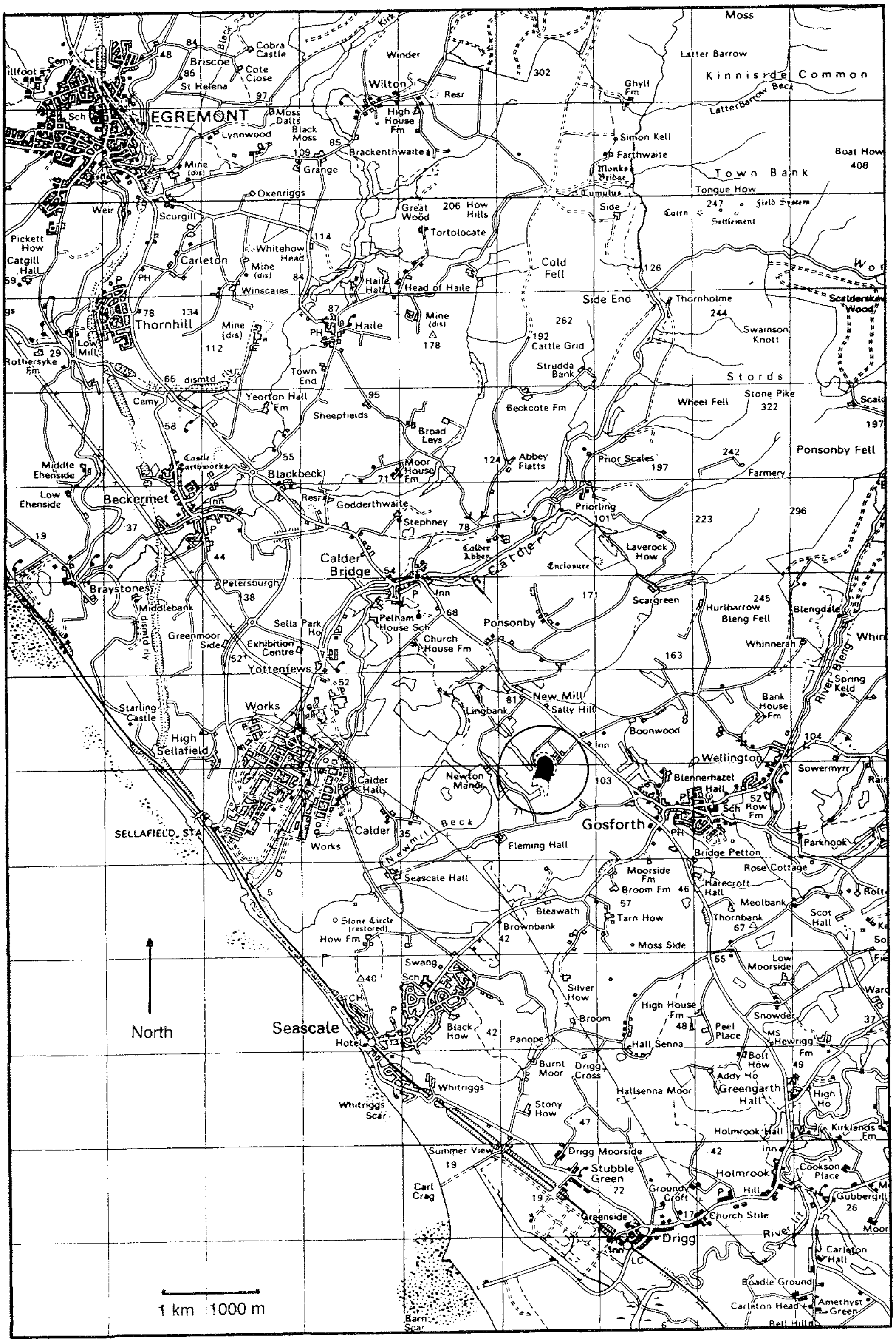


Figure 4.2-1. *Proposed site of Rock Characterisation Facility.*

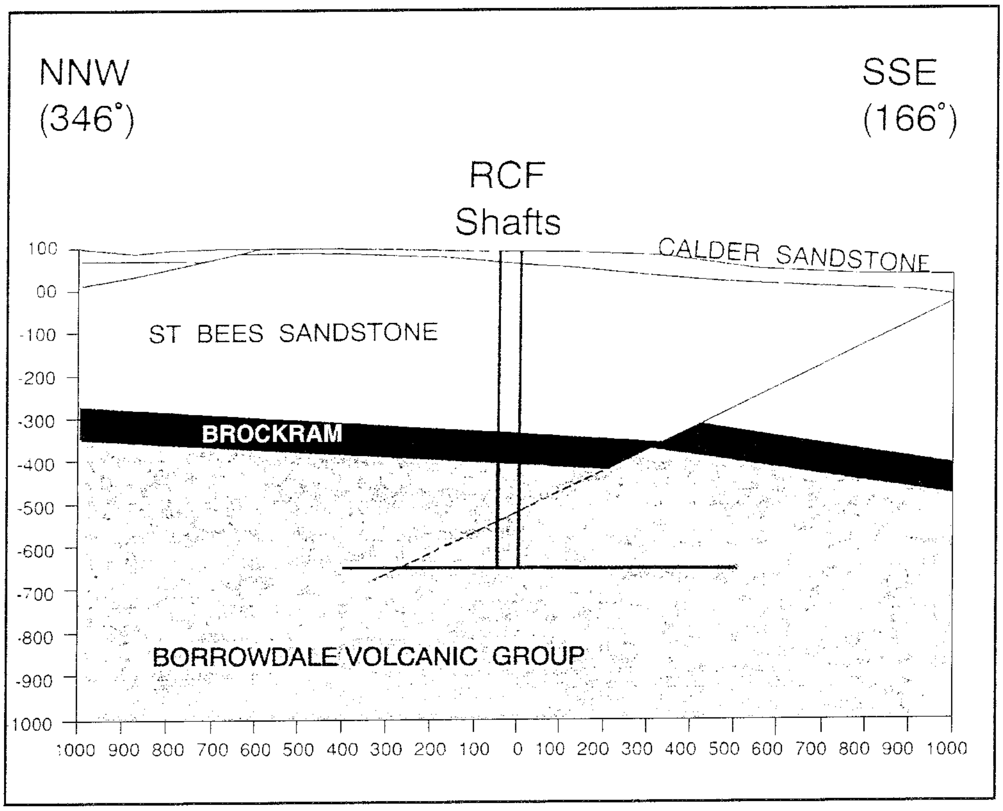


Figure 4.2-2. *Projection of RCF, Phases 1 to 3, viewed from WSW.*

The base surface level will be constructed on imported stone fill, laid over an impermeable membrane to prevent risks of contamination of the sandstone aquifer and nearby streams due to spillages. Arrangements would be made for collection and discharge, after appropriate treatment, of water from underground construction.

Office accommodation and storage facilities would be established on site together with major items of plant and equipment required in the early phases of development.

4.2.4.2 Freezing

Based on available geological information the most reliable and appropriate means of groundwater management will be provided by ground freezing. This process entails the drilling of a ring of external boreholes concentric to each shaft. These would be equipped with double-wall freeze tubes to allow circulation of chilled brine in order to form an icewall surrounding each shaft. The freeze would be extended into the Brockram layer to ensure that there are no leakage pathways.

Pressure relief holes would be drilled in the centre of each shaft to relieve water pressure during freezing and additional monitoring holes would be placed around the shaft to monitor the build-up of the icewall.

As the shafts are sunk through the frozen strata they will be lined with a permanent concrete lining to prevent water ingress once sinking is complete and the ground is allowed to thaw. At this stage some backwall grouting will be necessary to fill any voids behind the lining and provide an effective seal.

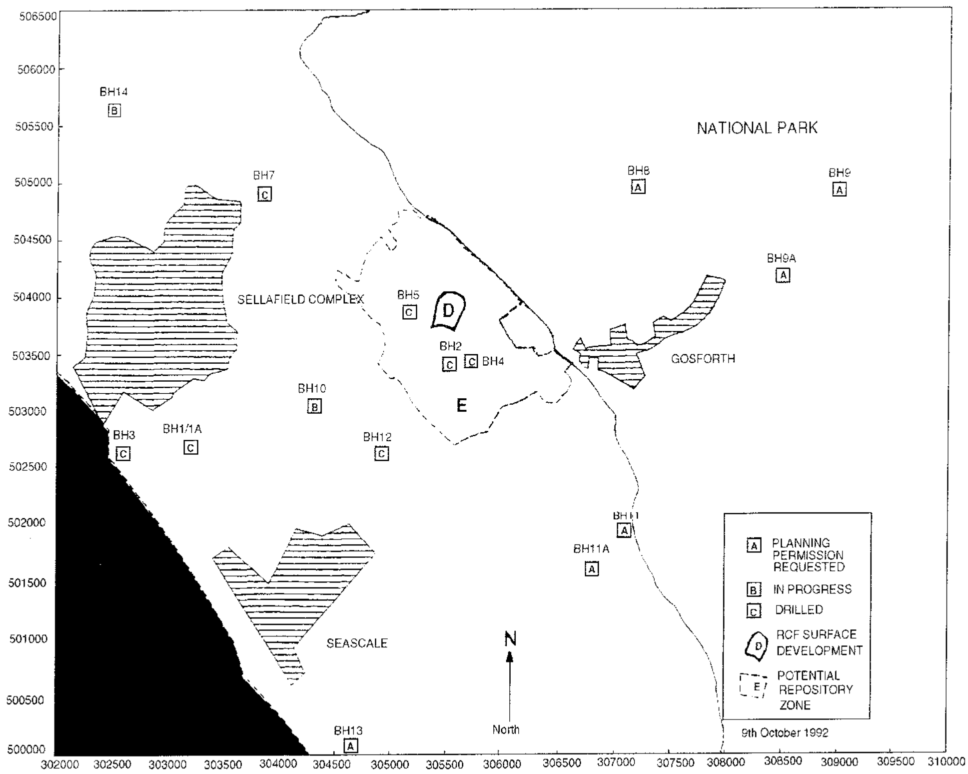


Figure 4.2-3. Location of regional boreholes.

Freezing is a well-tried technique which has been successfully applied to a variety of difficult shaft sinking operations particularly throughout the UK, but also in Europe and North America.

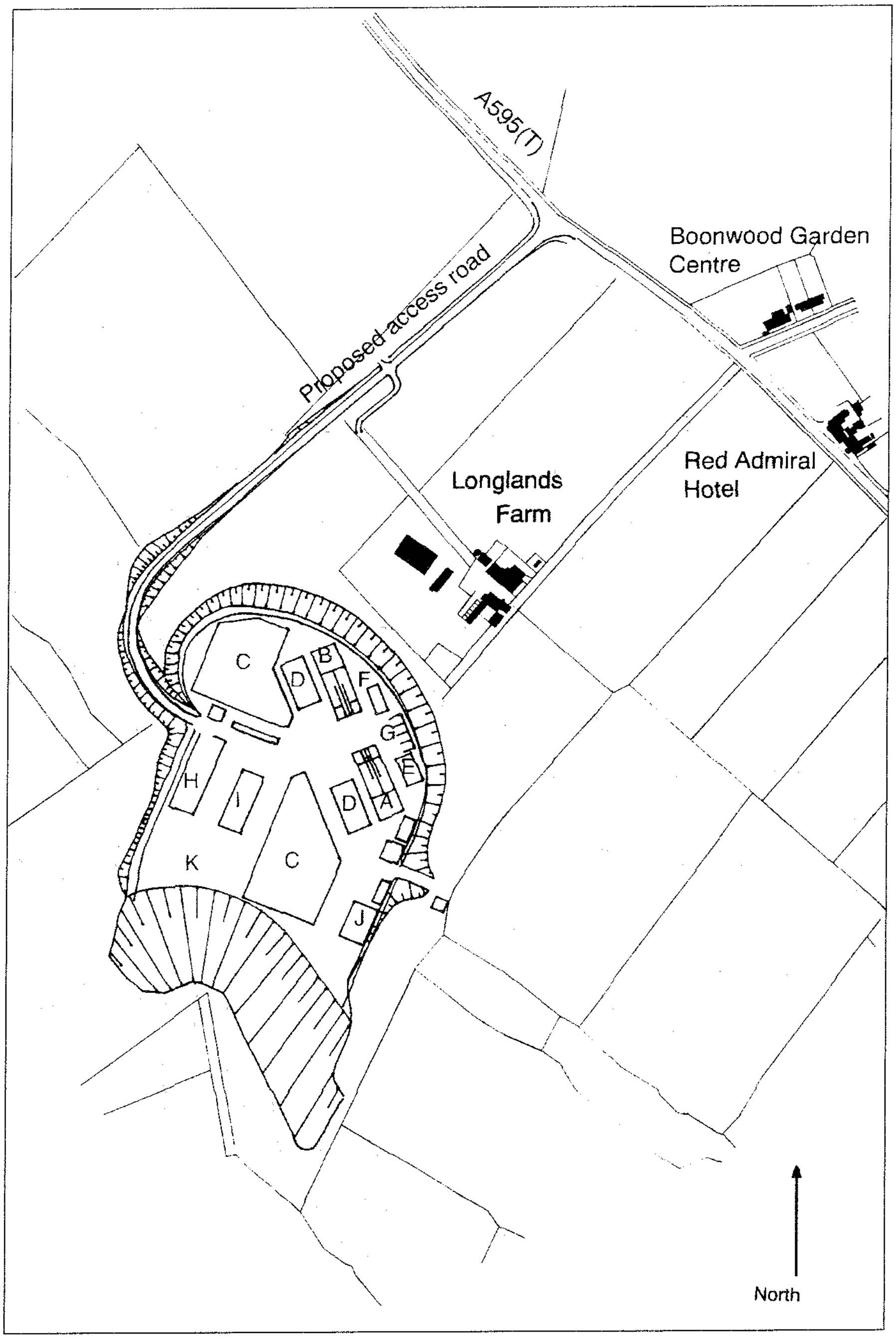
4.2.4.3 Collar and foreshaft construction

Prior to shaft sinking a foreshaft to some 30 m depth below the shaft collar would be constructed using conventional civil engineering techniques. This will provide sufficient depth to install the sinking stage below surface level as well as providing a rigid structure to support the shaft headgear.

4.2.4.4 Shaft sinking and gallery construction

Shaft sinking will be carried out using a modular shaft sinking unit and employing conventional drill and blast techniques. The sinking stage would be designed to enable characterisation work, essential to the purpose of the RCF, to be carried out safely and effectively. The emphasis for this work would increase considerably from the point of access to the BVG.

Shaft insets would be established at various horizons as shown in Figure 4.2-5. These would be constructed to enable testing work and potential access to selected horizons in the ongoing characterisation work. Exploratory galleries and chambers would be driven at the 650 m level to investigate this horizon more fully. There would also be an option of investigating other horizons down to 900 m bOD.



A - Shaft 1
B - Shaft 2
C - Shaft stock yard
D - Freeze plant
E - Concrete plant
F - Cement silos
G - Aggregates
H - Offices
I - Stores
J - Settlement tank
K - Spoil tip

Figure 4.2-4. *Preliminary layout of surface development for RCF.*

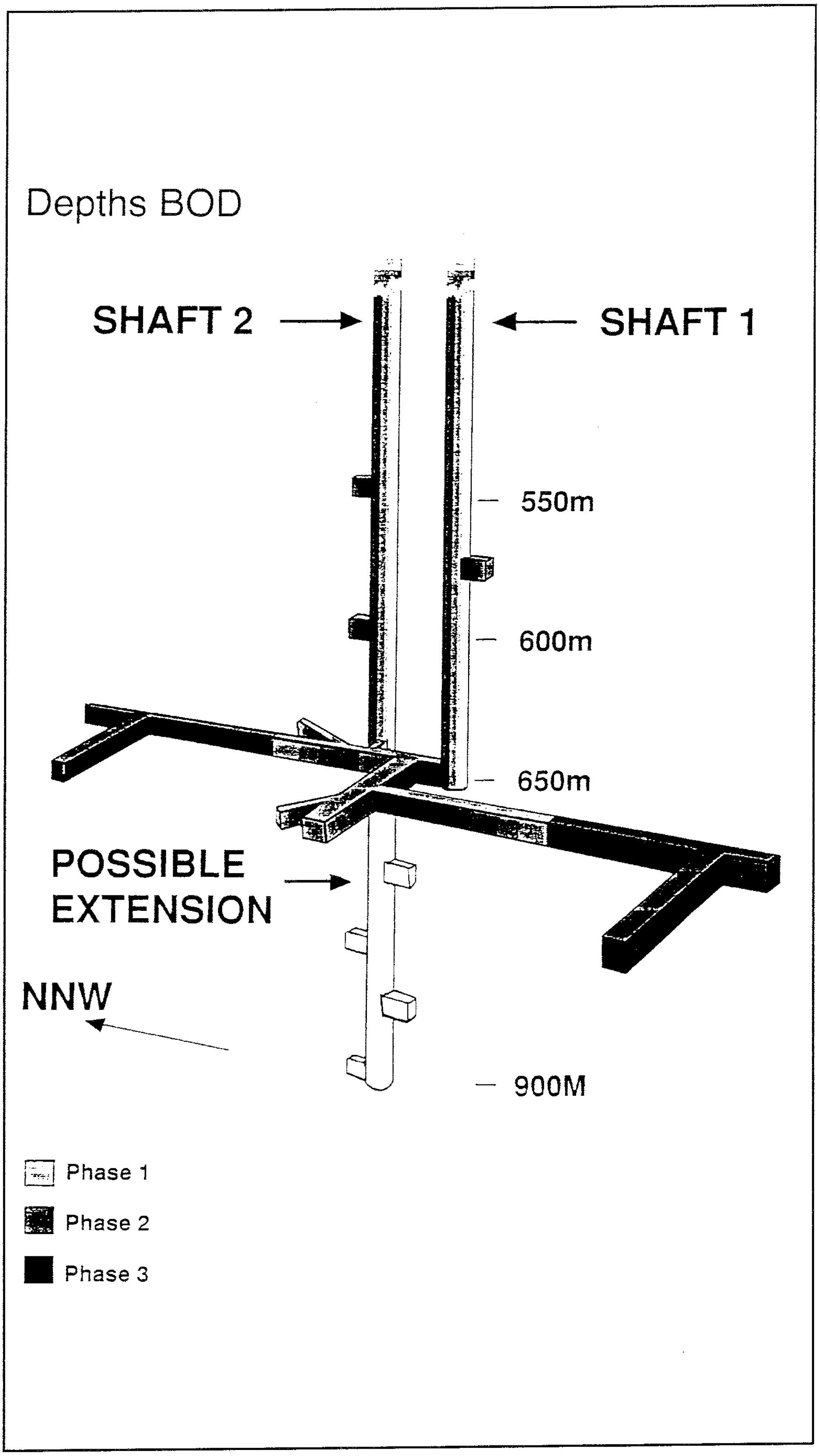


Figure 4.2-5. *Schematic design of a rock characterisation facility.*

PRELIMINARY IMPACT ASSESSMENT MATRIX								
	Surface construction	Servicing	Transport	Spoil disposal	Underground construction	Surface operations	Underground operations	After-use
Land use/land take	●		●	●		●		●
Habitat	●	●	●	●		●		●
Soil	●	●	●	●				●
Natural resources								
Landscape/visual	●	●	●	●		●		●
Lighting	●					●		●
Surface drainage	●	●	●	●		●		●
Ground water					●		●	●
Water courses	●	●	●	●		●		●
Emissions-gas	●				●	●	●	●
-particulate	●		●	●		●		●
-energy	●				●	●	●	●
-odour								
Noise	●	●	●	●		●		●
Vibration	●	●	●		●	●		●
Agriculture/forests	●	●	●	●		●		●
Recreation		●		●				
Rights of way		●		●				
Archaeology		●		●				
Utilities-water	●					●	●	●
-electricity	●				●	●	●	●
-gas								
-drainage	●				●	●	●	●
-pipelines								
-telephones	●					●		
Employment-direct	●		●		●	●	●	●
-indirect	●		●		●	●	●	●

Figure 4.2-6. *Preliminary impact assessment matrix.*

4.2.5 SCIENTIFIC TESTING REQUIREMENTS

Access from within the shafts would permit the direct evaluation of the in situ rock mass together with validation of the predicted geological structure. The provision of adjacent monitoring boreholes will enable the effects of shaft sinking on groundwater movement to be determined.

Once the galleries are established a programme of small diameter, multi-directional probe drilling will enable comprehensive mapping of fractures, faults and other features in the block of rock. This will be progressed in conjunction with a variety of hydrogeological, geochemical, geotechnical, chemical and thermal testing to enhance the determination and accuracy of the longer term predictions for disposal within the BVG.

4.2.6 ENVIRONMENTAL FACTORS

The overall provisions of the proposal are geared towards ensuring that the environmental effects of the establishment and operation of the RCF meet Nirex commitment to use the best available techniques to ensure good site management practice at all times. Figure 4.2-6 shows the preliminary impact assessment matrix.

4.2.7 REFERENCE

Sellafield Repository Project – A Rock Characterisation Facility Consultative Document Nirex Report No 327, October 1992.

Questions to Mr Brendan Breen

Mr Dale Wilder: If I understood you correctly you did not expect major flows in the sandstone?

Mr Brendan Breen: In the upper sandstone, yes. What I, however, did not mention was that in the freeze zone there will be some grouting in order to fill the voids and fractures which have been filled with ice. Equally, I think that it is important when you establish a freeze is that you put pressure relieving holes within the centre of the shaft to allow water to migrate. This is a very important issue.

Mr Dale Wilder: I think that anticipated the question I had, in that the freeze will not avoid the problem of water pressure. The water in the pores could create major fractures that could be more of a challenge to the shafts.

Mr Brendan Breen: That actually shows a typical arrangement for the lining and various lining requirements. In advancing the center of the zone, we are allowing a zone of sealing and there will be series of sealing processes, that we have set a good base seal and the tightening will occur behind the lining. We anticipated that at the base of the freeze, quite rightly, there is a risk of some micro-fracturing. In fact, the lining takes account of that by going some distance into Brockram strata. Brockram strata is about 60 to 70 meters thick in this area. That gives us sufficient space to be able to get a good quality seal. From an operation point of view that is extremely important, because the last thing we want to do particularly in the middle of a delicate study on hydrogeology, is to have high water leakage into the shaft. But again, the quality of shaft and lining is very good and we can get down to very low inflows. This is again down to costs. What we find in a porous flow situation with a high flow resistance, as sandstone rock will have, by pumping grouts in some around 1 – 1.25 times the hydrostatic pressure you can get a good settlement of grout in the voids in and around the lining zone.

Mr Göran Bäckblom: We will be very interested to see how site characterization will be done with this approach. You must do some special characterization program instead of using the draw-down to characterize the repository.

Mr Brendan Breen: I think that what is important, and I am moving outside my zone of technical appreciation, is that there are two distinct water-flows in relation to the upper sands and the BVG. In fact, there is no evidence of mixing, that we can find in the repository zone. We talked yesterday about setting seals. I think I do not want to pre-empt our scientists.

Mr Christian Sprecher: On one of your view graphs you showed the relatively large area you were planning to investigate from your Rock Characterization Facility. I suppose that you are planning to gather a data-set on which the later layout of the repository will be based. I would be interested to know how you are planning to investigate this relatively large area from underground.

Mr Brendan Breen: I think that the principal investigation from below ground will be concentrated on one major fault zone in the first place. In fact, one of the first pieces of work will be to look at this. The zone of investigation is one of about 400 – 500 meters wid and really we consider it to be a practical limit for diamond drillings. What we will have to do at stages along the way is to take stock of where we are and what our conditions are. Probe-drilling will give us a good start in relation to knowing what the conditions are. The advantages here are that by doing probe-drilling for those positions, we are not puncturing the interfaces with the sediments in the same way we would do with a series of surface holes. By going for small diameter holes by putting a series of holes with sealing, we are maximising the information gain with a minimum of disturbance. If you looked at the block of rock, most of the holes that will go in, will be somewhere along the route of the drivage. There is a requirement for us to be able to know what the block contains. The full scheme for it has not yet been fully designed. I think it will be wrong to say that at this stage we have every element designed.

Mr Christer Svemar: What rate of sinking of the shaft are you assuming?

Mr Brendan Breen: We are looking to be able to sink the shafts in about 2 1/4 years. I think that the important thing is that the key expense for shaft sinking is during the period of freeze.

Mr Jean-Michel Hoorelbeke: Since you are obliged to freeze for shaft sinking, don't you expect major difficulties for the excavation of an accessing line to the repository?

Mr Brendan Breen: I think that the first thing in looking toward the establishing of the decline, we need better information to be able to establish what we require to freeze. It certainly will not be a feasible proposition to freeze a decline over the distances involved. A section of a decline was frozen at Selby, Gascoigne Wood Drifts, in order to be able to get access through running sands. We currently do not have sufficient geological information to be precise about the access route. I think that it may be a combination of freezing near to the surface, where we are expecting difficulties, and section grouting with grout bandages for the rest of the drainage through potential water-bearing sandstone. There are a lot of techniques available to magane water.

4.3 EXPERIENCE WITH GROUTING AT AECL RESEARCH

Maria Onofrei, AECL Research, Pinawa, Manitoba, Canada
Malcolm Gray, " " " " "

ABSTRACT

Atomic Energy of Canada Limited is undertaking a research and development program on cement-based grouts for possible use in sealing an underground nuclear fuel waste disposal vault. Silica fume and superplasticizer were added to a finely reground sulphate resistant portland cement to produce a durable, low-permeability grout that would penetrate very fine fissures in granitic bedrock.

The superplasticizer additive permits very low water-cement ratio grouts (W/C less than 0.6 by mass) that exhibit no segregation or bleed. The silica fume additive contributes to improved chemical stability and leach resistance of the grout. The developed grout has been injected into granitic rock at AECL's Underground Research Laboratory (URL) in Canada and at the NEA/OECD Stripa Facility in Sweden. No problems were encountered in the field trials in mixing, handling or pumping of the grout. The injected grout produced only a very limited geochemical signature in the groundwater and appears capable of penetrating micro-fissures in the granite with apertures of less than 20 m.

4.3.1 INTRODUCTION

The Canadian Nuclear Fuel Waste Management disposal vault concept focuses on the disposal of nuclear fuel waste in corrosion-resistant containers at 500-1000 m depth within a granitic pluton in the Canadian Shield. The overall concept relies on both engineered and natural barriers to limit the environmental impact of radioactive materials released from the waste. The engineered barriers include concrete bulkheads strategically placed within the backfilled emplacement rooms, access tunnels and the shafts. Grouting will likely be required to treat the concrete/rock interface as well as the excavation damage zone within the rock. Moreover, widely-spaced, hydraulically-active fracture zones and faults are known to be present within plutons in the Canadian Shield and are expected to be encountered at a disposal site. Wastes will not be placed within such zones, but the fracture zones are expected to be encountered in the access drifts and shafts. Also grouting will be required to enhance seals and improve overall barrier pillar performance at strategic locations relative to fracture zones.

The grouts proposed for use in a vault must be shown to be durable over very long periods of time. They must resist leaching and not significantly alter groundwater chemistry. A grout material with a low hydraulic conductivity is essential. The ability of the grout to predictably penetrate very fine fissures within the rock is particularly important. Furthermore, sufficient understanding of grout performance must be gained to ensure that the technology is available to achieve and quality assure the desired overall engineering performance.

Atomic Energy of Canada Limited has developed the Underground Research Laboratory (URL) in Manitoba to undertake in-situ experiments related to the disposal vault concept in a realistic geological and hydrogeological environment (Figure 4.3-1). Cement-based grout materials and appropriate grouting methods are being investigated in the laboratory and in-situ at the URL. This paper describes the grout materials and their properties, as well as the field techniques and practical engineering aspects of the selected grout and grouting operations. Results and observations from the field trials at the URL and the NEA/OECD Stripa Facility in Sweden are presented.

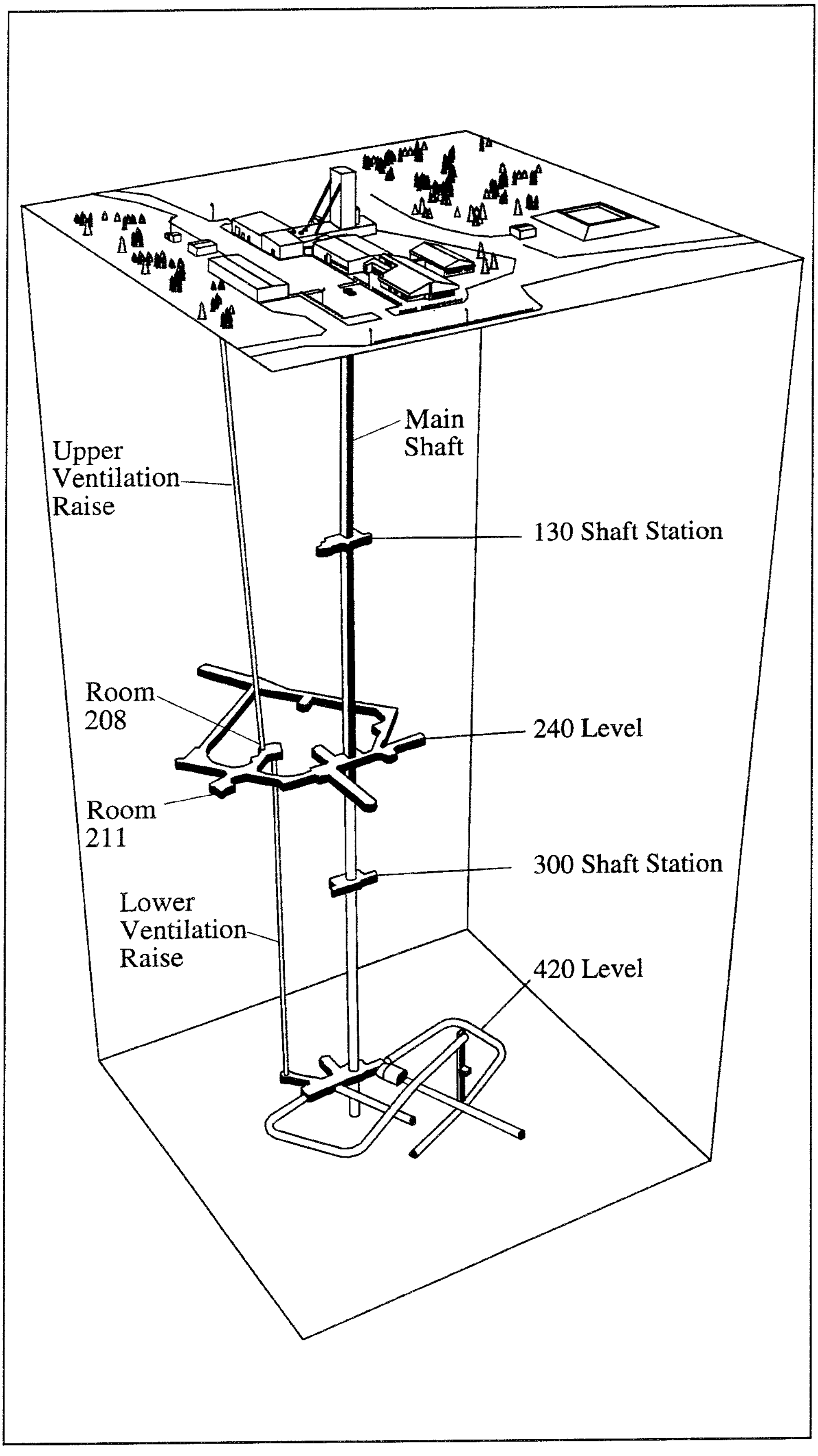


Figure 4.3-1. *Isometric view of the URL.*

4.3.2 GROUT MATERIALS

Extensive research and laboratory testing were undertaken to investigate the detailed properties and expected performance of alternative grout materials and mix proportions. The objective of this program was to select a reference grout that would best meet the requirements and objectives for vault sealing and to document the properties and behaviour of both the unset and hardened grouts.

Three types of cement were considered in these investigations (Table 4.3-1):

- Sulphate-resistant portland cement (Canadian Type 50) indicated that Type 50 cement is probably most appropriate for use in granitic rock in the Canadian Shield, where some groundwaters contain sufficient quantities of sulphates to be considered aggressive. This cement is also the most chemically stable portland cement type.
- Extensive cement (Canadian Type K): while portland cement grouts injected into saturated fracture zones should not significantly shrink during setting and hardening, the use of expanding cements may be advantageous in special sealing applications. They may improve the contact at the seal-rock interface and thereby limit flow.
- Micro-fine MC-500: a commercially available slag cement with extremely fine gradation.

Since penetration of very fine fissures was of interest, cements with finer particle size distribution were considered. Both the sulphate resistant and the expansive cement were investigated at their normal Blaine fineness as well as after regrinding (Figure 4.3-2, 3). The MC-500 cement is extremely fine and no regrinding was necessary.

4.3.3 OBSERVATIONS AND RESULTS

4.3.3.1 Mixing and handling

The grouts mixed easily and thoroughly in the high-shear colloidal mixer (Figure 4.3-4), and retained their fluidity for in excess of one hour after mixing. Even after one hour in the holding tank, when the temperature of the grout was becoming noticeably hot, the grout remained pumpable. This is important when considering large-scale field applications.

Laboratory studies had utilized an industrial blender for mixing small batches of grout. As shown in Table 4.3-2, the Marsh Cone times for the 0.6 water-cement ratio mixes obtained in the laboratory agreed well with those obtained in the field using the high-shear colloidal mixer. There was less agreement for the 0.4 water-cement ratio mixes incorporating smaller proportions of superplasticizer. Specifically, a laboratory-mixed grout with 0.75% superplasticizer was too thick or viscous to flow through the Marsh Cone – yet the same mix made in the high-shear colloidal mixer had a cone time of 150 s and was easily pumped. Other mixes developed in the laboratory exhibited varying grout viscosities depending on temperature of the ingredients, chemistry of the mixing water, mixing energy and mixing duration. Thus, relative to viscosities, variations may occur when extrapolating laboratory experience to field applications.

Pumping of the grout for injection into the holes similarly proceeded without incident. Increased grout penetration was achieved by the "Bean" piston pump in comparison

Table 4.3-1. Chemical composition of cements studied.

Chemical constituent	TYPE OF CEMENT Microfine MC-500 mass %	Type K mass %	Type 50 mass %
SiO_2	30.0	18.5	21.6
Al_2O_3	11.2	5.0	3.1
TiO_2	0.6	0.3	0.2
P_2O_5	0.2	0.1	0.1
Fe_2O_3	1.3	2.9	4.0
CaO	47.2	59.7	61.4
SrO	0.1	0.1	0.0
MgO	5.4	4.2	4.4
Na_2O	0.2	0.2	0.4
K_2O	0.4	1.1	0.5
SO_3	3.0	5.7	2.1
Loss on ignition	0.5	2.8	1.3
TOTAL	**99.9**	**100.7**	**99.2**

Table 4.3-2. Grouting materials: URL trials.

Hole No	Batch No	W/C*	SP** (%)	Vol (L)	Marsch Cone (time, seconds) Lab	Field	Density (Mg/m^3) Act	Theor
GH1	1	0.6	1.25	137.6	42	38	1.72	1.72
	2	0.6	1.25	137.6	42	40	1.78	1.72
	3	0.6	1.00	137.4	47	41	1.70	1.72
	4	0.6	1.00	137.4	47	42	1.73	1.72
GH2	1	0.6	1.25	137.6	42	41	1.62	1.72
	2	0.6	1.25	137.6	42	42	1.65	1.72
	3	0.6	1.25	137.6	42	41	1.65	1.72
	4	0.6	1.25	137.6	42	46	1.64	1.72
	5	0.6	1.25	137.6	42	46	1.61	1.72
HC9	1	0.6	1.25	137.6	42	45	–	1.72
	2	0.6	1.25	137.6	42	46	1.64	1.72
	3	0.6	1.25	137.6	42	43	–	1.72
	4	0.4	1.50	108.4	68	65	1.69	1.91
	5	0.4	1.25	108.2	84	57	–	1.91
	6	0.4	1.25	108.2	84	73	–	1.91
	7	0.4	1.15	108.1	132	73	1.73	1.91
	8	0.4	1.00	108.0	200	73	1.71	1.91
	9	0.4	0.75	107.7	no flow	150	–	1.91
	10	0.4	0.75	107.7	no flow	142	–	1.91

* W/C : water cementitious materials ratio (by mass).
** SP : superplasticizer content (by mass solids).

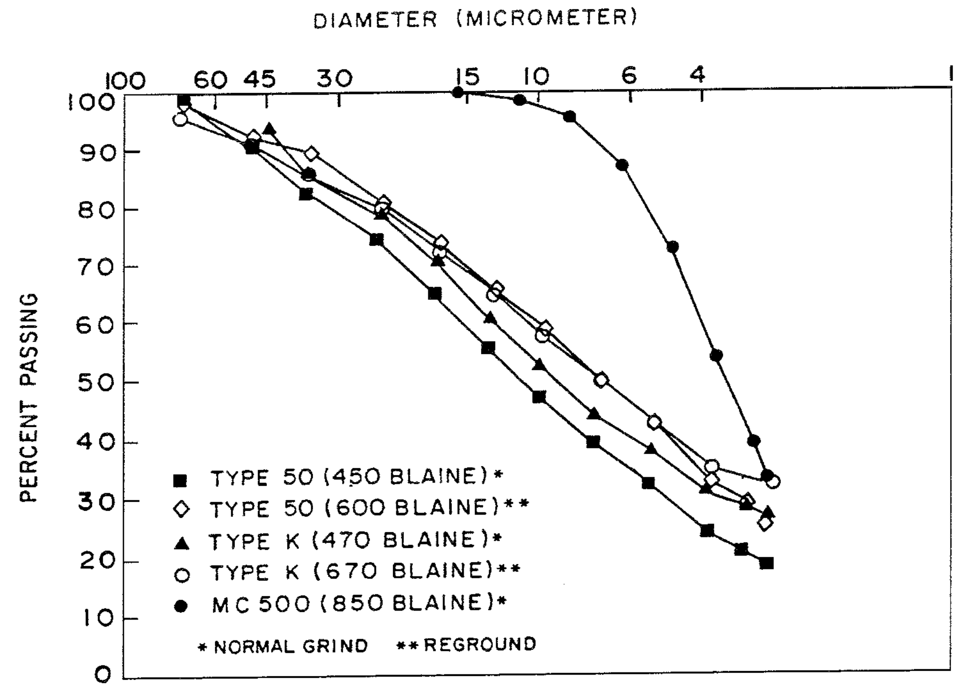


Figure 4.3-2. *Grain size of cements studied.*

to that by the Moyno pump. The Stripa studies utilizing the dynamic injection pump also confirmed that vibration enhances grout penetration and retards refusal.

Setting times for the grout mixes used in the field trials agreed well with those observed in the laboratory – although after "final set", as defined by the Vicat test, a further 24-h period was necessary before the grout had sufficiently hardened to permit reaming of the hole. The overall setting times (Table 4.3-3) were more affected by the cool rock temperatures than by the small amount of superplasticizer incorporated into the mix.

4.3.3.2 Hydrogeological and geochemical effects

Single-hole step withdrawal hydraulic conductivity tests were performed in each grout hole before grouting. After the grout had hardened, the hole was reamed out and the tests repeated. The results of these tests are summarized in Table 4.3-4. Grouting the rock through the holes GH1 and GH2 to refusal successfully reduced the hydraulic conductivity by one and two orders of magnitude, respectively, to approximately 10^{-9} m/s. The hydraulic conductivity of the fracture zone around hole HC9 was reduced from 2.1×10^{-6} m/s to 1.6×10^{-8} m/s. This may be related to the fact that HC9 was not grouted to refusal.

Geochemical monitoring was performed throughout the grouting period. The results obtained were not totally conclusive since mine water, contaminated by cement, had been injected into the formation during the drilling and water pressure testing activities associated with the program. However, it was clear that the geochemical signature of the grout injected into the formation was very limited both in lateral extent and in duration. For example, a rise in pH was not detected in the nearby holes

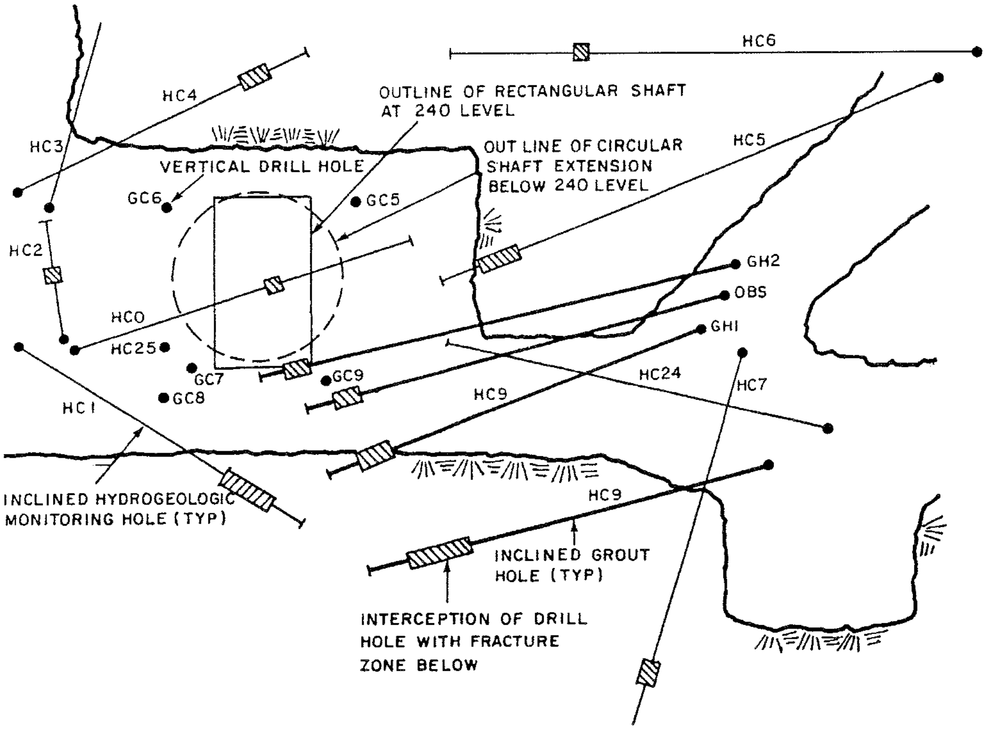


Figure 4.3-3. Plan of drill holes for URL grout trials.

Table 4.3-3. Grout mix properties.

	Mix 1	Mix 2	Common
Water-cementitious materials ratio (by mass)	0.4	0.6	3.0
Silica fume content (SF)*	10%	10%	nil
Superplasticizer content (SP)**	1.5%	1.25%	nil
Marsh Cone time (sec)	68	43	35
Bleed or segregation	nil	nil	16%
Initial set at 8°C, Vicat Needle (h)	10	19	>43
Final set at 8°C, Vicat Needle (h)	25	29	–

* : 10% SF = 9 parts cement, 1 part SF by mass
** : superplasticizer in liquid form, 42% by mass solids

$$1.5\%\ SP = \frac{\text{mass of solid SP}}{\text{mass of cement + SF}} \times 100$$

Table 4.3-4. URL hydraulic conductivity test results.

	GH1	GH2	HC9
Before grouting			
Transmissivity (m2/sec)	$3.2x10^{-7}$	$4.2x10^{-7}$	$1.5x10^{-5}$
Equivalent single fracture apterture (2b) (μm)	83.4	91.2	298
Hydraulic conductivity (m/sec)*	$4.0x10^{-8}$	$7.0x10^{-8}$	$2.1x10^{-6}$
After grouting			
Transmissivity (m2/sec)	$1.0x10^{-8}$	$5.7x10^{-9}$	$1.1x10^{-7}$**
Equivalent single fracture aperture (2b) (μm)	26.3	21.7	58**
Hydraulic conductivity (m/sec)*	$1.2x10^{-9}$	$9.5x10^{-10}$	$1.6x10^{-8}$**

Note: * hydraulic conductivity is based on the thickness of the total fracture zone as observed in the drill logs.

** HC9 was not grouted to refusal

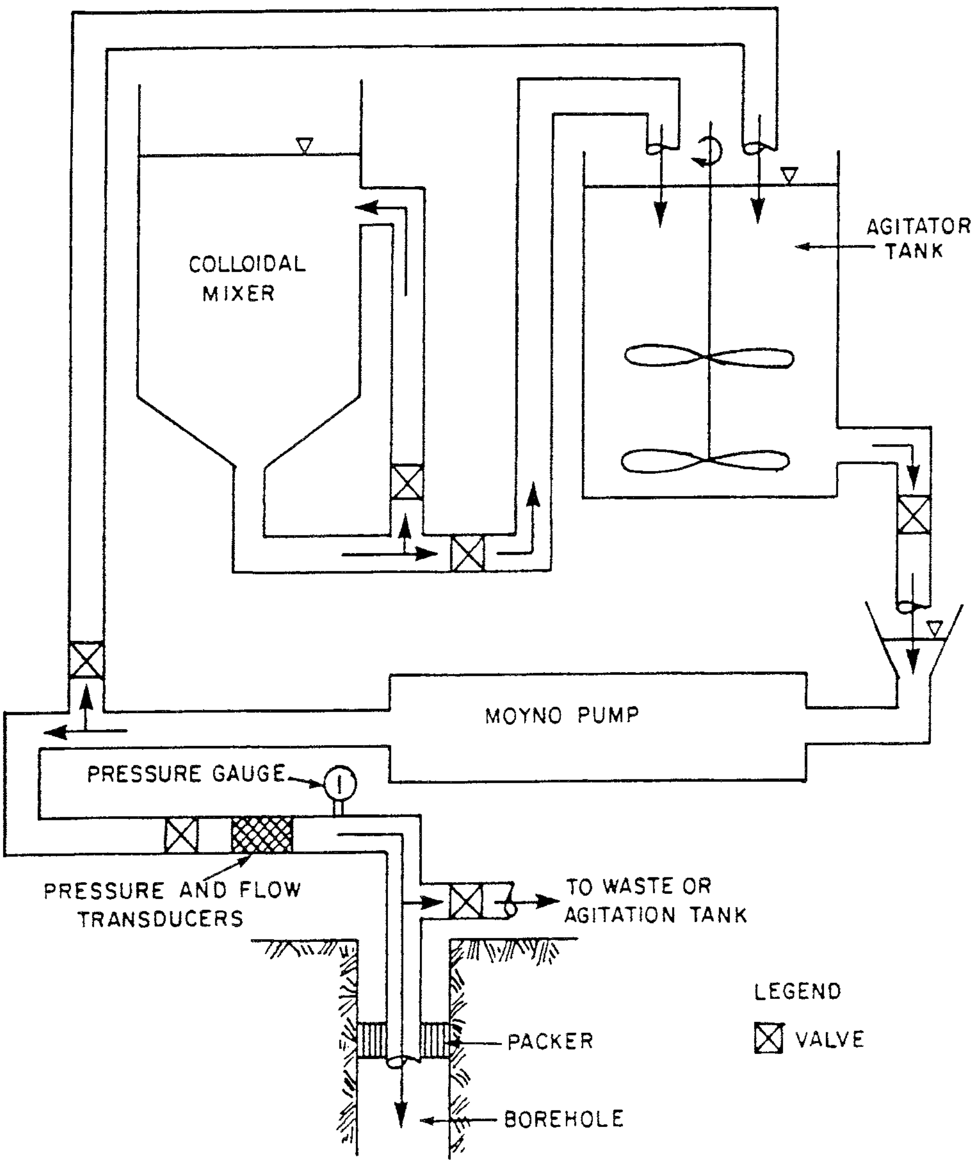


Figure 4.3-4. Schematic arrangement of grouting plant.

except in those where the grout itself had migrated. This lack of a significant geochemical signature or plume is considered to be due largely to the fact that these grouts exhibit no segregation or bleeding during or after injection.

The data is not proof that a continuous grouted barrier of such low hydraulic conductivity was achieved at significant distances from the grout holes. However, the data does suggest that a carefully controlled program with superplasticized grout and closely spaced holes has the potential to develop an effective barrier without a significant geochemical signature or plume.

4.3.3.3 Shaft wall mapping

During 1987 August, the URL shaft deepening exposed a portion of the fracture zone into which the grout has been injected. Grout penetration of fractures was observed up to 1.0 m from the borehole. The thicker groutfilled zones closer to the borehole reflected where fault gouge had been removed by a special washing technique prior to grouting.

A sample of grouted rock was recovered from the excavation. Thin sections were prepared from this sample and were examined under a microscope. This examination showed no evidence of voids or segregation within the grout. The grout totally conformed to the irregular boundaries of the rock, including even the complex individual grain boundaries. Moreover, the grout had penetrated into the individual fractures within the rock, including mineral cleavage planes and micro-fractures with apertures of less than 20 μm.

The penetration of micro-fissures of such small aperture is at variance with conventional hypotheses which suggest that the smallest aperture that can be effectively grouted is three times the maximum grain size of the cement used for the grout. On this basis, the reference grout would only penetrate fractures whose apertures were greater than approximately 200 μm. One possible explanation for what was observed in the URL thin sections is that the grain size of approximately 60Z by weight of the cement used is finer than 10 μm. In addition, the mean particle size of the silica fume is only 0.1 to 0.2 μm.

4.3.4 SUMMARY

Atomic Energy of Canada Limited is investigating low water/ cement ratio superplasticized cement grouts, containing silica fume. Alternative cements, both normal grind and finely reground, were studied and a reference grout using reground sulphate-resistant portland cement, 10% by weight silica fume and superplasticizer was adopted for use in field trials at the Underground Research Laboratory and at the Stripa Facility in Sweden. The reference grout exhibits a hydraulic conductivity of the order of 10^{-14} m/s, which is about the same as that of the in-situ intact granite.

The use of superplasticizers enables the water/cement ratio to be reduced to 0.4 yet still retain the low viscosity needed for pumping and injection into the rock. The unset grout exhibits no segregation or bleeding, an important consideration relative to grout efficacy and groundwater contamination. The grout appears capable of penetrating microfissures in the granite with apertures of less than 20 μm.

Research and development of superplasticized grout is continuing, with an emphasis on both engineering properties and long-term resistance to leaching. These studies are also being extended into concrete for use in plugs and bulkheads. All indications to date suggest that superplasticized grout with silica fume, and the similarly based concrete, will find application in sealing a nuclear waste disposal vault. These materials will also find application in a wide range of other civil engineering projects.

Questions to Mrs Maria Onofrei

Mr Dale Wilder: My concern is whether or not you have any microbial action that might be solved by using superplasticizer.

Mrs Maria Onofrei: I do not know if there is such a super-plasticizer, but if you expect any microbial activities in the disposal, I think the one that could affect the long term performance will be the one that will increase the pH in the groundwater. That is the only one that can effect the long term performance of the cement.

Mr Göran Bäckblom: Before the choice of the cement, have you discussed if we should have brittle or ductile grouting material?

Mrs Maria Onofrei: I would like to have a ductile one, but I do not think it is possible.

Mr Håkan Stille: For your grout you are using some kind of non-linear Newtonian model and you are measuring some apparent viscosity. Why don't you use the Bingham model and how do you measure the viscosity?

Mrs Maria Onofrei: At that time it was more convenient for the selection, but in the new development we try to improve the compositions for features for the grouting experiment we intend to do at ÄHRL. At the time when we selected the mixtures, as I indicated, silica fume was considered one of the most reactive lining materials. Since then we have got more information from France, which has practically abandoned the use of mixtures with silicon fume. As you know, some of the silica fume is a by-product from silicon iron alloys. Oxidation is not fully completed in this process. Some particles of silicon metal, in contact with water with relatively high pH, will produces hydrogen. We are trying to develop new grouts now to replace silica fume. I would not replace silica fume with "fly-ash" because it is inconsistent from a chemical point of view and also you have to introduce a new step in your preparations to refine this material.

Mr Marc Blanchin: You demonstrated that your grout has an ability to self-heal. I have a question about hydration of material. Does it include a shrinkage or not? Does it have a constant volume? Did you reduce the volume?

Mrs Maria Onofrei: No. Constant volume. Hydration reaction does not increase the volume. You produce new products which have higher volumes. We have seen some slight increases. We put this grout for 80 days at about 85°C in a dryer and of course it did shrink considerably. We took that dry grout and put it in water. It did have the capability to regain its original volume. In fact it extended a little bit more than originally.

Mr Marc Blanchin: So, it means that in small cracks the grout will be able to create stress?

Mrs Maria Onofrei: I do not think so, but we were talking yesterday about the use of Portland cement. Those phases react with sulphate and produce a so called interact phase which can create problems for the grout if it is not confined. The pressure of these growing crystals of the mineral was measured to be 40 MPa. That has to be an extremely large quantity to do that. Those kinds of reactions are to be avoided. If it is not confined it can crack the construction. This is overcome by added inner cement-iron.

I should like to indicate that we have seen that the grout penetrated 2 – 3 meters.

Mr Håkan Stille: Did you measure the effect of the high pressure water jets? The distance of the penetration of the water jet?

Mrs Maria Onofrei: The only place where we were able to observe it, was the one near the shaft wall. It was merely the volume of gravel and sand and clay that came out of the fracture zone. There is no way to trace it back to its emplacement. So we did not have a direct measurement.

Mr Jukka-Pekka Salo: There is a hydrogen production caused by the silicon metal. Have you made experiments to verify? Did it crack the grout?

Mrs Maria Onofrei: No, because it is a very slow reaction. It is an accumulation of hydrogen. It is not a kind of explosion or a large amount of gas. Unfortunately it was difficult to quantify that. Every time you take a sample from silica fume it reacted with incoming water. You can produce different amounts of hydrogen to know how much of the silicon particles it took.

Mr Jukka-Pekka Salo: But is it your point that it is impossible to predict a long term behaviour of such a material? Why are you afraid of hydrogen production if it does not cause you a problem?

Mrs Maria Onofrei: From my point of view it is not a material problem, the gas does not affect the grout chemically. The concern is the construction of the repository. If you have bubbling hydrogen accumulated in some media you risk an explosion in certain situations.

4.4 DEMONSTRATION OF THE EMPLACEMENT OF A CLAY ENGINEERED BARRIER IN A GRANITE FORMATION

A Lajudie – Michel Dardaine, Commissariat à l'Energie Atomique, Fontenay-aux-Roses, France

This work has been carried out as part of the European Atomic Energy Community's R&D programme on Management and Storage of Radioactive Waste (shared cost action).

4.4.1 INTRODUCTION

This is a presentation of the test carried out in France at Fanay-Silord intended to demonstrate the feasibility of the emplacement of an engineered barrier for backfilling and sealing a shaft for the disposal of vitrified wastes. In the scenario chosen, which is close to that developed in the framework of other community studies, it is planned to stack twenty canisters containing vitrified wastes in a 30 metre-high 1 metre-diameter shaft bored in granite.

The surrounding clay engineered barrier, which consists of high-density compacted clay bricks, is placed between the waste and the granite.

To reduce the uncertainty in the evaluation of the minimum residual clearances for satisfactory backfilling of the disposal shaft and to find an acceptable compromise between ease of handling and clearance in the emplacement, a full scale test is being carried out.

4.4.2 DESCRIPTION OF SITE

The Fanay-Silord site is a at La CROUZILLE in the north of LIMOGES and is in a former COGEMA uranium mine.

The workings (Figure 4.4-1) are reached by two inclined access galleries, one to the upper level 44 m below the surface and one to the lower level 40 m below.

There is therefore a 37 m vertical thickness of granite to be bored, which taking into account the lower plug and the fissured section at the top of borehole, leaves a height of at least 30 m in the sound granite.

The upper level where the experimental work is carried out was enlarged and arranged so as to have a recess, at this level, that is precisely above the lower gallery.

A diagram of the working zone, 20 m long, 7 m wide and 4.5 m high, is shown in Figure 4.4-2. The primary ventilation of the workings is ensured by a 1 m diameter hole connecting the base of the gallery to the surface, outside of the working zone.

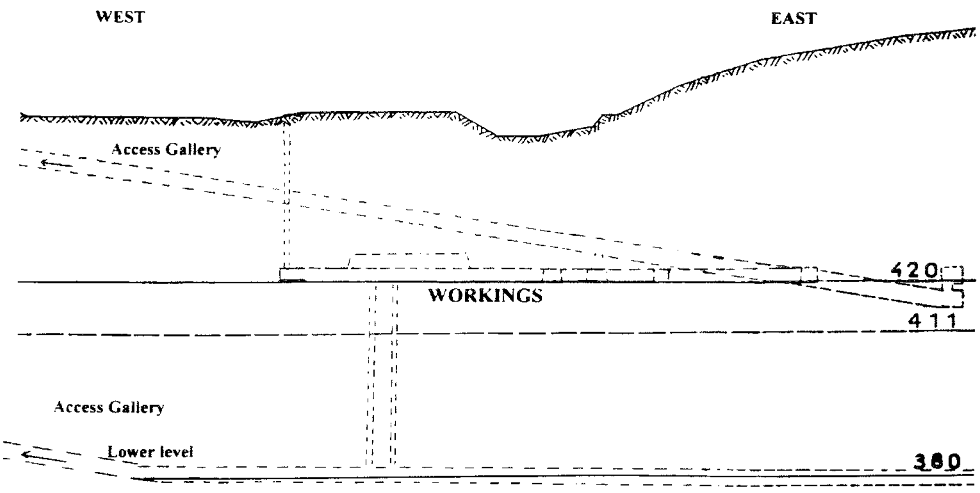


Figure 4.4-1. *The Fanay-Silord site.*

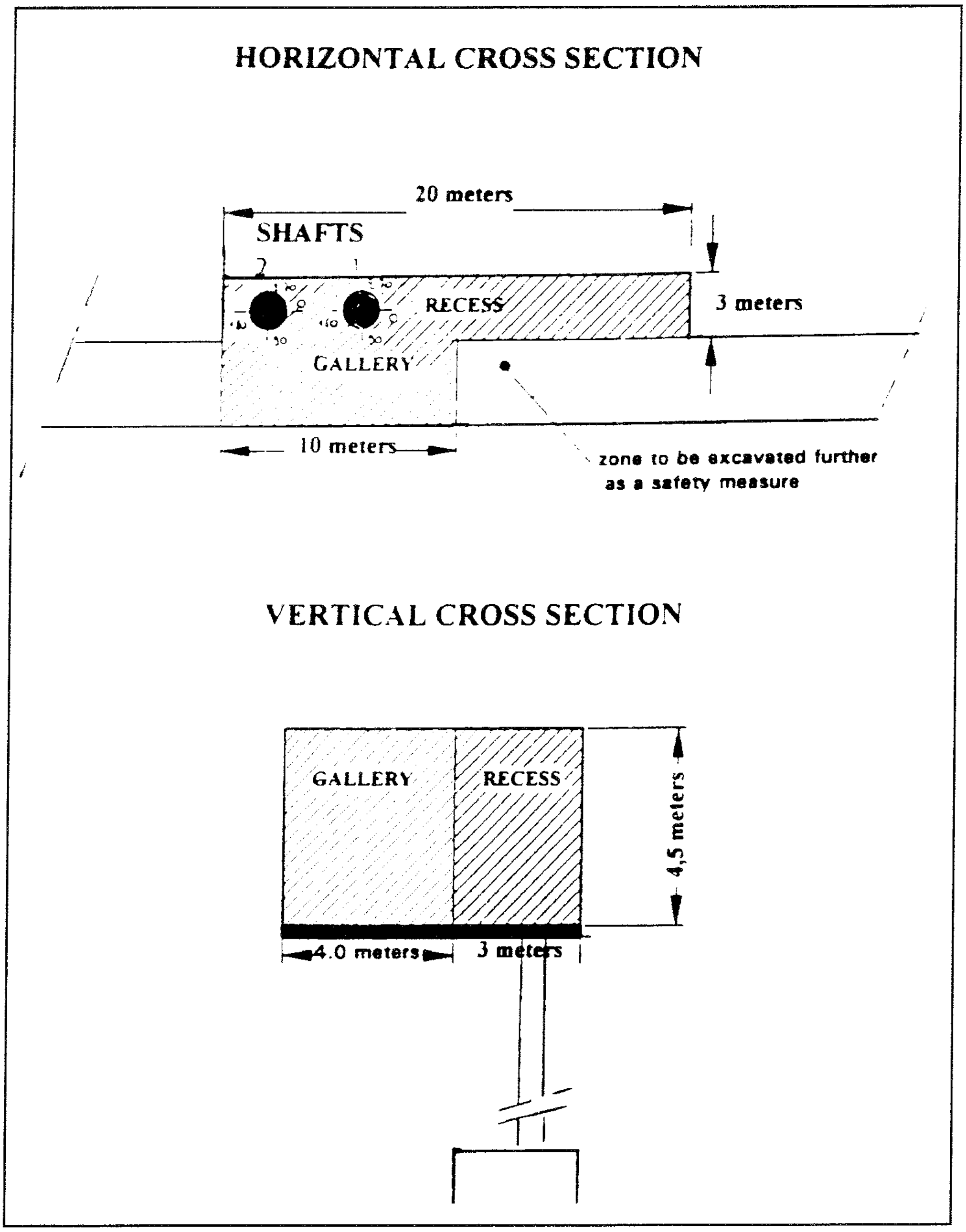


Figure 4.4-2. The working zone at Fanay-Silord.

4.4.3 CALCULATION OF DENSITY (Figure 4.4-3)

Weight of the emplaced clay (taking account of the 10% sand present):

In 40.331 metric tons of the dry mixture, there are

> M = 36.298 metric tons of dry clay and
> m = 4.033 metric tons of sand representing
> Volume of the sand $V_S = 1.493\ m^3$.

Note: The baskets were weighed before their emplacement and the weight of the dry clay they contained was subtracted. This gave a value of 36.280 metric tons, in excellent agreement with the above value.

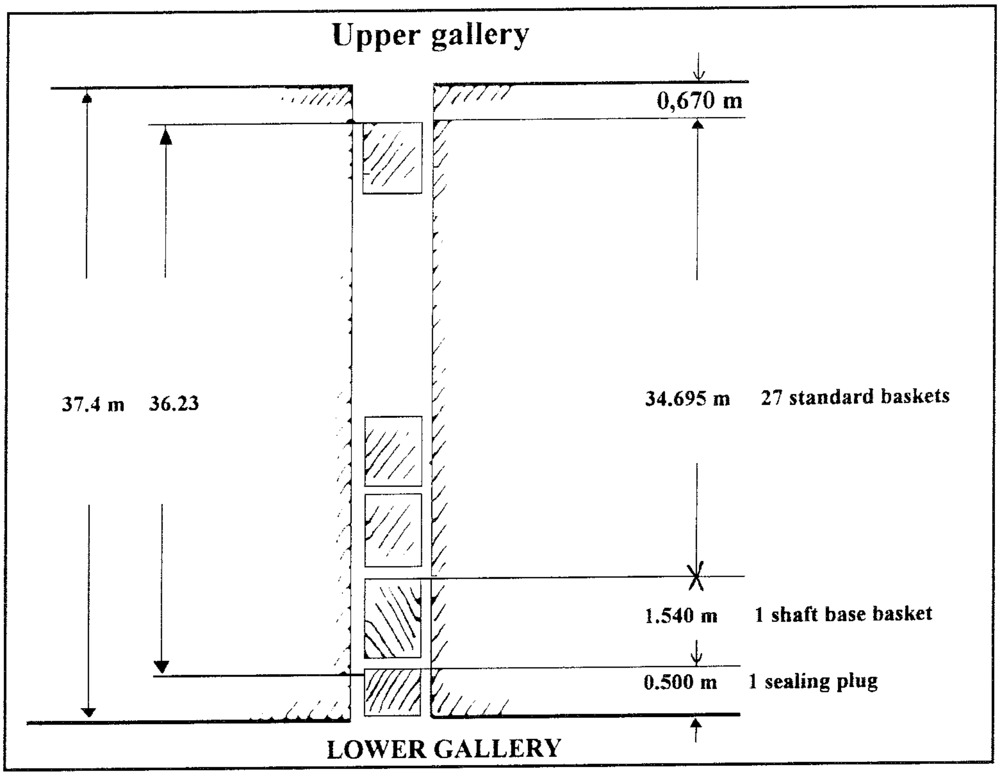


Figure 4.4-3. *Specification of the stacking.*

– *Number of baskets:*	*28*
– *Total height of the shaft:*	*37.1 m*
– *Useable height of the shaft:*	*36.23 m*
– *Useable volume of the shaft:*	*28.10 m^3*

– Volume of the metal baskets:

Standard basket:

weight:	73 kg
specific gravity:	7.8 (or density: 7.8 kg/dm^3)
volume:	9.36 dm^3

Shaft-base basket:

weight:	94 kg
specific gravity:	7.8
volume:	12.05 dm^3

– Total volume of the baskets:

$(27 \times 9.36) + 12.05 = 265$ dm^3 $\qquad V_B = 0.265$ m^3

– Volume occupied by the canister:

$28 \times 0.194 = 4.423\ m^3$ $\qquad V_C = 5.423\ m^3$

– Volume of the voids:

$28 \times 0.020 = 0.560\ m^3$ $\qquad V_D = 0.560\ m^3$

– Volume of the shaft to be filled:

$V_E = 28 \cdot 10^3$

– Free volume in the shaft to be filled by the swelling of the clay:

$V = V_E - V_S - V_B - V_C + V_D = 21.473\ m^3$

– Dry density of the clay in the shaft:

$$d = \frac{M}{V} = \frac{36.928}{21.473}$$

$d = 1.69$

4.4.4 CONCLUSION

The dry working density of the clay as emplaced in the shaft, determined taking the residual voids into account, is 1.69. Provided that a leak-tight plug is put in place at the head of the shaft, this value guarantees the satisfactory filling of these voids on water inflow.

In fact, the curve below shows that this density makes it possible to attain a swelling pressure of about 6 MPa after complete saturation of the clay (Figure 4.4-4). In addition, this provides the engineered barrier with a hydraulic conductivity less than that of sound granite at a depth at 500 m (10^{-12} m/s) thus avoiding the "drainage effects" (Figure 4.4-5).

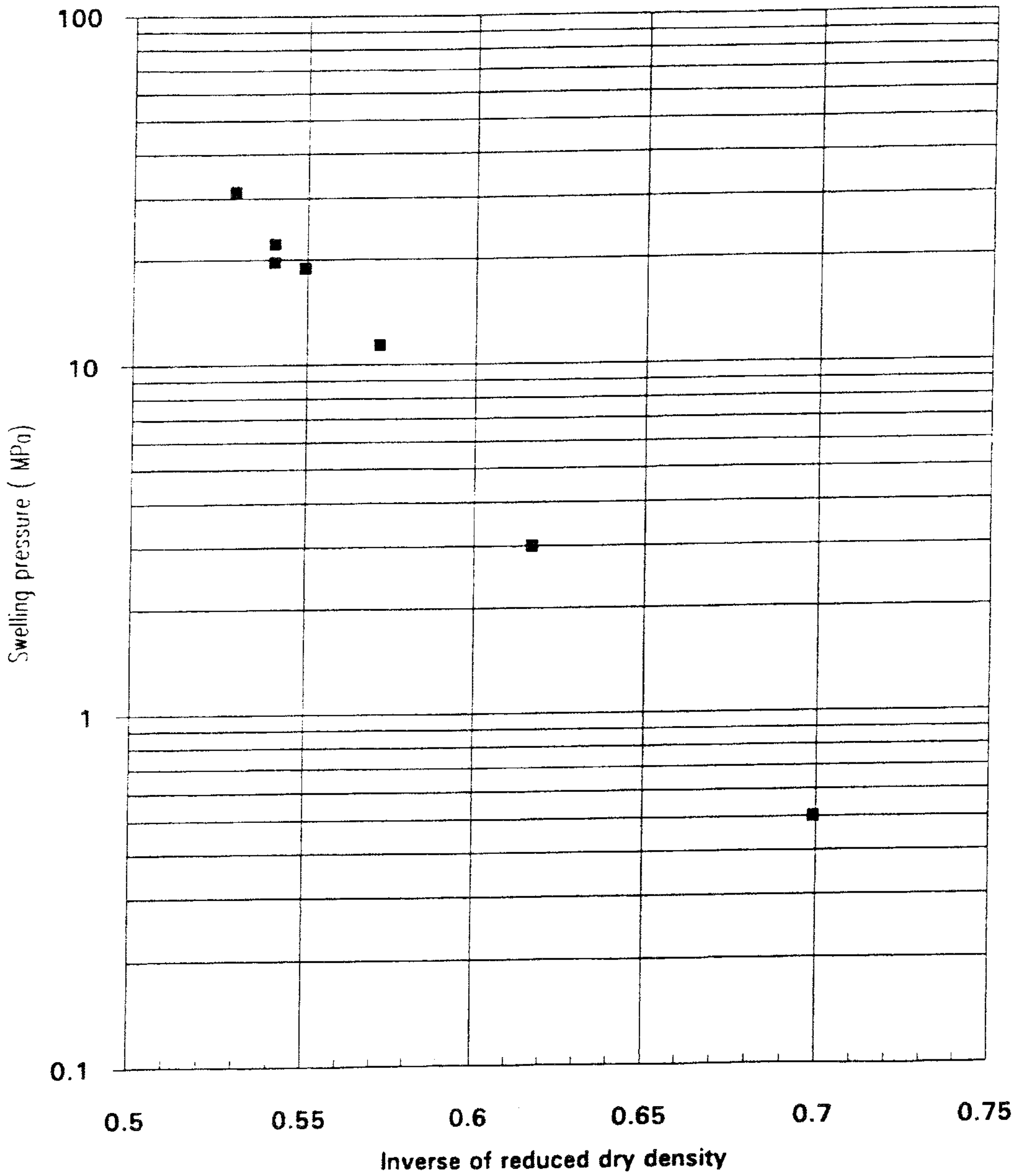


Figure 4.4-4. *Swelling pressure (MPa) of pure FoCa clay versus inverse of reduced dry density.*

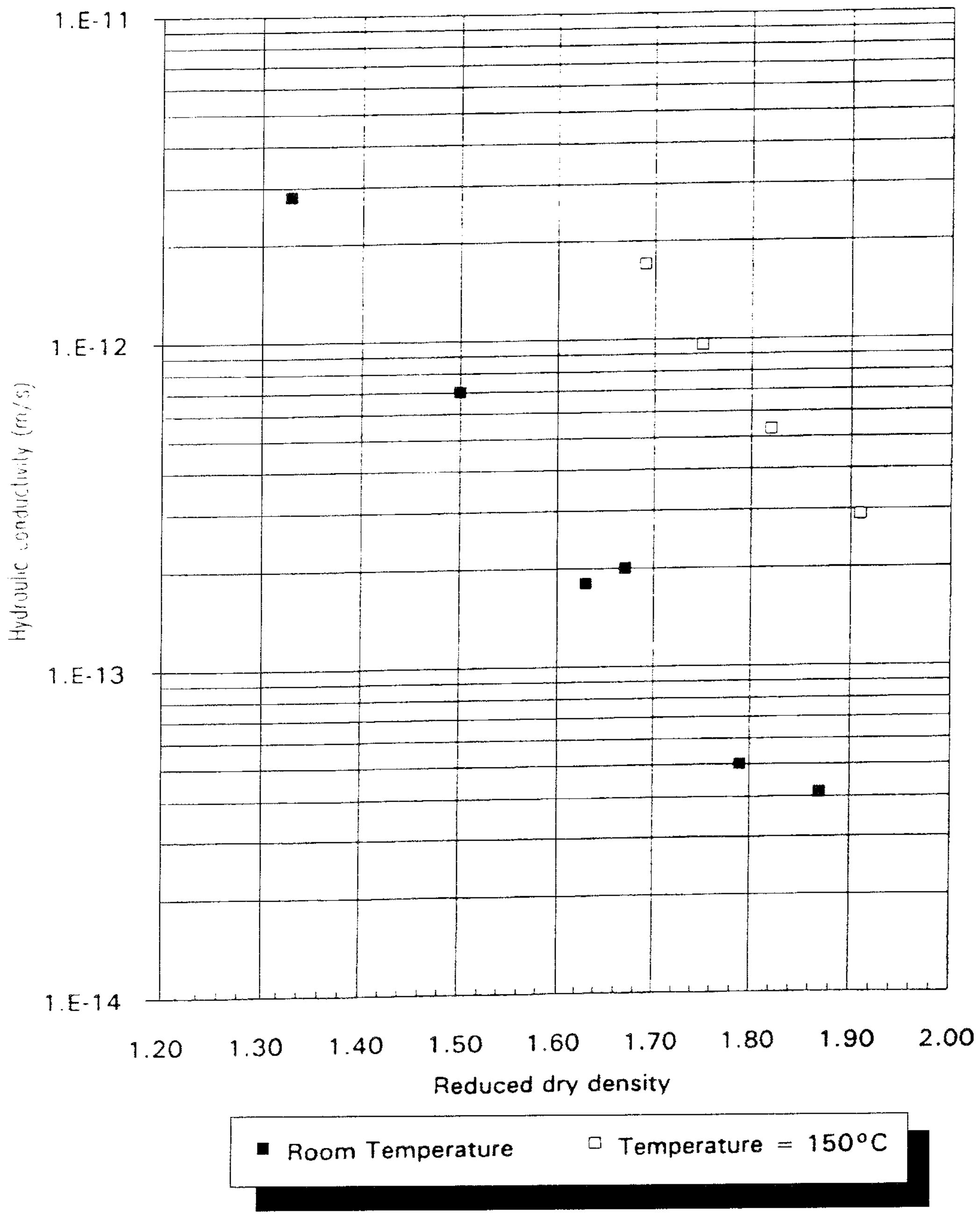


Figure 4.4-5. *Hydraulic conductivity (m/s) of FoCa clay versus reduced dry density.*

Questions to Mr Michel Dardaine

Mr Brendan Breen: Was there any reason why you did not blind bore your shaft?

Mr Michel Dardaine: No, the main purpose of this demonstration test was only to prove the backfilling process of the storage shaft.

Mr Christer Svemar: What was the reason for the 10% quartz sand in the mixture? Why not 100%?

Mr Michel Dardaine: It is not fundamental, perhaps because the aspect of the blocks is better with small quantities of sand but it does not modify the properties.

Mr Gunnar Gustafson: Was that only manufacturing emplacement? You did not do any measurement of what happened with it, when you had got it there. The extension you got?

Mr Michel Dardaine: We did not measure. It was only a demonstration test.

Mr J-M Hoorelbeke: We have performed other experiments in the lab for the study of the hydro-mechanical and thermal-mechanical properties of this engineered barrier. So this field-test was focused on feasibility of emplacement. Then we had of course other tests for physical and chemical behaviour. It is only a part of the over-all program in this field.

Mr Paul Thompson: I noticed that the walls were fairly rough. We used button bit cutters, similar to those you used, on our upper ventilation raise at the URL, and a rough surface also resulted. We used disc cutters on the lower ventilation raise, and these produced a smooth wall surface. If it is desirable for you to achieve smoother walls, you may also wish to try disc cutters.

Mr Michel Dardaine: We know this. It was not really the main issue of the test.

4.5 SEALING OF EXPLORATION BOREHOLES IN AND AROUND A POTENTIAL RADIOACTIVE-WASTE DISPOSAL SITE

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J F Ouvry, Department of Geotechnical Engineering, BRGM, France

This work has been carried out as part of the European Atomic Energy Community's R&D programme on Management and Storage of Radioactive Waste (shared cost action).

ABSTRACT

The problem of sealing boreholes in and around a potential radioactive-waste disposal site is studied The materials used for backfilling must have the necessary properties for ensuring a lasting protection. Any sealing products must be adapted to the conditions of the natural environment, taking into account specific parameters such as local fracture patterns, hydraulic conductivity and the void distribution of fractures. A sealing slurry must thus be tailor-made for each case in order properly to backfill any fractures.

4.5.1 INTRODUCTION

The burial of radioactive waste in a deep geological environment aims at its immobilization and isolation from the human environment. This must be for as long as is necessary to ensure that any possible release of radioactive substances from the storage facility will not cause unacceptable radiological risk, even in the long term */AEN/OECD, 1988/.*

The creation of underground facilities requires preliminary geological, hydrogeological, geotechnical and geochemical investigations that are mostly carried out from exploration boreholes. Such holes are drilled from the surface or from the drifts and shafts that constitute underground research laboratories.

The fundamental safety concepts for such a storage facility require that all boreholes, which form preferential pathways for fluids and any radionuclides, must be permanently backfilled or sealed.

This paper presents a discussion of the sealing of exploration boreholes in crystalline rock. It is part of R&D work carried out for a project funded by the European Community that studies safety problems associated with radioactive-waste disposal sites.

4.5.2 PROPERTIES REQUIRED FOR THE SEALING OF BOREHOLES

The sealing of boreholes should lead to the best reconstitution of the initial geological environment before it was disturbed by drilling. In order to fulfil their purpose in a durable manner, sealing materials must have certain properties that were described by *Meyer et al. (1980)* and *Come (1984)*, and in the *Mott, Hay & Anderson* report of *1984*.

Such properties are:

- a physico-chemical stability in relation to the host rock and circulating groundwater;
- sufficient deformability to follow later deformation of the rock without rupture;
- a very low permeability;
- a very long life ensuring that the material will fulfil its functions for a very long time.

Table 4.5-1.

Material Group	Principal Attributes	Material types	Ranking	Comments
Spoil	b, e	Crystalline rock argillaceous rock saliferous rock	1	Principal attributes are geochemical and physical compatibility with the host environment on-site availability at nominal basic cost. However technical and economic aspects of spoil processing may preclude their use in some instances.
Clays	b, c, d, e	illites, kandites, palygorskites, smectites, vermiculite, chlorites	1	Geological evidence suggests clays may exhibit excellent longevity. However, certain types may undergo alteration depending on chemical/physical conditions e.g. Na-bentonite Ca-bentonite. Diagenetic changes are precluded by temperature-pressure design constraints.
Zeolites	d	analcime, chabazite, erionite, clinoptilolite, ferrierite, mordenite, phillipsite	3	Potentially reactive except under evidence of longevity for naturally occurring pozzolanas, although the influence of ambient repository conditions. Dehydratation precluded by temperature-pressure design constraints.
Pozzolanas	d, e	natural pozzolanas* pulverised fuel ash	2	Good geological and archaeological evidence of longevity for naturally occurring pozzolanas, although the influence of ambient repository conditions requires investigation. The long-term stability of PFA is likely to be comparable to or better than natural varieties.
Hydraulic cements	b, e	Portland cements polymer cements hydrothermal cements	2	Favourable archaeological evidence exists although further research is requires to determine optimum formulation. Hydrothermal cements are inherently more stable than Portland cements. Longevity of polymer-baed cements is doubtful(R).
Mineral/aggregates	a, e	anhydrite, natural aggregates, crushed aggregates	2	Geological evidence suggest excellent longevity provided mineralogy is matched with that of the host formation.
Metals and Metallic compounds		lead, copper, iron, magnesium, manganese	3	Longevity depends on geochemical environment. Some forms can be susceptible to groundwater transport.
Bitumens	b, e	natural bitumens, industrial bitumens	1	Geological evidence suggests excellent longevity, mainly due to lack of affinity to water.
Chemical grouts	b, e	silicate based* acrylic-based* formaldehyde-based* lignin-based epoxy-based*	R	Silicate grouts likely to be stable based on comparable geological evidence (1). However, longevity of all organic chemical grout is suspect and requires fundamental research (R).
Carbons	a, b, e, d	graphite charcoal	1	Graphite and charcoal are essentially inert in the geochemical sense.

Attributes	Ranking
a good, heat transfer properties b low permeability c favourable chemical buffering properties d favourable retention properties e favourable mechanical properties * includes more than one variety	1 documented evidence of geochemical stability over geological periods of time 2 documented evidence of stability over significant time-intervals 3 some doubt as to long-term stability under certain physico-chemical conditions R fundamental uncertainties concerning longevity

To these properties we can add further requirements if the material will have to seal boreholes very close to the disposal facility:

- stability in the face of temperature increases;
- a retention function for radionuclides.

Table 4.5-1 shows a list of potential materials that might be used in a sealing slurry, drawn up in 1984 by Mott, Hay & Anderson Consulting Engineers. Another criterion, technical this time, plays a role in the choice of sealing materials. This is related to the methodology of implementation, combine with the possibility of processing large linear volumes at a reasonable cost.

4.5.3 DESCRIPTION OF THE CRYSTALLINE MEDIUM TO BE SEALED

To be able to reconstruct the initial characteristics of the rock before they were disturbed by drilling, it is necessary to obtain a geological and structural understanding of the medium. This is done through a petrographical and structural survey, with oriented drill cores and geophysical logging of the boreholes.

The aim of such a survey is to understand the grouping of mapped joints and fractures */Bles and Feuga, 1981/*, each of which is defined by strike and dip. For each group it is then possible to define a morphology, extension in continuity, spacing, width, and possibly the type of infilling and the amount of open space. Such parameters are not always easy to quantify, but their study must be systematic for all borehole-sealing work.

Fractured crystalline rock commonly contains circulating water that follows fractures, in particular those that are opened and interconnected. Water tests in boreholes are useful for determining certain fracture characteristics, and are in fact the only satisfactory method for the determination of the most important parameter: hydraulic fracture opening */Billaux, 1990/*. To complete the definition of a fractured environment, BRGM engineers have developed laboratory methods that can be used for quantification of the void space between fracture walls */Gentier and Billaux, 1989/*, based on the taking of wall casts according to the following principle:

Voids that are left between the two walls are filled with a coloured silicone resin. This cast is then studied under transmitted light and the resulting image is digitized with a camera. Among the various shades of grey, the darkest areas indicate the thickest voids. (Figure 4.5-1).

The conversion from shades of grey into thickness is done with a calibration wedge, which results in a map of fracture openings. This map then is used to plot a void distribution in the shape of a cumulative histogram (Figure 4.5-l). Each fracture group will thus be defined by a hydraulic opening determined from borehole tests and by a statistical distribution of voids between the walls. These are essential parameters for defining the mixture of potential components of a borehole sealant.

4.5.4 THE CONCEPT OF BOREHOLE TREATMENT

Over the past decade, numerous laboratory studies and in-situ pilot tests have investigated the conditions for use, and the advantages and drawbacks of certain sealing materials or methods */Esnault and Ouvry, 1993/*. Borehole sealing methods or materials must be adapted to the rocks intersected by the borehole. For instance, for those

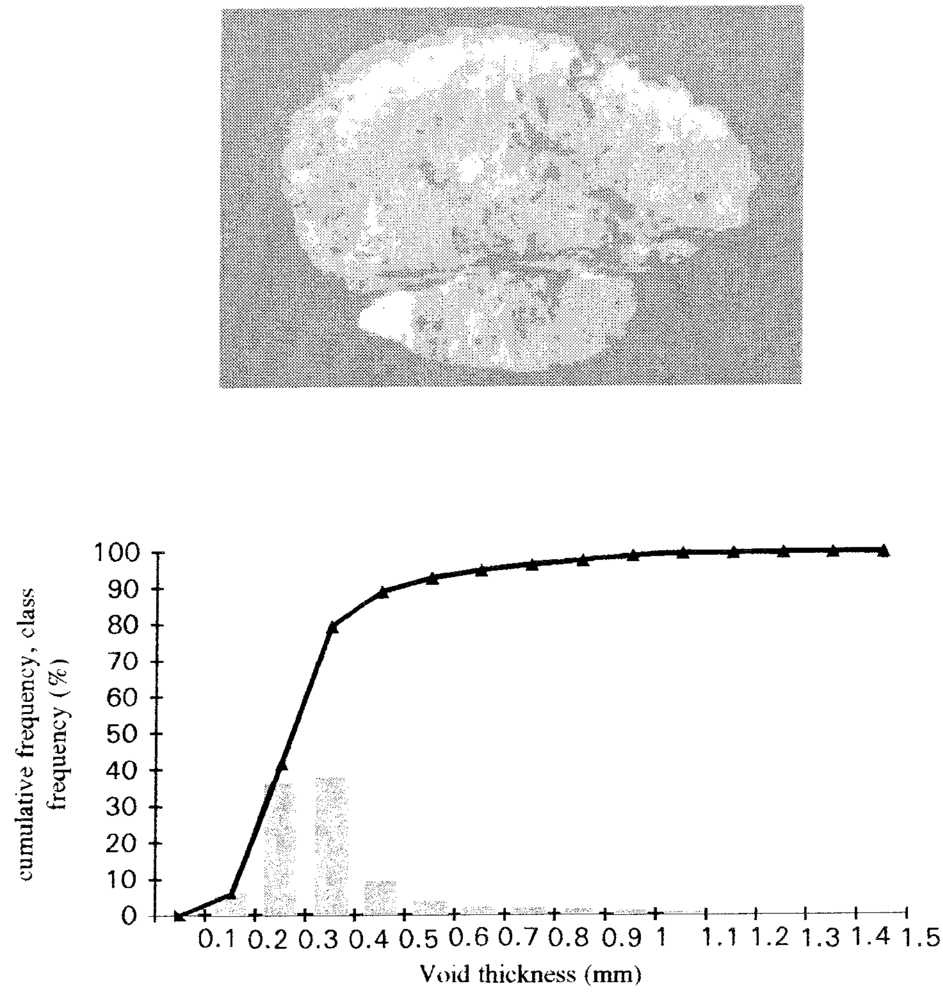

Figure 4.5-1. Image of a fracture-wall cast, and the resulting normal and cumulative histograms of void thicknesses in the fracture wall.

parts of a hole that traverse an unfractured environment, which is unlikely to be the site of fluid circulation, it is possible to use dry and compacted bentonite as a plug However, in borehole sections through fractured rock that may be subjected to potential fluid circulation, such bentonite plugs might be washed out */Boisson, 1992/.*

It is thus necessary to design a sealant that not only treats the volume surrounding the hole by plugging of the fractures, but also plugs the hole itself (Figure 4.5-2). The material to be injected for treatment of the volume surrounding the hole should be able to plug the voids between fracture walls in order to reduce fluid circulation towards the borehole. Such a sealant should combine certain of the potential materials mentioned in Table 4.5-1 and should have the final properties described above.

The grain size of the basic components of the injection mixture should thus be suitable for filling the void space defined by a certain thickness distribution. The properties of the borehole plug should include the ability to overcome incomplete fracture sealing; a cohesive material combining clay and cement should be a good mixture for slowing down erosion phenomena within the borehole.

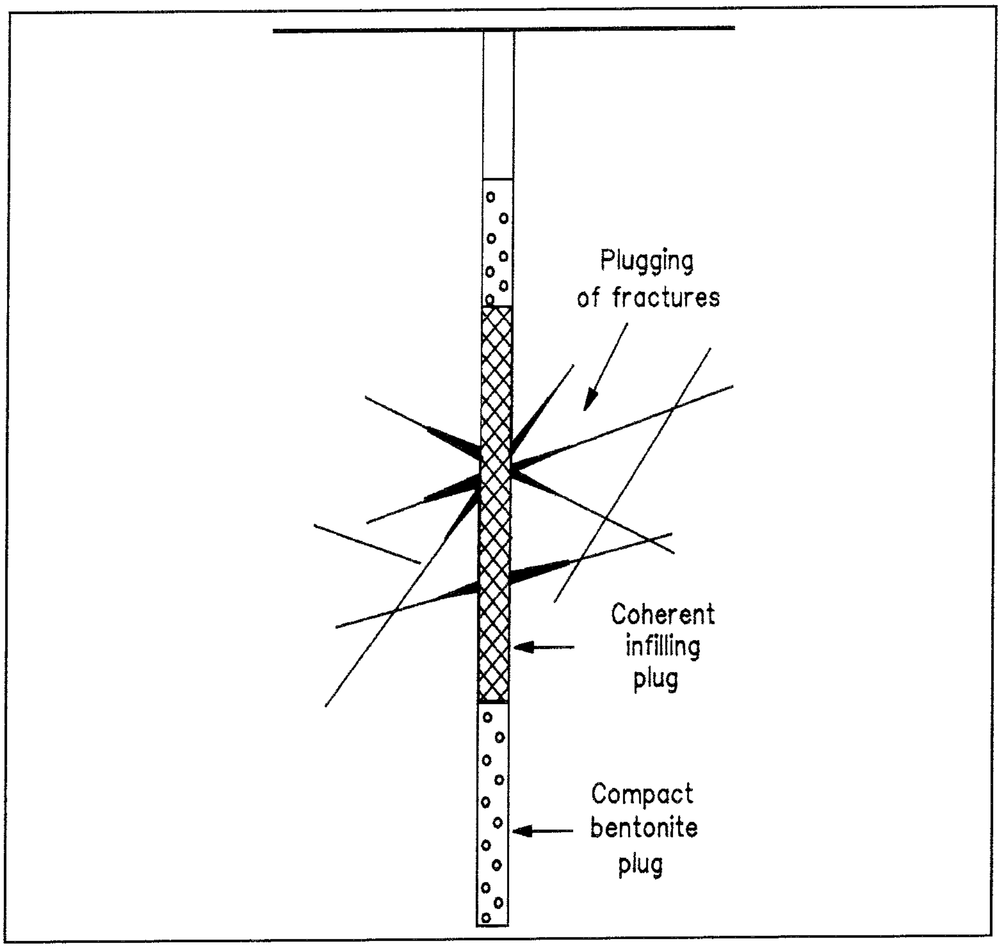

Figure 4.5-2. *Tailor-made borehole sealing.*

4.5.5 THE PLUGGING OF FRACTURES IN CRYSTALLINE ROCK

Fractures in crystalline rock can be plugged by injecting a granular suspension. The components of such suspensions must satisfy several criteria. These criteria may pertain to the materials and their physico-chemical interaction with the surrounding rock, or the behaviour of the granular suspensions of which they consist compared with the characteristics of the medium to be injected. Or they may pertain to the behaviour of the slurry in its solid state. The last point was discussed in section 2 above.

The component materials used for this study were selected on the basis of two criteria. The first is physico-chemical (see Table 4.5-1) and the second is related to granularity. The size of the grains of a slurry must be adapted to the thickness of the fracture to be injected. This means that all components have a fine grain size (Figure 4.5-3). The two bentonites selected are calcic, and a hydrocarbon phase may be added to certain slurries in the form of a bitumen emulsion, with a maximum grain size of 10 μm.

In cases where the injection aims at filling tiny fractures, the composition of a granular suspension will play a predominant role as the penetrative power of the slurry depends on its intrinsic qualities in the liquid state. The stability of the slurry

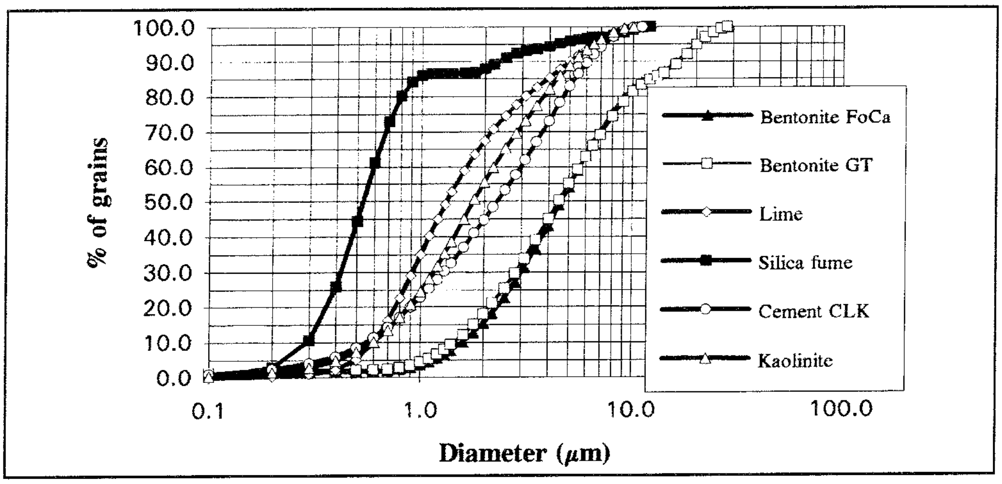

Figure 4.5-3. *Grainsize-distribution curves of sealants.*

will govern the simultaneous transport of its solid and liquid phases, which should not be disturbed by the decantation of particles. Loss of fluid will hinder the complete penetration of the suspension; when the slurry is kept under pressure part of its water will be expelled, causing a major modification in the composition, and thus the properties, of the remaining slurry. During the injection sealing of fractures in crystalline rock such water loss will mostly take place at the injection front, which thus will form a filtration surface.

Rheological properties are another very important parameter for good penetration. The flowage threshold should be as low as possible; after all, when the pressure applied to the slurry in a fracture at a distance "R" from the injection point drops below the flowage threshold, the slurry will stop moving. This pressure is related to the initial pressure that is applied at the point of injection.

Plastic viscosity associated with this threshold causes pressure loss that also hinders the movement of the slurry during injection.

The composition of the slurry should present optimum characteristics in the face of all these potential obstacles. A physical model was developed for studying the injectability of slurries. It permits measurement of suspension penetrability under constant pressure into fine fractures, whose width can be varied from 0.5 to 0.1 mm.

4.5.6 CONCLUSIONS

As part of the study of potential sites for the disposal/storage of radioactive waste in rock, it is necessary to plug all boreholes in a permanent fashion, in order to prevent fluid circulation from developing into a vector of spreading pollutants.

Such sealing operations must be based on an intimate knowledge of the geological environment to be treated, which will enable adaptation of the sealing process by choosing the correct component materials of the slurry with the correct grain sizes, and by selecting the appropriate technology to satisfy the requirements.

This conceptual study has been carried out as part of contract co-funded by the CEC, BRGM, SIF BACHY and Mott MacDonald, in the framework of an R&D programme on the management and disposal of radioactive waste. The study is supplemented by laboratory and field tests.

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Questions to Mr J F Ouvry, represented by Mr M Blanchin

Mr M Blanchin: The pump foreseen to be used to inject such a liquid grout is ordinary injection pump as the ones used by soil and rock grouting specialist companies, generally double piston mechanism.

Mr Jukka-Pekka Salo: So if you have boreholes that are 1.000 meters deep, do you think that you have any difficulties in getting the material to the bottom of the holes?

Mr M Blanchin: We don't think that there will be any insurmountable difficulty to inject at this depth provided adaptation of grout and/or adjustment of the injection pressure, for instance the grout could be "pushed" by another liquid more lightweight or by an air column.

The pressure can be measured at the point of injection in the bottom of the borehole. The injection is carried out between packers lowered in the borehole and located below and above the fissure to be treated.

Mr Jukka-Pekka Salo: Have you considered the situation where you have to transport the material 1 km from a point.

Mr M Blanchin: Due to the low viscosity, no blockage of the grout in the pipe is expected. Mixing of the grout is the know how of our partner SIF BACHY.

Mr Reijo Riekkola: I like to ask SKB how you should treat your boreholes, relying on the results that were gained in earlier phases?

Mr Christer Svemar: Our know-how is primarily what we obtained in the Stripa experiments. We have not reevaluated or taken into consideration new techniques yet.

4.6 ANALYSIS OF CLAY CORE DAM BEHAVIOUR

B Felix – J M Hoorelbeke – M Ollagnier,
ANDRA, France

C Imbert – C Gatabin,
CEA, France

4.6.1 FUNCTION OF THE DAMS IN A DEEP DISPOSAL SEALING SYSTEM

According to a recent French law on management of long-lived radioactive wastes, two underground laboratories are to be built in deep geological formations. Based on their characteristics, ANDRA will evaluate of the acceptability of a deep geological repository and intends to achieve this evaluation within the 13 years specified by the law. The laboratories will offer an opportunity to investigate the properties of the host rock and to study the performance of the sealing system. A deep geological repository could not be ready in France before 2017, and its closure would take place about 50 years after that, or even later. Nevertheless, an evaluation of sealing system efficiency is required within the next 13 years. It will necessitate the adaptation of accessible technologies and even the development of new ones.

In hard rock the convergences of the opening are very limited, so no long-term mechanical densification of the backfilling material will occur. The long-term imperviousness of the sealing system would be mainly guaranteed by the dams. Their tightness would have to be maintained at the requested level from the moment they are built for the very long period of time covered by the repository time scale, which means during many thousand years. A back- up system could been designed, which would be composed of several dams built side by side. Different materials could be used for each of them requiring different technologies. This redundant configuration would provide reinforced reliability. One of the most promising dam concepts is based on the sealing properties of swelling clays.

4.6.2 THE CLAY CORE DAM DESIGN

It is composed of a high density swelling clay buffer, complemented by sand filters and by retaining structures placed at its borders. Due to the fact that the clay will occupy almost the entire available opening volume and that the host rock and the retaining structures will be rigid, the hydration of the high density swelling clay will result in a considerable swelling pressure, which will ensure the clay core stability under loads. At almost constant volume, the clay will preserve its low permeability and ensure tightness. Figure 4.6-1a represents a schematic clay core dam layout with dimensions obtained from the stability analysis.

4.6.3 SELECTION OF THE CLAY, THE CORE DESIGN AND ITS CONSTRUCTION

A reference clay has been chosen from among a selection of different swelling clays. A calcium smectite from a surface deposit located at Fourges Cahaignes in France was selected and is referred to as FoCa clay . Table 4.6-1 sums up its properties in two referred states:

- the initial state results from a compaction of the air-dried smectite powder with an energy much higher than the Modified Proctor one. Two processes are believed to provide this energy,
- the final state corresponds to the complete saturation of smectite within an opening volume 25 percent larger than the initial clay volume. These 25 percent correspond to an overestimation of the residual cavities resulting from the construction process. It appears that large volume expansion of the FoCa clay may induce a reduction in mechanical properties and an increase of water permeability, but to an extent compatible with a sealing function.

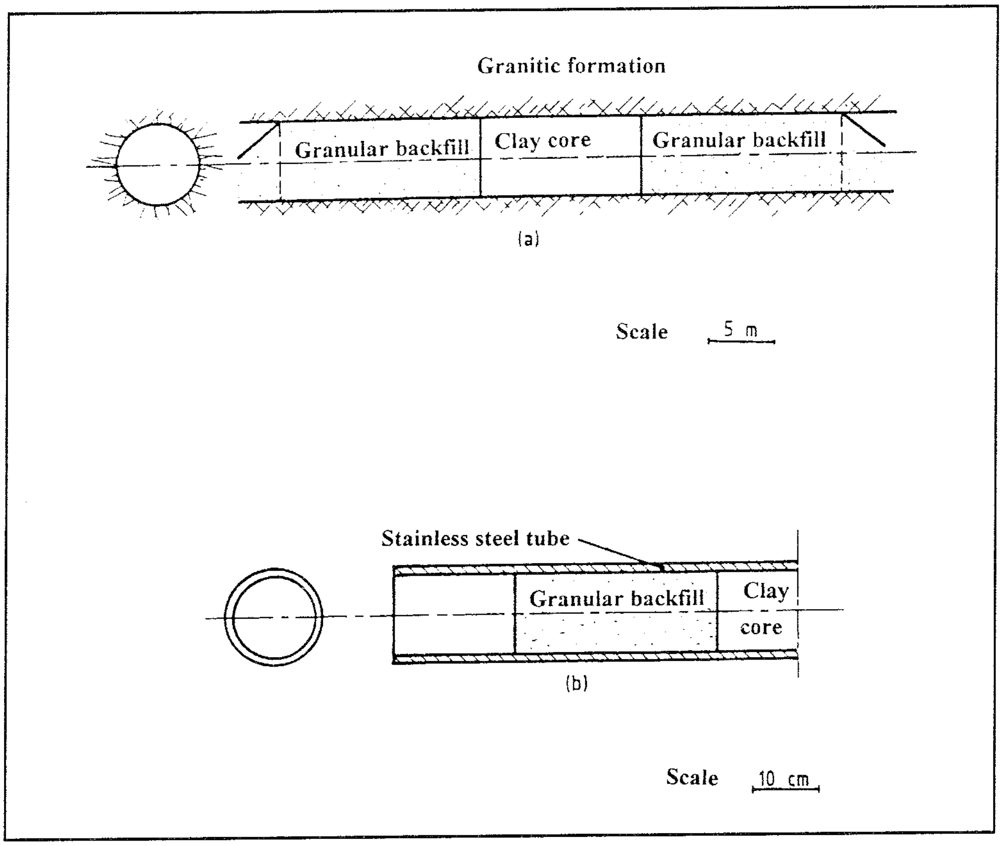


Figure 4.6-1. *Clay core dam schematic layout and its scale model*
a) dimensions issued from stability analysis,
b) scale model implaced in a stainless steel tube.

This expansion potential is related to the high density resulting from compaction at a given water content of the air-dried clay powder. For practical reasons it is impossible to depart from this water content, because it corresponds to the equilibrium with the usual atmospheric conditions.

Table 4.6-1. The properties of FoCa clay.

	Initial clay state	**Final state after swelling and complete saturation**
Water content	10%	27%
Dry density	1.85 g/cm^3	1.55 g/cm^3
Permeability	3 ·10-14 m/s	10-12 m/s
Thermical cond.	1.1 W/mK	< 1.2 W/mK
Cation exc. cap.	50 meq/100g	50 meq/100g
Mohr Coulomb Parameters	c = 2.5 MPa $\Phi = 25^o$	c = 0 MPa $\Phi = 10^o$
Swelling stress (const volume)	σ_S = 40 MPa	σ_S = 0.7 – 2 MPa

Swelling clays in general, because of their high plasticity index are very difficult to compact. Two specific methods have been developed, both of which involve substantial energy consumption

- Compacted clay is manufactured in the form of blocks of decimetre size. A power press uniaxially applies a static stress as high as 80 MPa, which results in a dry density of the clay of up to 1.8 g/cm^3 and a water content which is initially close to 10%. Subsequently, clay-based dams are built in the form of a block masonry.
- High compaction technology currently used to manufacture pellets out of granular materials as various as coal powder, aspirin, fertilizers, uranium oxide etc, has been applied to swelling clay. As a result, clay pellet density is more than 2.1. After crushing, then sieving, high-density clay aggregates are obtained with a uniformly distributed granulometry curve, which allows an in-situ material compaction with low energy consumption. This process has been experimented with in France and in Finland with various clays.

The duration of the hydration process is one of the main problems to be solved in this concept. The water outflow from the host formation will necessarily be very low because of the rock, which is high imperviousness, the reason it will be chosen, along with its probable desaturation at the end of the operating phase. Preliminary evaluations have shown that the process of hydration will last several decades, even if the pore pressure is supposed to be maintained at its final value on the host rock wall. This is due to the large core dimensions and to the low clay permeability. From this it can be concluded that the efficiency of clay core dams is either long-dated, or will depend on artificial hydration.

4.6.4 STABILITY ANALYSIS OF THE GRANULAR BACKFILLS RETAINING THE CLAY CORE

The retaining structures will be heavily compacted backfills. The granular material will be prepared from the excavated rock. Compaction by bumper provides higher density. The main problem lies in the completion of the work at the drift roof. A study of the feasibility of specific equipment designed in accordance with the repository sealing requirements has been initiated. A research programme is focused on the evolution of its granulometric characteristics under high stress levels (higher than 20 MPa) and of the associated rheological properties induced by grain rupture.

From analytical considerations based on the assumption of a rigid-plastic behaviour of the granular material, it can be deduced that a backfill as long as two and a half times the tunnel diameter is sufficient to ensure stability. Stress components $\sigma_{rr}(x)$ (normal) and $\pi_{rx}(x)$ (tangential) on the host rock wall smoothly decrease with the distance from the clay core, according to an exponential function given in formulae (1). The maximum value depends only on the clay swelling pressure ss and on the internal friction angle of the granular material Φ, r_o is the gallery radius.

$$\sigma_{rr}(x) = \exp \{- 2 \cdot x \cdot \tan\Phi / [(1 + 2 \tan^2\Phi) \cdot r_o]\} \; \sigma_s / (1 + 2 \tan^2\Phi)$$

$$\tau_{rx}(x) = \tan\Phi \cdot \exp \{- 2 \cdot x \cdot \tan\Phi / [(1 + 2 \tan^2\Phi) \cdot r_o]\} \; \sigma_s / (1 + 2 \tan^2\Phi)$$

Finite Element calculations of the clay core dam with granular backfills in a granite gallery have been carried out. According to them the dam stability is ensured, and the granular backfill rigidity is sufficient to limit clay core expansion. The granular material dilatancy at shear has little influence. The stress distributions of an exponen-

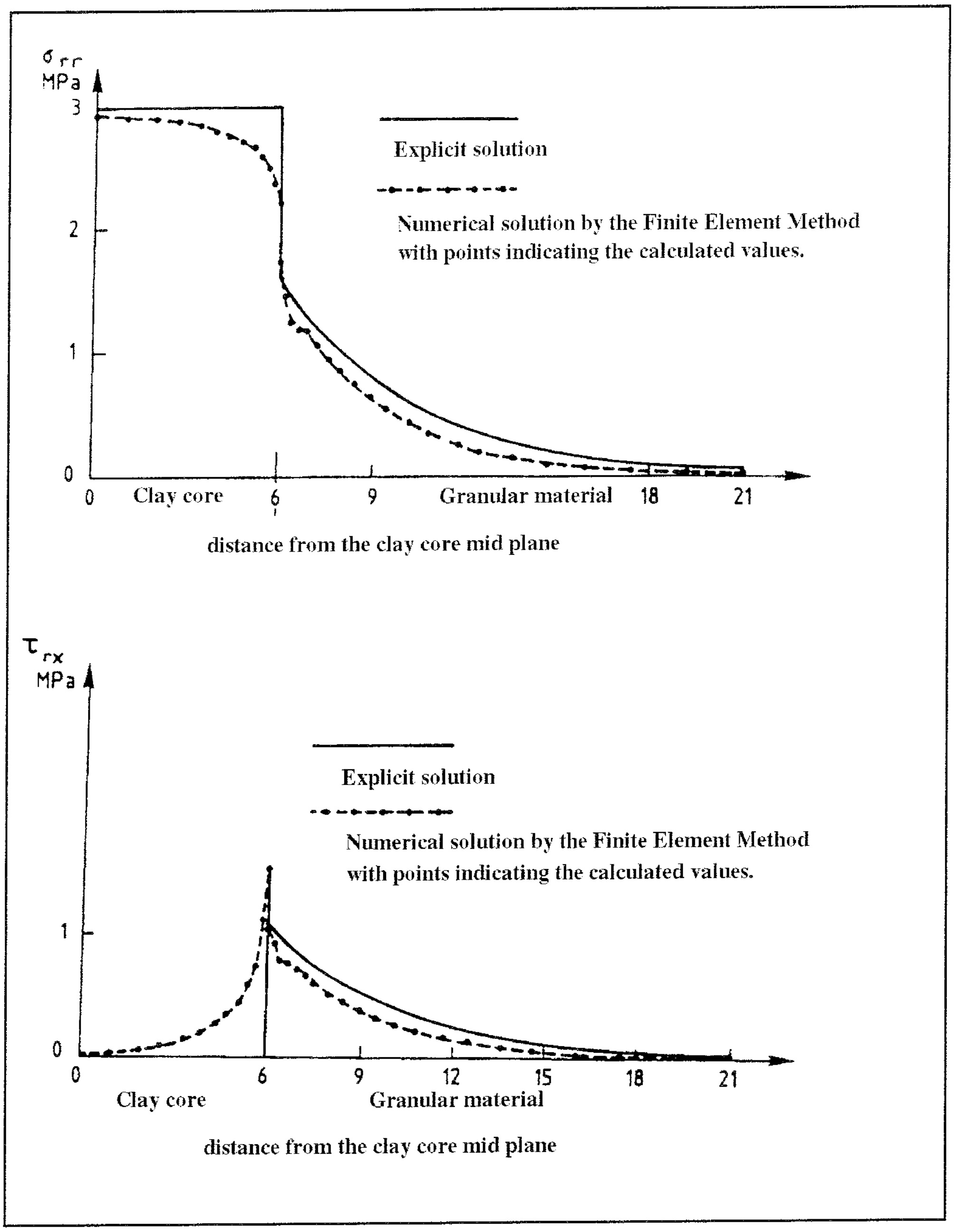
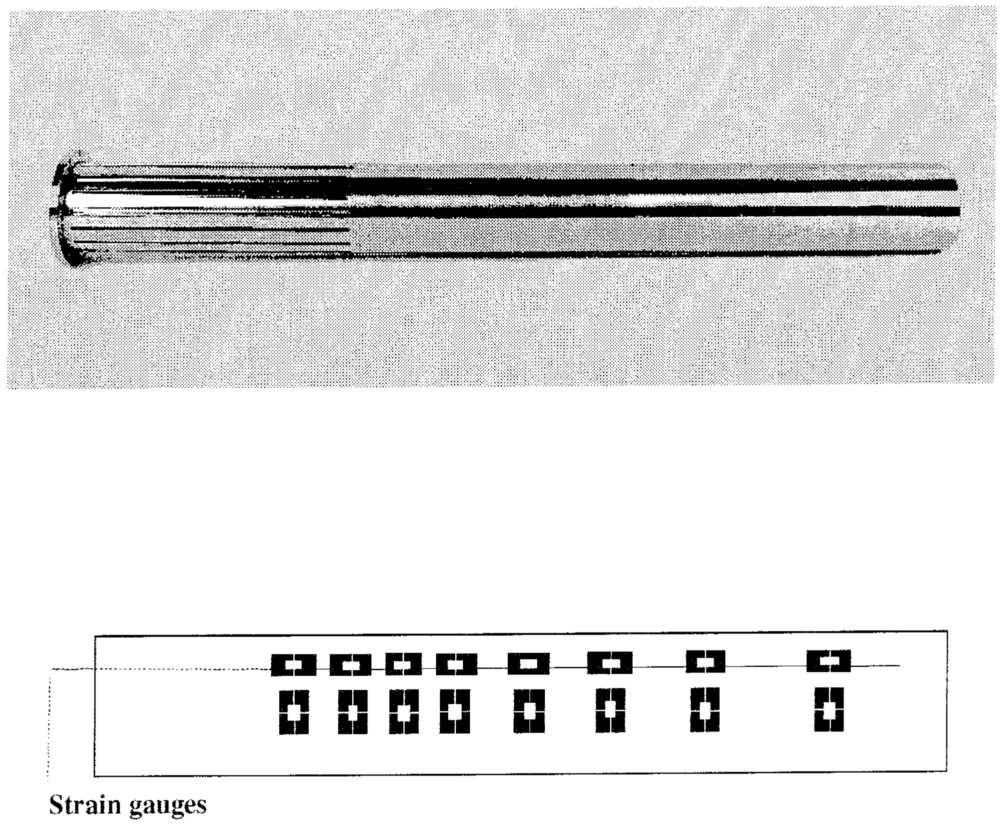


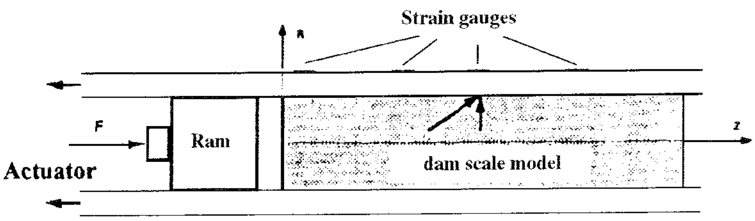
Figure 4.6-2. *Normal and tagential stress components at the gallery wall as a function of the distance from the clay core dam mid plane.*

tial type at the tunnel wall, which have been obtained from the previous analytical calculations, are confirmed, see Figure 4.6-2. The internal friction angle of the granular material is of paramount importance. FE simulations have also established a correspondence between repository dam behaviour and scale model behaviour, see Figure 4.6-1b and Figure 4.6-3.

In this simulation the clay core is supposed to be hydro-elastic. In a similar manner to thermo-elasticity, free hydration (with no volume constraint) results in an isotropic clay dilation. Because of the shear stress along the granite wall, clay expansion is disturbed. The stress distribution as shown in Figure 4.6-2 indicates a significant decrease of the swelling stresses exerted on the retaining granular backfills compared



a



b

Figure 4.6-3. *Clay core dam scale model*
a) the tube with a rigidity equivalent to the rock mass one
b) schematic cross view of the scale model.

with the clay swelling pressure (at constant volume) which is developed at the center of the clay core.

4.6.5 PREPARATION OF THE EXPERIMENTAL INVESTIGATION BY SCALE MODEL

Scale models of drift and shaft dams are placed in a stainless steel tube as shown in Figure 4.6-3. This tube represents the gallery. Its internal surface is rough and its thickness has been chosen so that the relative expansion induced by a pressure is equivalent to that of the granite gallery. At one end the tube has sixteen longitudinal slots to give it flexibility. This part of the tube is used to guide the actuator for load application. Strain gauges placed all around the tube record deformation. This equipment is currently being used in the experimental investigation of the behaviour of the clay core dam.

Questions to Mr Bernard Felix

Mrs Maria Onofrei: You have very high density – about 1.8 g/cm^3 – as well as high pressure – 40 MPa. Is that correct?

Mr Bernard Felix: Not the pressure, which for the type of clay used is about 5 MPa. In the calculation I used 3 MPa.

Mrs Maria Onofrei: In a situation where water comes into contact with your clay, do you consider that you might extrude your clay?

Mr Bernard Felix: Yes. In the same concept we have a little more detail than I showed in the transparencies, but between the backfill and the clay core there will be a sand filter that strengthens the materials.

Mr J M Hoorelbeke: Another material that could be used is concrete. Concrete and water could be a possible feature. Or the backfill itself could be concrete. I think you have the same idea Mr Bernard Felix. To emplace first a backfill and then to grout. It is also a way to avoid the erosion of clay in the backfill. We just have to be careful regarding the chemical interaction between clay and cement.

Mr Håkan Stille: Did the swelling pressure decrease with the deformation? And did you take that into consideration in your calculation?

Mr Bernhard Felix: Yes. The swelling pressure decreases, which the calculation gives an indication of. At constant volume the clay exhibits a pressure of 3 MPa and at the interface between clay core and backfill we get a result of 2 MPa. But the low support introduced for the clay behaviour is very simple. It is hydro-elastic. That means that the clay behaves elasticly and exhibits a changed volume. It needs more sophisticated development. At present this is impossible, because we have no good laboratory data to build a vertical model, which could give an indication of the stress, strain and hydraulic data.

Mr Håkan Stille: But there is a lot of testing of swelling in the rock in order to get a better model for the stress.

Mr Bernhard Felix: Yes. We have swelling tests. And now we have some interesting progress in the area of hydrogenisation at the Polytechnical University of Barcelona. Intellectual tools exist, but it is not simple to introduce the right data in these tools. It requires high stress level tests with control of the hydraulic state of the material.

5 PART III – FUTURE WORK

5.1 DISCUSSION OF FUTURE COOPERATION ON QUESTIONS RELATED TO CONSTRUCTION, OPERATION AND SEALING OF A REPOSITORY

Discussion leader: Mr Göran Bäckblom

Mr Göran Bäckblom: It has been very interesting indeed to see what is going on in different parts of the world. There is very much new knowledge to me, some of which we have heard here, and I think that most of us have realised or will realise that a repository will not only be analyzed with respect to long term safety, but it will actually will be constructed and maybe we will have some restrictions on how to construct a repository. We will have a need for more analysis of factors such as constructability, layouts, occupational safety and long term safety. So far in this workshop we have examined how to pass major fracture zones. We have also such as constructability experiences from tunnelling in Finland and Switzerland and we have all, I think, realised the need for use of observational methods, or what we call design-as- you-go. I think it was a very interesting remark made yesterday, that we should really look for the decisive layout features. It will need more work. We have also seen today that maybe you should not grout when it is better to freeze, as you will do in UK. We have examined how to grout very fine fractures in the repository. We examined on how to plug and seal boreholes in the repositories. So we have covered a range of aspects that we have to deal with, now and in the future.

It is now time for summing up this workshop and I would like you to express your opinion on the most important issues involved in the construction of a repository.

Mr Michael Raynal: We have looked for several specific techniques used for rock and characterization of features that we will face in the construction during the establishing of openings in underground facilities. In my opinion it is important that we develop a method for defining how to characterize and describe the material used in backfilling or sealing in order to be able to evaluate the safety aspects of closing or sealing. It seems to me that this question has not been really touched on during this seminar.

Mr Reijo Riekkola: I think we also need to develop of the methods to take into account the properties of rock and material required for layout and design. It seems that all grouts are not always running when you look at it more closely. Mr Christer Svemar told us this morning about observational methods. There are a lot of things to do in this area, also flexibility of the layouts during the construction. What does that mean in practice? I think you have to have some very simple and realistic tools for design. Practical tools for how you convert your technical models into practical designs.

Mr Håkan Westerlund: It was interesting to learn about the Swiss project. How to go forward when you have problems like that. When you work from the principle to take the problem when it comes; how close can you be to the problem and how can you solve it? I think we will be there, specially when using TBM.

Mr Håkan Stille: So far at this meeting we have discussed how to treat major water bearing zones. We have even discussed grout properties in some cases, but we have not discussed demands on sealing effects. Do we need for example limited penetration of the grout? Or do we need some maximum allowable water inflow in such zones, in order not to disturb the ground water regime? In the Stockholm Subway we have a water inflow of about 2 to 5 l/m and 100 m length. In the NE-1 zone in Äspö it is an inflow of several hundred litres. We need a lot of more grout. For this and similar cases a lot more of development of grouting technology is required.

Mr Gunnar Gustafson: I'd like to go on with that. One observation is that it is the first time this seminar aims to see that constructing repositories is also civil engineering. If you look at Stripa, a lot of nice work was done there, but the civil engineering aspects were not really touched. I also think that we need to differentiate between waste isolation and civil problems. Like for instance when we have talked about grouting here, you find that technology really is at the ends of what can be done with grouting. Of course the theories and ideas behind them may be the same to a great extent. It is very important that we find a strategy to use the right thing in the right place and do not mix things up too much. I have seen that there is information in abundance in this group, specially in the field of grout and grouting. If we are going to suggest future workshops I think a more specialised workshop on grouting and grouting theories would be a reasonable suggestion. Today and yesterday we have made some kind of overview of what we have done and I think it can be worth discussions in detail. Another thing that bothers me a bit is a finding from a cavern construction project within the American oil industry. They had a QA organisation that really was a nightmare. It was designed so that when you went to an oil-platform in the North Sea you would not forget the range to use to open that particular well. Of course that is important, because it would cost you some hundred thousand dollars an hour if you forget this range. I think that what we are doing here is a branch of the nuclear industry which also has a long QA tradition. If you take these QA people, they want us to do things according to specifications. Specifications are drawings and if we make a drawing of a cavern and build like that, it will be a very expensive one and a very bad one. I think we have to manage this quality assurance in such a way that we get the necessary flexibility. We encourage learning and development. That is a more general problem than about how we do things.

Mr Christian Sprecher: It struck me that talking about grouting has two contexts. One I would call a civil engineering context. It was perhaps highlighted by the talk of Mr Gottlieb Eppinger, where the grouting problems came about, in case we need to demonstrate water inflow during tunnel construction and the problem is how to control it, so you can continue the construction work. The other aspect might be called the safety-related aspect, in the direction of boring and sealing. I would suggest that perhaps in another workshop one should separate these two aspects of grouting. About general repository construction, I think on the two points that Mr Göran Bäckblom mentioned, I agree that they are interesting also from my point of view. One is the point about layout determining features and there I think that we should try to establish a more clear idea of what we are really looking for. I mean, how do we define these layout determining features? Most of us, and certainly myself, have still only a vague idea of what we mean about that. Although I realise that real determining features for one type of repository are not the same as for another type. Still I think that it might be useful to try to establish a common philosophy on how to define such features. That would have the advantage that already in the exploration phase, in the site investigation phase, you have a clear idea of what to look for. That could also help in our investigation programs, in the way that we do not gather data which in the end we do not really need.

The other point about items I am interested in, is plugging and sealing of tunnels and boreholes. I think this should really be a common concern for all of us. You may say that it will be far in the future before we have to seal a repository. That may be true, but at least in Switzerland it may very well be that our safety authorities ask us to demonstrate that we are able to seal boreholes, before we get the permission to penetrate into the proper repository area. If such a demand is expected to come from the safety authorities I think we should be ready to prove and to demonstrate. That is why I think that this complex of questions is of a more immediate interest.

Mr Bertrand Vignal: I think an idea undergoing great progress is retrievability. We can see in Sweden, that SKB in the new research and demonstration program has introduced a phase of retrievability. In France, due to the latest and applicable law we have to work on retrievable repositories. This retrievability is something that has to be taken into consideration, perhaps in the next workshop. It is a new idea, which compels changes of the previous concept we have built. Also in KBS-3, it has an influence. In all French concepts we are now thinking of some changes. Exchange between people involved in construction would be of interest.

Mr Jean-Michel Hoorelbeke: If you want to dispose of waste underground you have first to excavate, then to operate and finally to seal your repository and maybe you have to consider the possibility of retrievability of the waste disposed of. One could imagine that all these items are disconnected. That is completely wrong. Wrong, because of the question of long term safety. I could give an obvious example. If you seal a drift using a dam like the dam presented by Mr Gunnar Gustafson you have to be careful so that the groundwater does not find a path way into the facility through the open fractures. Then you have two options to cope with this. You can grout. Grouting comes with plugging. Also you can try to minimize the fractures due to the excavation work. That is the purpose of some programs we have in the field of smooth-blasting. All that means that excavation and sealing are connected, even after 50 years or more. Maybe this connection between excavation, sealing and retrievability could be a good subject for a seminar. Of course this connection has to be considered in relation to the safety issue. From this seminar I have got the idea of design-as-you-go is a common idea. That means that you must have a flexible concept and a flexible design. You must have in your pocket a catalogue of possible techniques, depending on the conditions in your geological formation.

Another idea for future cooperation could be the durability of the materials that we discussed today. Durability of rocks, durability of materials including different materials used like cement and clay. All these items are connected to safety issues, as Mr Michael Raynal said a few minutes ago.

Mr Gary Simmons: I think that what we have heard is that there are still developments required in the areas of characterization and application of grouts as they affect construction in these highly fractured wet zones. This includes development and demonstration of characterization methods dealing with issues like zone stability and zone groutability; equipment development and demonstration for improved drilling, improved monitoring and improved grouting in fractured rock zones with high water pressures; development in the areas of controlled grout applications, specifically involving equipment; development of the grout materials to specific characteristics that suit waste disposal, including the long-term durability if it is a long-term seal; and improved methods of grout application. The outcome of this workshop could be a statement that says: You should be very careful in detailed advance planning of the grouting and lining alternatives that might be applied before you approach these highly-fractured wet zones. When you get there, there are alternative available, materials at hand, the experts can be called in fairly quickly to work on the project.

This is an aspect of the observational method with which I fully concur. I am also very sympathetic to the idea that we have to start educating the public and educating the standard construction and grouting industry users who will be looking on our work. The point is that the observational method is an essential element of success in the geotechnical environment. If we can not achieve acceptance of this approach, if we are restricted to a design prepared in advance, if we are required to apply it, we are going to have a very difficult time completing our work successfully.

I believe there are several issues which should come out in the future workshops. One, that I think will be an interesting challenge for some people in this group, is the issue of retrievability and safe guards for used fuel disposal and how those will effect repository design.

Mr Paul Thompson: I haven't heard anything that indicates to me that any significant advances in grouting techniques have been made over the last decade or two. Problems were recently encountered in Chicago with the flooding of the old tunnel system, which paralyzed the business district for several days. The methods used to seal the leaks included throwing truckloads of mattresses, amongst other things at the problem. Although the leaks were eventually sealed, it appeared to be more of a trial-and-error methodology than a science. It seems to me that we still require better methods for designing grouting programs, and the techniques to properly apply them. I think we are making considerable advances in materials science and this work should continue. We need to improve the science of grouting. I do not know how to achieve that, but it is what we need to do.

Another unrelated aspect you mentioned was layout-determining features. One thing we have really noticed in our URL program is the effect of rock stresses and rock properties on underground conditions, including hydraulic conditions in fracture zones. Therefore it is important to carefully consider rock stresses and properties when determining the underground layout of a repository.

Mrs Maria Onofrei: I agree with everything which has been said so far. Personally I agree with what Mr Gunnar Gustafson said. That we did not treat what we could do in grouting development in general for materials. There is still a lot to do. As we advance, we change our requirements for materials to use in the repository. Now we know that pH is a problem and we have to solve that. There is something else that came up. You said we use two materials, clay and cement. So far we do not know how compatible those two are. We have some information, which makes us think that they may be compatible.

Mr Dale Wilder: On the surface it may not be apparent that in the US program we have those issues you term grouting because our site is not saturated. We do not have to control water-flow situations. For the US program we are looking more to assess what are the performance impacts of cement materials like shotcrete, and especially how they influence those few fractures which may have water in them. Having said that, I guess I would note that I was interested to see that the techniques, not the material but the techniques of grouting, do not seem to have changed a great deal from the time I was introduced to them, some 20 – 30 years ago. Somebody has made the comment that it was a kind of "black art". I think that to some extent this reflects the difficulty. The same challenges you have been talking about in terms of grouting are the ones that we are facing our project, in trying to describe the impacts of introduced materials on repository performance and how you really can place bounds on performance. I think one of the things that came out of this discussion is the observation that there is probably existing information in all of the projects suggesting that performance impacts may vary widely, depending on the design detail of the individual projects.

Mr Andy Nold: I can only join Mrs Maria Onofrei. I would point out one thing, which I think is quite important for the Swiss party but also for others. We were talking about layout determining features. To get any further we should involve all the safety people to know what we are talking about. It was said that we should somehow use more civil engineering, not only safety, but I think that we should bring all of it together to solve the problem about layout determining features. Otherwise we do not know what to search for, what to look for, and at the end we spend too much money on things not needed. For me this is the critical point.

Mr Dwayne Chesnut: There are a couple of things that I have thought of. There are some great differences in philosophy about design performance of repositories. Just an other relation, the function of grouting in SKB's approach and the Canadian approach. In one case, of what I understood, was to provide access to better rocks so you could place the repository where you do not need to grout in order to have any impact on long term performance. In the Canadian program the grout will actually be a part of the long term containment. That seems to require very different appraisals of the functional problem. Very different things have to be proved in terms of long term repository operations, which is one example, where we have had different approaches in different countries. Several countries' strategies are that, without having to guarantee, they make isolation much more provable. You need an integrated approach for characterization of design. The design-as-you-go idea really makes a lot of sense that we are not able to use in our program. I really feel that there is a lot of promise in this kind of approach.

Mr Hideaki Osawa: I think that in Japan there are many tunnel construction projects. We have not considered the tunnelling work on the point of repository construction, in so called design-as-you-go. We do not have any criteria or strategy for repository construction. Sometimes we have some programs on this criteria where we combine grouting. In my opinion the criteria and strategy of repository construction and characterization are important to discuss.

Mr Kimio Miyakawa: I think the distances might have a great influence on how the grout spreads. It is important for the understanding of the effect of the grout in the rock mass. How can we investigate and estimate the grout spreading? How can we cut out control grout? I also think about the groundwater supply mapping. How can we exclude this effect and carry out a correct investigation?

Mr Michel Dardaine: Concerning such a development of swelling clays, I will say that in the future we must be able to design representative full-scale demonstration tests of the front barriers. To study the compartment in situ and to take into account for example the temperature effects.

Mr Tomas Franzen: The technical aspects have already been discussed. Design-as-you-go is quite obvious for us in this room and the rock engineering society, what we mean and why we have to do it like this. But I think that it might in other fora be a symbol of lack of knowledge in some way. We had a problem to demonstrate that we know enough to do this. It might seem like "Let's see what to do when we get there". It is not very good. We should use a word like "observational method". That sounds positive and active.

Mr Gottlieb Eppinger: I will point out one simple thing. There seem to be different questions about consolidated reference zones when grouting. A civil engineer asks if the stability of the consolidated zone is enough to go through. If it is OK, he is satisfied. But the safety engineer for the repository has many different questions with respect to stability.

Mr Jukka-Pekka Salo: We have discussed different design processes. I think we should go more deeply into these different designs and what should be integrated into them. Nobody, except Mr Andy Nold, mentioned money and economy. I think they should also be taken into account. It is important for the layout-determining features to discuss in detail and take up the safety assessment tools.

Mr Jorma Autio: The first part of this workshop was about excavation through water conducting major fracture zones. I think that I have found out that this was a big problem when we came here and that we have found many solutions to it. I personally want to emphasize the need to find properties for the characterization of the near-field, which will actually guide our "design-as-we-go-according-to-our-observations". I think that is a better word.

Mr Bernard Felix: From this seminar I have noticed that grouting techniques can be divided into two areas: treatment of major failure zones and treatment of various host rocks. That means in grouting within joints. Maybe the techniques are completely different. I should like to ask a question that could be answered now. I have noticed that the first problem is a sort of a civil engineering problem. A laboratory is being built now in a deep granite formation with high stresses and the problem is important. It is very similar to the problem that we have run into in tunnel construction and the mining industry. Will the treatment of this major failure zone be able to contribute to the sealing system or is it completely impossible? Will the sealing of the repository be completely assumed by dams and treatment of the host rock or limited damaged host rock? I have noticed that the reference of grouting a major fracture zone has durability aspects.

Mr Göran Bäckblom: I have listened and heard what ideas you have for possible future discussions. SKB took the initiative to discuss a few experiences and see if there were other ideas about how to construct and design repositories. From what has been said around this table I have identified six topics of interests: 1) chemical inventory of the repository, 2) grouting, 3) layout-determining features, 4) plugging and sealing, 5) retrievability and 6) durability of materials.

I would like to propose, as there are several interesting topics here, for instance 2, 3 and 4, that one workshop on these topics be considered.

Mr Gary Simmons: I would suggest that grouting should not be the next topic because it was a major part of this workshop. My personal preferences, without remembering the specifics of the list, are the layout determining features, retrievability and safeguards, and the whole issue of plugging and sealing, including the material side. However, we are probably best prepared to deal with the layout determining issues. In fact, you could include retrievability and safeguards into this topic as two of the issues that can effect design.

Mrs Maria Onofrei: To me it would be better to combine some of these six, instead of having independent workshops.

Mr Gary Simmons: The chemical inventory issue is an interesting one, but I would personally think that some work would have to be done to prepare such a meeting. We would be interested in items three and five. If I was to suggest a workshop topic, I would propose that three and five be combined items under a title of "Factors that effect the layout and design of repositories" and that this would be the next subject we attempt to address. AECL could probably arrange this workshop late 1993 or early 1994.

Mr Göran Bäckblom: As a general conclusion I find that everyone agrees that it would be very nice to have a combined workshop on layout-determining features and retrievability, possibly in Winnipeg.

With this I hereby close the workshop and thank you all very much for your attendance and your contributions.

AGENDA

March 30

9.00	**WELCOME** *(Per-Eric Ahlström)*
9.15	**PRACTICAL MATTERS** *(Torsten Eng)*
9.30	**INTRODUCTION** *(Göran Bäckblom)*
9.45	Coffee
PART I –	**EXCAVATION THROUGH WATER-CONDUCTING MAJOR FRACTURE ZONES**
10.00	**Characterization of major water-bearing fracture zone to** optimize the location of a ventilation shaft *(Paul M Thompson)*
10.45	**Finnish experiences of sealing water-conducting zones** in rock *(Reijo Riekkola)*
11.30	**DISCUSSIONS**
12.00	LUNCH
13.30	**Passage of the major fracture zone NE-1 at the Äspö Hard** Rock Laboratory *(Gunnar Gustafson, Ingvar Rhén, Roy Stanfors, Håkan Stille)*
14.15	**Excavation through major water-conducting fracture zones** *(Gottlieb Eppinger)*
15.00	Coffee
15.30	DISCUSSIONS

March 30 (contd.)

16.00 **NEED FOR DEVELOPMENT OF TECHNOLOGY**
(Major Fracture Zones) – GENERAL DISCUSSION
(Per-Eric Ahlström, Göran Bäckblom)

17.30 **END OF SESSION**

19.00 DINNER

March 31

PART II – CONSTRUCTION, OPERATION AND SEALING

8.30 **Design and construction of repository – Aspects on demands**
and considerations
(Christer Svemar)

9.00 **The planning of the Rock Characterisation Facility (RCF)**
atn Sellafield
(Brendan J Breen)

9.30 DISCUSSION, BREAK

10.00 Experience with grouting at AECL Research
(Maria Onofrei)

11.00 **Demonstration of the emplacement of a clay engineered**
barrier in a granite formation
(Michel Dardaine et al.)

11.30 DISCUSSION

12.00 LUNCH

13.30 **Sealing of exploration boreholes in and around a potential**
radioactive-waste disposal site
(J F Ouvry et al.)

March 31 (contd.)

14.00 **Analysis of a clay core dam behaviour**
(Bernard Felix et al.)

14.30 **GENERAL DISCUSSIONS**

15.00 **SUMMARY OF DISCUSSIONS,** BREAK

PART III – FUTURE WORK

15.30 **DISCUSSION OF FUTURE COOPERATION ON** QUESTIONS RELATED TO CONSTRUCTION, **OPERATION AND SEALING OF A REPOSITORY**

16.15 **FUTURE WORKSHOPS**

16.30 END

16.45 Bus transport to Norrköping for those attending the visit to Äspö Hard Rock Laboratory, April 1

April 1, 1993

OPTIONAL VISIT TO ÄSPÖ HARD ROCK LABORATORY

10.00 ARRIVAL AT SITE OFFICE

11.00 **VISIT TO THE ÄSPÖ ISLAND**

12.00 LUNCH

13.00 **UNDERGROUND VISIT**

15.00 END

TRANSPORT TO AIRPORT

LIST OF PARTICIPANTS

AHLSTRÖM, Per-Eric	SKB
ALESTAM, Mats	Sydkraft Konsult
AUTIO, Jorma	TVO
BREEN, Brendan J	UK Nirex
BRO, Kari	Laisvall Mine
BÄCKBLOM, Göran	SKB
CHESNUT, Dwayne	LLNL
DARDAINE, Michel	CEC
DIDRIKSSON, Valter	Sydkraft Konsult
ENG, Torsten	SKB
EPPINGER, Gottlieb	NAGRA
FELIX, Bernard	ANDRA
FRANZEN, Tomas	SveBeFo
GUSTAFSON, Gunnar	CTH
HAIJTINK, Bert	CEC
HEDMAN, Tommy	SKB
HOORELBEKE, Jean-Michel	ANDRA
KICKMAIJER, Wolfgang	NAGRA
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MIYAKAWA, Kimio	CRIEPI
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OUVRY, J F, absent, represented by M BLANCHIN	BRGM
RAYNAL, Michel	ANDRA
RIEKKOLA, Reijo	Saanio & Riekkola Consulting Engineers
SALO, Jukka-Pekka	TVO
SIMMONS, Gary	AECL Research
SPRECHER, Christian	NAGRA
STANFORS, Roy	R S Consulting AB
STILLE, Håkan	KTH
SVEMAR, Christer	SKB
THOMPSON, Paul	AECL Research
VIGNAL, Bertrand	ANDRA
WESTERLUND, Håkan	Håkan Westerlund AB
WILDER, Dale	LLNL

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TR 94-01
Anaerobic oxidation of carbon steel in granitic groundwaters: A review of the relevant literature
N Platts, D J Blackwood, C C Naish
AEA Technology, UK
February 1994

TR 94-02
Time evolution of dissolved oxygen and redox conditions in a HLW repository
Paul Wersin, Kastriot Spahiu, Jordi Bruno
MBT Technología Ambiental, Cerdanyola, Spain
February 1994

TR 94-03
Reassessment of seismic reflection data from the Finnsjön study site and prospectives for future surveys
Calin Cosma[1], Christopher Juhlin[2], Olle Olsson[3]
[1] Vibrometric Oy, Helsinki, Finland
[2] Section for Solid Earth Physics, Department of Geophysics, Uppsala University, Sweden
[3] Conterra AB, Uppsala, Sweden
February 1994

TR 94-04
Final report of the AECL/SKB Cigar Lake Analog Study
Jan Cramer[1], John Smellie[2]
[1] AECL, Canada
[2] SKB, Sweden
May 1994

TR 94-05
Tectonic regimes in the Baltic Shield during the last 1200 Ma - A rewiew
Sven Åke Larsson[1,2], Eva-Lena Tullborg[2]
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[2] Terralogica AB
November 1993