International Progress Report

IPR-01-17

Äspö Hard Rock Laboratory

Report on the installation of the Backfill and Plug Test

David Gunnarsson
Lennart Börgesson
Harald Hökmark
Lars-Erik Johannesson
Torbjörn Sandén

Clay Technology AB

May 2001

Svensk Kärnbränslehantering AB
Swedish Nuclear Fuel
and Waste Management Co
Box 5864
SE-102 40 Stockholm Sweden
Tel +46 8 459 84 00
Fax +46 8 661 57 19

Äspö Hard Rock Laboratory

Report no.	No.
IPR-01-17	F61K
Author	Date
Gunnarsson, Börgesson, Hökmark, Johannesson, Sandén	01-05-01
Checked by	Date
Approved	Date
Christer Svemar	02-06-07

Äspö Hard Rock Laboratory

Report on the installation of the Backfill and Plug Test

David Gunnarsson
Lennart Börgesson
Harald Hökmark
Lars-Erik Johannesson
Torbjörn Sandén

Clay Technology AB

May 2001

Keywords: Backfill plug, compaction, instrumentation, hydraulic conductivity, engineering barriers

Abstract

This report describes the work with the installation of the Backfill and Plug Test until start of the water saturation.

The main objectives of the test are

- to develop and test different materials and compaction techniques for backfilling of tunnels excavated by blasting
- to test the function of the backfill and its interaction with the surrounding rock in a tunnel excavated by blasting
- to develop techniques for building tunnel plugs and to test the function

A 28-meter long tunnel section has been backfilled and instrumented. Half of the length of the tunnel was backfilled with a mixture of 30 % bentonite and 70 % crushed rock (30/70) and the other half was backfilled with crushed rock (0/100) with bentonite blocks placed at the roof. The backfill was placed and compacted layer wise with vibrating plates that were developed and built for this purpose. A technique with inclined compaction was used in the entire tunnel section from the floor to the roof and the inclination of the layers was about 35 degrees. The backfill was split by permeable mats every second meter with the purpose to artificially water saturate the backfill and be able to apply a hydraulic gradient between the mats for flow testing. The backfill and surrounding rock has been instrumented with about 200 transducers mainly for the purpose of measuring water pressure, total pressure and water content. A tunnel plug made of concrete with a half-meter thick bentonite O-ring has been constructed at the end of the test section.

The backfilling equipment worked well, both in terms of function and achieving expected densities, but needs to be improved concerning reliability, durability and safety. Although the dry density of the 30/70 backfill close to the roof and walls was lower than 1650 kg/m^3 the bulk average dry density is estimated to be between 1650 and 1700 kg/m^3. The mean measured dry density of the 0/100 backfill was 2170 kg/m^3.

Sammanfattning

I denna rapport beskrivs arbetet med installationen av Backfill and Plug Test fram till start av vattenmättnaden.

Projektets huvudsakliga mål är

- att utveckla och testa olika återfyllningsmaterial och tekniker för återfyllning av tunnlar drivna med sprängning
- att testa återfyllningens funktion och dess samverkan med omgivande berg i en tunnel driven med sprängning
- att utveckla teknik för att bygga tunnelpluggar och testa funktionen

En 28 meter lång tunnelsektion har återfyllts och instrumenterats. Halva tunnellängden återfylldes med en blandning av 30% bentonit och 70% krossat berg (30/70). Den andra halvan återfylldes med 100% krossat berg (0/100) med bentonitblock placerade närmast taket. Återfyllningen lades ut och packades lagervis med vibratorplattor som utvecklats och byggts för detta ändamål. Tekniken med packning av lutande skikt användes i hela tunnelsektionen från golv till tak med lutningen c:a 35 grader. Återfyllningen avdelades med permeabla mattor var annan meter med syfte att bevätа återfyllningen konstgjort och att kunna applicera en hydraulisk gradient mellan mattorna för flödestestning. Återfyllningen och omgivande berg har instrumenterats med c:a 200 givare för mätning av vattentryck, totaltryck och vatteninnehåll m.m. En tunnelplugg av betong med en halv meter tjock O-ring av bentonit har installerats i slutet av testsektionen.

Utrustningen och tekniken för återfyllning och packning fungerade väl, både avseende funktion och erhållen densitet, men pålitligheten, hållbarheten och säkerheten behöver förbättras. Fastän torrdensiteten hos 30/70 återfyllningen blev lägre än 1650 kg/m^3 uppskattas att medeltorrdensiteten blev mellan 1650 och 1700 kg/m^3. Den mätta medeltorrdensiteten hos 0/100 återfyllningen blev 2170 kg/m^3.

Acknowledgements

The installation of the Backfill and Plug Test has been accomplished with the assistance of many persons and organisations. The assistance of the following persons are greatly appreciated:

Olle Olsson, Christer Svemar, Stig Pettersson and Gunnar Ramqvist (SKB) for support and guidance.

Roland Pusch (Geodevelopment) for checking and reviewing the manuscript.

Lennart Hallstedt (Halltech) for the design of the roof compactor and the slope compactor and for participation in the process of developing the backfilling equipment.

Resa Goudarzi (Clay Technology AB), Nils-Göran Myrén (NCC) and Bo Hultgren for assistance during the fieldwork.

Irene Hansen (Clay Technology AB) for typing assistance.

Lief Stenberg (SKB) for supervising the characterisation of the near field rock.

Thomas Karlsson (SKB) for the design of the automated data acquisition system for the resistivity probes and work with the main data collection system Orchestrator.

The following persons at Geosigma:

- Jan-Erik Ludvigson, who was responsible for the hydraulic testing of the core-drilled holes for monitoring water pressure in the rock.
- Daniel Lindström, who made the design of the system for filling the permeable mats with water and controlling the pressure. He also made the design of the long bentonite packers.
- Agne Bern and Jon Olausson, who did the installation of packers, installation of system for filling permeable mats with water and controlling pressure and the leakage testing of the lead-throughs.

Torleif Dahlin (Lund Institute of Technology) for the work with designing the resistivity probe.

Rickard Karlsén (NCC Oskarshamn), who was responsible for the mixing of the backfill material.

Finn Johnsson (Finn Johnsson Engineering AB) for help with the design of the pressure cylinders.

Göran Nilsson (JAA), who made a tentative mapping of lithology and structures and examined the drill cores from the holes for monitoring water pressure in the rock.

Christer Ericsson (Geocon), who was responsible for the surveying, handling and reporting of data to the SICADA database.

The following two organisations have made their own installations in the test:

AITEMIN have developed and installed the system for measuring local permeability under the guidance of José-Luis Garzia-Sineriz and José-Luis Fuentes-Cantilla (see Appendix 8).

The Microbiological department of the University of Gothenburg have installed microorganisms in the backfill. The work was made under the guidance of Karsten Pedersen (see Appendix 12).

The following companies have also contributed to the work with the installation of the test:

Mediabolaget have the different parts of the installation.

Berg Bygg Konsult performed the laser scanning of the tunnel.

Höganäs Bjuf AB compacted blocks for the backfilling.

BEPEX-Hoskawa manufactured the bentonite pellets.

Hultdins in Malå manufactured the vibrating plate according to the design drawings.

Oskarshamns Maskintenik was responsible for service and repair of the compactors.

LM Bulk-& Materialhantering AB delivered equipment for the pellet-blowing machine, which was assembled by Oskarshamns Maskinteknik AB.

Bergman-Axab delivered the flange packing.

AB Verkmetall manufactured the steel cones.

Skandinavisk Ytförädling AB manufactured a prototype of the pressure cylinder and also supplied the know-how with respect to corrosion protection.

Herrströms Mekaniska Verkstad AB manufactured three more pressure cylinders.

The Swedish Institute of Corrosion (KI) assisted in the choice of material for the sensors in the backfill.

Executive summary

General

The *Backfill and Plug Test* includes tests of backfill materials and emplacement methods and a test of a full-scale plug. It is a test of the integrated function of the backfill material and the near field rock in a deposition tunnel excavated by blasting. It is also a test of the hydraulic and mechanical functions of a plug. The test is partly a preparation for the Prototype Repository.

The entire test set-up with backfilling and casting of the final part of the plug was finished in autumn 1999 and the water saturation, with water filling of permeable mats, started in late 1999. This report describes the work with the installation until start of the water saturation.

The main objectives of the test are

- to develop and test different materials and compaction techniques for backfilling of tunnels excavated by blasting
- to test the function of the backfill and its interaction with the surrounding rock in full scale in a tunnel excavated by blasting
- to develop technique for building tunnel plugs and test the function

Test layout and installation procedures

Figure 1 shows an overview of the test layout. The newly excavated inner part of the tunnel is not used for the test but only filled with drainage material. The test region, which is about 28 m long and located in the old part of the ZEDEX tunnel, can be divided into the following three test parts:

1. the *inner part* filled with backfill containing 30% bentonite.
2. the *outer part* filled with backfill without bentonite and with bentonite blocks at the roof.
3. the *plug*.

The backfill sections were applied layer wise and compacted with vibrating plates that were developed and built for this purpose. It was concluded from preparatory tests that inclined compaction should be used in the entire cross section from the floor to the roof and that the inclination should be about 35 degrees.

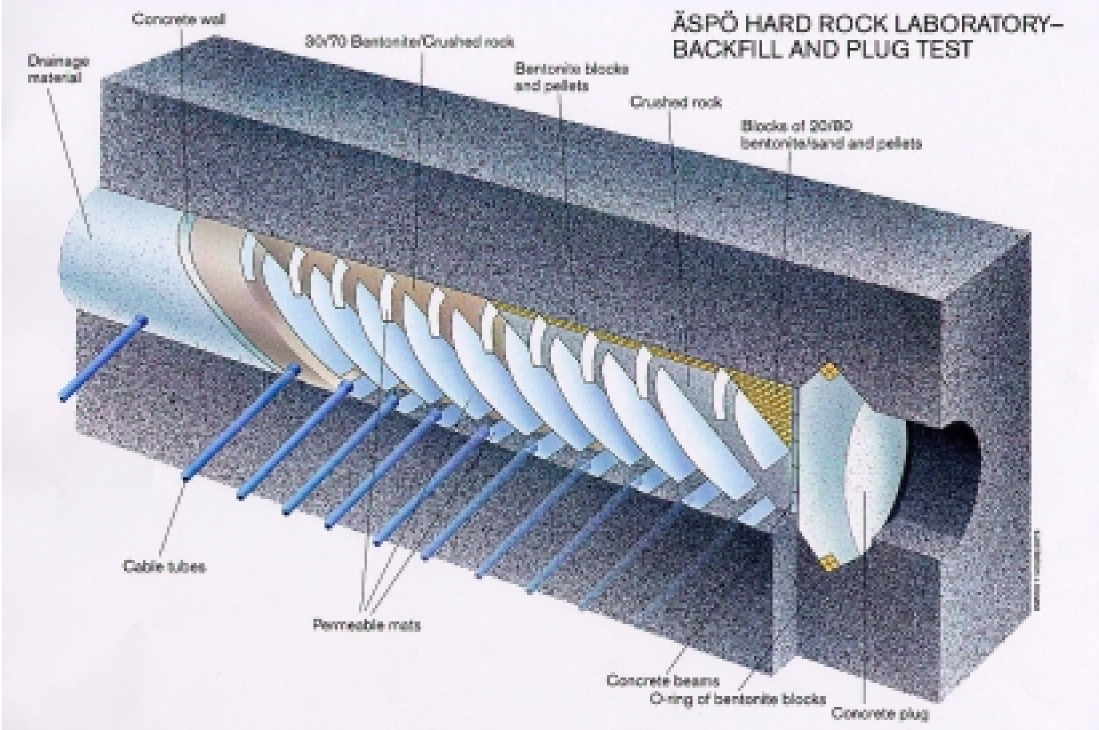


Figure 1. *Overview of the Backfill and Plug Test.*

The inner test part is filled with a mixture of bentonite and crushed rock with a bentonite content of 30%. The composition is based on results from laboratory tests and field compaction tests. The outer part is filled with crushed rock with no bentonite additive. Since the crushed rock has no swelling potential but may instead settle with time, a slot of a few dm was left between the backfill and the roof and filled with a row of highly compacted blocks with 100% bentonite content, in order to ensure a good contact between the backfill and the rock. The remaining irregularities between these blocks and the roof were filled with bentonite pellets.

The two test parts are about 14 meter long. Drainage layers of permeable mats divided each test part in order to apply hydraulic gradients between the layers and study the flow of water in the backfill and near field rock. The mats are also used for artificially water saturating the backfill. The mats were installed with the individual distance 2.2 m. Each mat section was divided in three units in order to be able to separate the flow close to the roof from the flow close to the floor and also in order to separate the flow close to the rock surface from the flow in the central part of the backfill.

The outer section ends with a wall made of prefabricated beams for temporary support of the backfill before casting of the plug. Since in situ compaction of the backfill cannot be made in the upper corner, this triangle was instead filled with blocks of bentonite/sand mixture with 20% bentonite content.

The backfill and rock were instrumented with piezometers, total pressure cells, thermocouples, moisture gauges, and gauges for measuring the local hydraulic conductivity. The axial conductivity of the backfill and the near field rock will be tested after water saturation by applying a water pressure gradient along the tunnel between the mats and measuring the water flow. All cables from the instruments were enclosed in Tecalan tubes in order to prevent leakage through the cables. The cables were led through steel pipes through the rock to the data collection room in boreholes drilled between the test tunnel and the neighbouring Demo-tunnel. The Tecalan tubes were led through watertight connections in a flange fixed to the end of the steel pipe in each borehole.

The *plug* is designed to resist water and swelling pressures that can be developed. It was equipped with a filter on the inside and a 1.5 m deep triangular slot with an ”O-ring” of highly compacted bentonite blocks at the inner rock contact.

The construction of the plug was made in five steps:

1. Slot excavation (drilling and reworking of slot surfaces)
2. Casting of abutment and conical rim
3. Placement of prefabricated retaining wall beams during backfilling
4. Placement of O-ring bentonite blocks
5. Casting of plug main body

The flow testing in the backfill is planned to start after saturation, when steady state flow and pressure have been reached.

Results

Since this report only deals with the set up of the test, no conclusions can be made regarding function and test results, except for the densities reached during backfilling. In general the installation worked well and all problems that occurred were apparently solved in a satisfactory way. On the other hand the test set up was complicated, since many new techniques had to be developed and applied and many procedures took longer time than expected.

The backfilling technique requires a lot of skill from the operator of the backfilling equipment. The successively increased skill was evident when analysing the densities of the material close to the roof. It was found to be increased from the first couple of layers to the last three sections of 30/70 backfill. The instrumentation in the roof disturbed the compaction for almost all of the layers. Extreme irregularities in the roof of the tunnel also affected the density in this area in a negative way.

The backfilling equipment worked well, both in terms of function and achieving expected densities, but it needs to be improved concerning reliability, durability and safety.

In the areas that could be accessed with the slope compactor a mean dry density of 1700 kg/m^3 was achieved. When backfilling proceeded without disturbance a median dry density of more than 1500 kg/m^3 at the roof was achieved. The skill of the operator of the carrier increased during the length of the backfilling. The density close to the roof in the two first sections was thus lower. Avoiding harming instruments in the roof resulted in local zones of lower density also in the later sections. Although the density close to the roof and walls is lower than 1700 kg/m^3 the influence on the average density is not very strong due to that the low-density zone is not reaching more than a few decimetres away from the rock. The bulk average dry density is therefore estimated to be between 1650 and 1700 kg/m^3.

The mean measured dry density of the 0/100 material was 2170 kg/m^3. The installation of bentonite blocks and blowing of pellets between the backfill and the roof worked well.

The decisions for the design of the plug were preceded by literature studies, case studies and numerical calculations. The bentonite O-ring, which is motivated by the high demands on the sealing effect, is however a component that is unique to this application, and made both design and construction of the plug complicated issues.

Altogether, the casting seems to have been successful but the effect of the sealing cannot be evaluated until after the plug has been flow tested and dismantled.

Table of contents

1 Introduction

1.1 Scope of reporting

This report describes the installation phase of the Backfill and Plug Test and the preparations made in advance of the backfilling. Laboratory investigations of properties of the backfill and modelling of the hydro-mechanical processes during wetting and flow testing are described in separate reports /1-1, 1-2, 1-3/.

1.2 Background

The Äspö Hard Rock Laboratory is an important part of SKB's work on design of the deep repository and is developed with the aim of providing possibilities for research, development and demonstration in realistic, undisturbed rock that can be considered for the deep repository. The main objectives of the laboratory are to

- increase the scientific understanding of the function and safety margins of a deep repository
- develop and test technologies for minimising costs with maintained high quality and safety levels
- demonstrate the technology that will be used for deposition of spent nuclear fuel and other types of long-lived radioactive waste

The work in the ÄHRL is divided into three phases: pre-investigation, excavation and operation. During the **pre-investigation phase** (1986-1990) the site of the laboratory was selected. The properties of natural rock were determined and described and predictions made with respect to geohydrology and other conditions to be observed during the excavation phase. During the **excavation phase** (1990-1995) a ramp was excavated down to 460 meters depth below the ground surface and additional shafts for ventilation and personal transports as well as surface facilities were completed. Extended bedrock investigations and tests were carried out in parallel. During 1995 the **operation phase** started. For this phase the work in the ÄHRL has been grouped into the following four target areas

1. Access preinvestigation methods
2. Work out detailed investigation methods
3. Test models for description of the barrier functions of the rock
4. Demonstrate technology for and function of important parts of the deep disposal system

The fourth target area ”Demonstrate technology for and function of important parts of the deep disposal system” presently comprises the following main projects

- Prototype Repository
- Technology Demonstration
- Retrieval Test
- Backfill and Plug Test
- Long Term Test of Buffer Material (LOT)

The **Prototype Repository** will be a full-scale copy of a repository with six deposition holes. Its aim is to demonstrate the integrated function of the repository components and to compare the results with models and assumptions. It will include the testing of characterisation methods in the deposition tunnel, boring of deposition holes, placement of buffer, canister and backfill, construction of plug and instrument installations. The modelling is carried out as integral parts of the tests.

The **Technology Demonstration** constitutes a full size version of a deposition tunnel with four deposition holes. It serves to develop and test technology and equipment as well as demonstrate the different steps required for deposition of spent nuclear fuel in a deep geologic repository. The project comprises construction of equipment in full scale for emplacement of buffer and canisters in deposition holes under radiation shielded conditions.

The **Retrieval Test** is a full size copy of a repository with one deposition hole but no backfilling of tunnel. It aims at testing methods for canister retrieval from a saturated buffer.

The **Backfill and Plug Test** is a full size version of a repository with backfilled tunnel and a confining plug. It aims at testing different backfill materials and techniques for backfilling and plugging, and studying of the integrated function of rock, backfill and plugs. The test is partly a preparation for the Prototype Repository.

LOT is carried out in scaled down geometry, in single boreholes. The project aims at either simulating conditions that are unlikely to occur but still possible, or exaggerate conditions with normal conditions. LOT will also include some long time reference tests.

1.3 Relevance and justification of the Backfill and Plug Test.

The advantage of and the possibilities with the Äspö Hard Rock Laboratory is that it is possible, in full scale and under realistic conditions, to test and demonstrate equipment, methodology, and performance of important parts of the repository system. The basic strategy for development and testing of techniques in the Äspö laboratory is to demonstrate the technique of the reference concept (described in SKB reports FUD 95 /1-4/, SR 95 and SR 97) and to compare the reference concept to new techniques and new concepts. When it comes to the design of new concepts it is very important that these are tested in the Äspö Hard Rock Laboratory to verify that they meet the demands for quality and safety. Tests at the Äspö HRL are supported by performance and safety analyses.

Little attention has so far been paid to <u>deposition tunnels</u> and <u>plugs</u>. The role of the backfill in the overall safety assessment of the repository is being investigated at present. Substantial cost savings can be made and environmental care taken if a major part of the backfill can be produced from the rock excavated from the deposition tunnels. This material can either be mixed with bentonite, as in the KBS3 concept, or be used without additives.

A reason for making a separate *Backfill and Plug Test* is that it will be made in a tunnel excavated by blasting while the *Prototype Repository* will be located in a TBM tunnel. Furthermore, the hydraulic performance of the backfill and near-field rock can only be tested with techniques with filter mats, which destroy the natural saturation processes in the backfill.

There are some necessary requirements of the materials for backfilling of tunnels, rooms and shafts. The following is stated in the FUD 95 /1-4/ (in italic):

The backfill requirements are

- *to obstruct upwards swelling of bentonite from the deposition holes*
- *to prevent or restrict the water flow in the tunnel and around the canister*
- *to resist chemical conversion during a long period of time*
- *not to cause any significant chemical conversion of the buffer surrounding the canister*

When the water transport around the canisters is not affected by the hydraulic conductivity in certain sections of the tunnel, the demand for mechanical stability is the most important requirement. Bentonite-free ballast can then be used.

The demands on the backfill should be considered in relation to the properties of the excavation-disturbed zone surrounding tunnels and rooms in rock. Since it is the combined functions of the backfill and damaged zone that is important for the hydraulic and mechanical performance, it is not meaningful to put higher demands on the backfill than on the damaged zone. The flow in the damaged zone can be made insignificant by boring or careful blasting. If the cost for making sure that the damaged zone will have no hydraulic influence has been taken, one should prevent the tunnel from being a significant transport pathway. For a bored or a carefully blasted tunnel it is thus only motivated to try a tight backfill.

For a blasted tunnel with a certain axial permeability of the damaged zone, it might be relevant to try both a tight backfill with a low transmissivity compared to the damaged zone as well as a permeable backfill that makes the influence of the damaged zone insignificant.

The following materials and their hydraulic interaction with the damaged zone will be investigated:

1. The reference material, backfill with a high content of bentonite.
2. An alternative material with no bentonite content but with the zone closest to the roof being sealed with bentonite blocks and pellets.

The location of the test in the TAS Z tunnel, where the ZEDEX test was performed, means that the test is performed in very well characterised. The conditions for predicting the integrated hydraulic and mechanical function of the backfill and near field rock are thus good.

The test is also a complement to the ZEDEX test, since it gives a possibility to further investigate the hydraulic properties of the damaged zone.

1.4 State of knowledge at commencement of the test

Studies of the concept of deep geological disposal have been conducted for about 20 years worldwide. These studies have included research in laboratories and in underground sites in Sweden, Canada, Belgium, Switzerland and lately also Japan and Finland. In Sweden applied field research has been performed as part of the International Stripa Project, which provides a basis for the various projects conducted at the Äspö HRL in Sweden.

The behaviour of <u>backfill material</u> with low bentonite content has to a minor extent been investigated in projects related to investigations of the behaviour of buffer materials. Not very much was known of the behaviour of backfill material with a bentonite content of 10%-30% under field conditions that prevail in deposition tunnels at commencement of the test. The most detailed information on the saturation process in the integrated backfill and near field rock system was obtained from the Buffer Mass Test in Stripa /1-5//1-6/.

Laboratory investigations of the properties of bentonite mixed backfill and buffer materials have been made by e.g. DOE, AECL, and SKB. A literature and data base study of such work has been made by Daoman and Ran /1-7/. Several investigations of the hydraulic behaviour of bentonite-mixed sands as clay liners for bottom or top sealing of waste management facilities have also been made. A compilation of this work has been made by U.S. Environmental Protection Agency /1-8/. One conclusion of this work is that the field properties may differ substantially from the properties measured in the laboratory.

The design, behaviour, and requirements of <u>plugs</u> in a repository have been sparsely investigated. The function was to some extent investigated in the Shaft and Borehole Sealing Project in Stripa /1-9/. Numerous plugs have been built for other purposes e.g. water power stations, underground defence plants, and gas storage caverns. Such plugs have been studied and the experience taken into account in the proposal of the design of the plug for the Backfill and Plug Test /1-10/.

1.5 Preparatory tests

Preparatory field tests and supporting laboratory tests with backfill materials were made in 1995 and 1996 within the framework of a project named "Field Test of Tunnel Backfilling".

The field tests /1-11/ comprised the three main engineering activities required for backfilling tunnels in a repository, namely

- crushing of TBM-muck for ballast material (1600 tons)
- preparation of processed backfill material by mixing ballast material, bentonite, and water in different proportions (1025 tons)
- transportation, emplacement, and compaction of the processed backfill material and unprocessed TBM-muck

Moreover, the tests comprised the following activities:

- development and testing of compaction tools and techniques
- continuous quality control with measurement of density and backfill composition
- excavation of the backfill for examination after termination of the field tests

The following preliminary conclusions were drawn from these tests:

- Crushing of TBM muck to desired grain size distribution was successful.
- The procedure that was developed to mix bentonite, crushed rock, and water was successful.
- The technique to compact horizontal layers yielded problems in wet areas, that could not be solved
- The technique to compact inclined layers was successful but the compaction technique close to the rock needs to be improved. The following observations were made:
    - Very good compaction results were achieved
    - Acceptable results were achieved also in wet areas
    - Improved quality and design of the vibrating plate is required
    - Less good results were achieved close to the rock
    - Supplementary compacting may be required close to the rock
    - An open space at the roof is inevitable if backfill without bentonite is used
- Emplacement and compaction of backfill material in very wet areas may require special techniques
- Excavation of compacted backfill was successful

Laboratory tests for determination of important physical properties were made on mixtures of crushed rock and MX-80 bentonite with a bentonite content varying between 0 and 30%. The ballast material was taken from the TBM boring in Äspö and then crushed to a desired grain size distribution.

The laboratory tests comprised compaction tests, hydraulic tests, compression tests, and shear tests. Tests were made on samples compacted with different compaction effort and compaction techniques. The pore water composition and confining pressure were varied in the tests.

These tests were supplemented with investigations of the saturation and swelling processes of backfill materials with special respect to the pore water composition, degree of water saturation and wetting technique.

2 Project description and test geometry

2.1 General

The intention with the layout of the test is to investigate two types of backfill material. The inner test part is filled with a backfill material that is aimed at having sufficient swelling capacity and sufficiently low hydraulic conductivity to prevent the tunnel from being a preferential flow path in the rock. The outer test part is filled with a backfill of crushed rock with no bentonite added with mainly mechanical demands.

An updated overview time schedule for the Backfill and Plug Test, with preparations and planned continuation, is shown in Figure 2-1. A more detailed time schedule with the final outcome of the duration of different phases of the characterisation and installation is shown in Figure 2-2.

ENRESA is participating in the experiment and takes active part in the following work:

- Development of test design and planning as participants in joint technical project meetings
- Design and installation of in situ hydraulic conductivity measurement devices

Modelling with scoping calculations and predictions of the THM processes

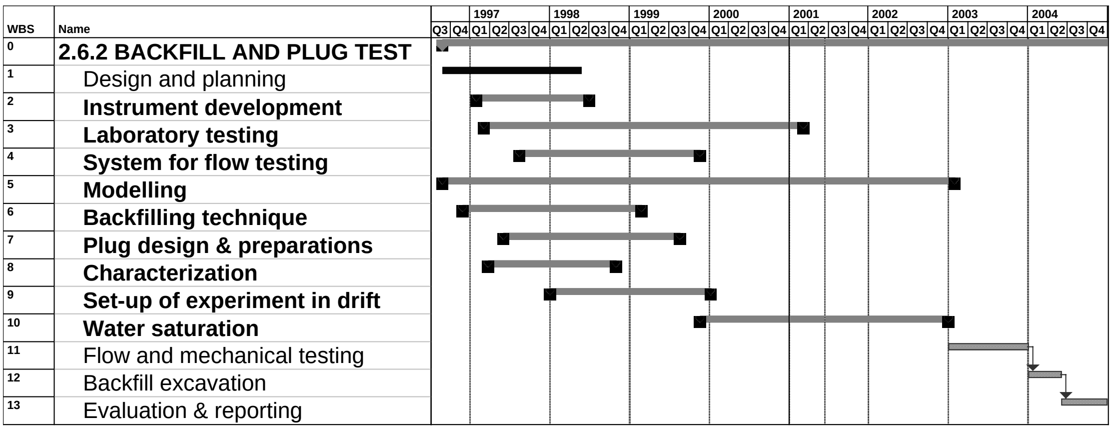


__Figure 2-1.__ Overview time schedule for the Backfill and Plug Test

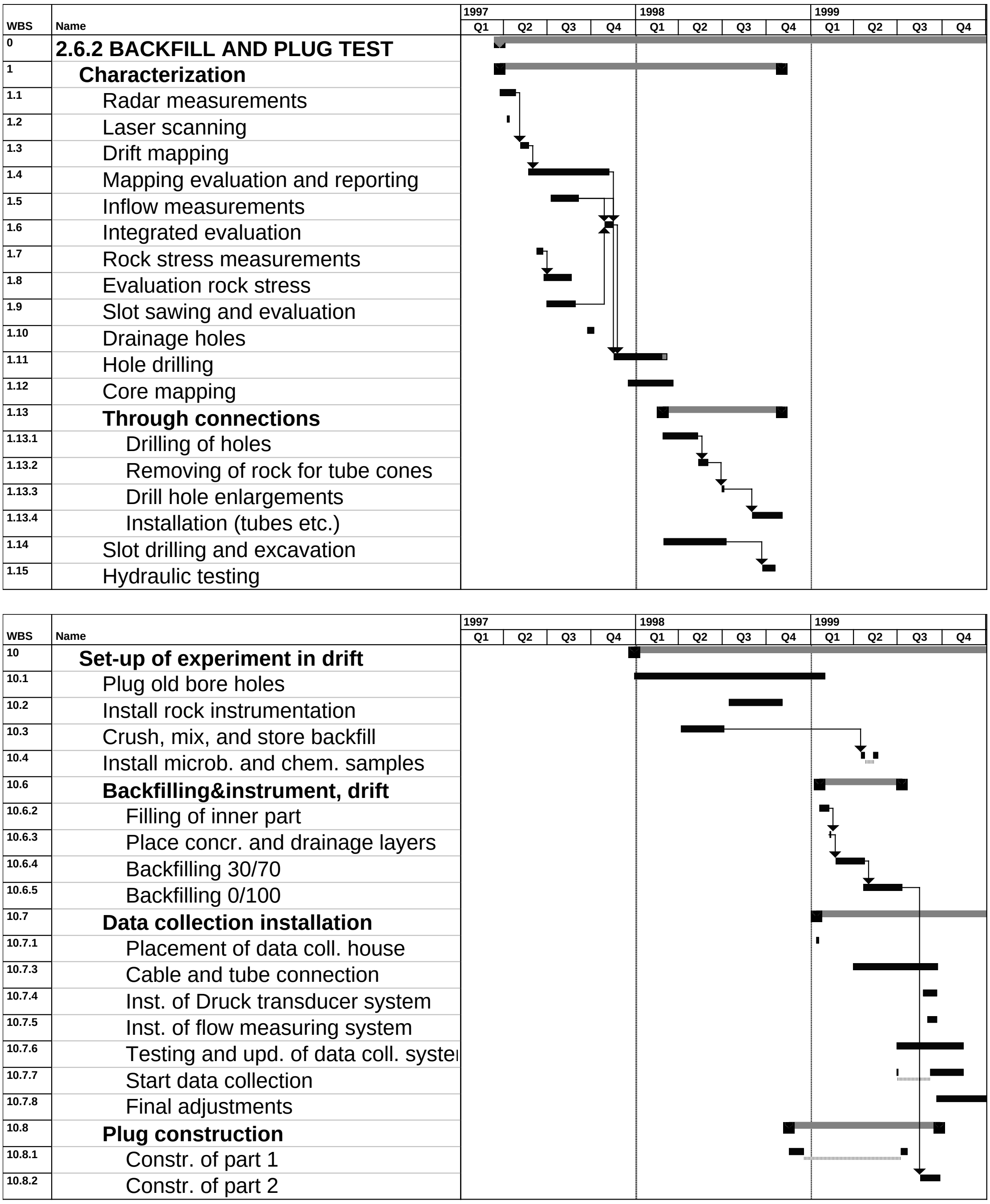


Figure 2-2. *Time schedule of the installation phase*

2.2 Location of experiments

The *Backfill and Plug Test* takes place in the so-called TAS Z tunnel. Figure 2-3 shows the test location of the *Backfill and Plug Test* and the other tests in the Äspö HRL.

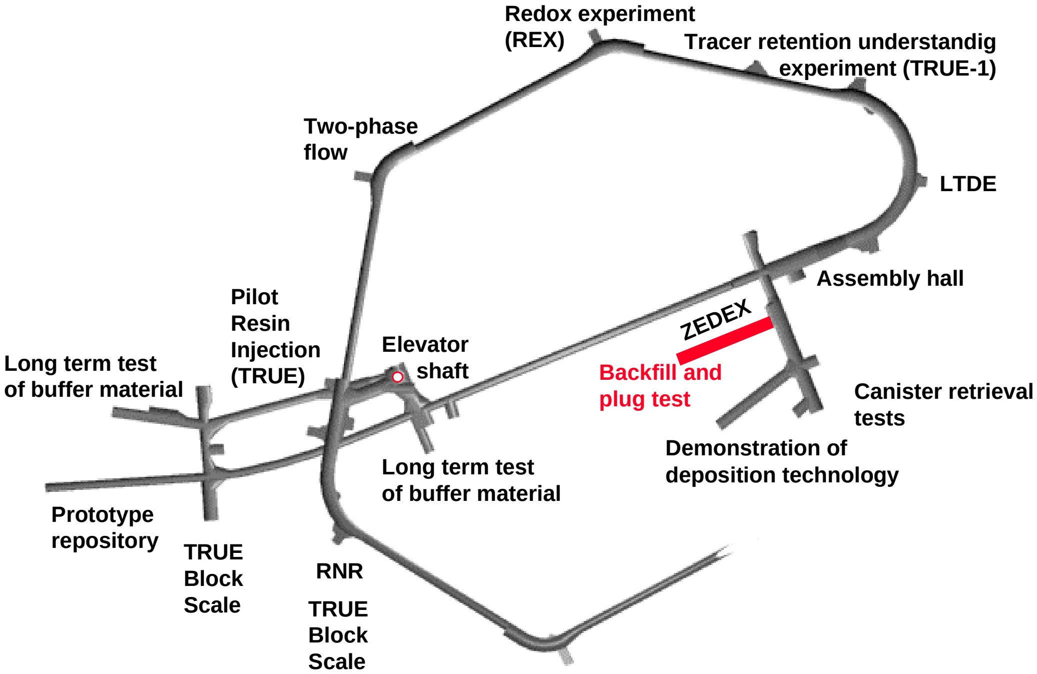


Figure 2-3. *Location of the Backfill and Plug Test and other tests*

In spite of some advantages of bored tunnels no decision has yet been made on whether the deposition tunnels will be excavated by boring (TBM) or blasting. By locating the *Backfill and Plug Test* in the ZEDEX tunnel and the Prototype Repository in a TBM drilled tunnel the difference in near field rock behaviour and difference in interaction between the rock, backfill and plug can partly be studied.

2.3 Test overview and processes to be studied

Figure 2-4 shows an overview of the *Backfill and Plug Test*. The test region, which is located in the outer part of the TAS Z (ZEDEX) tunnel, can be divided into the following three test parts:

1. The *inner part* filled with backfill containing bentonite.
2. The *outer part* filled with backfill without bentonite.
3. The *plug*.

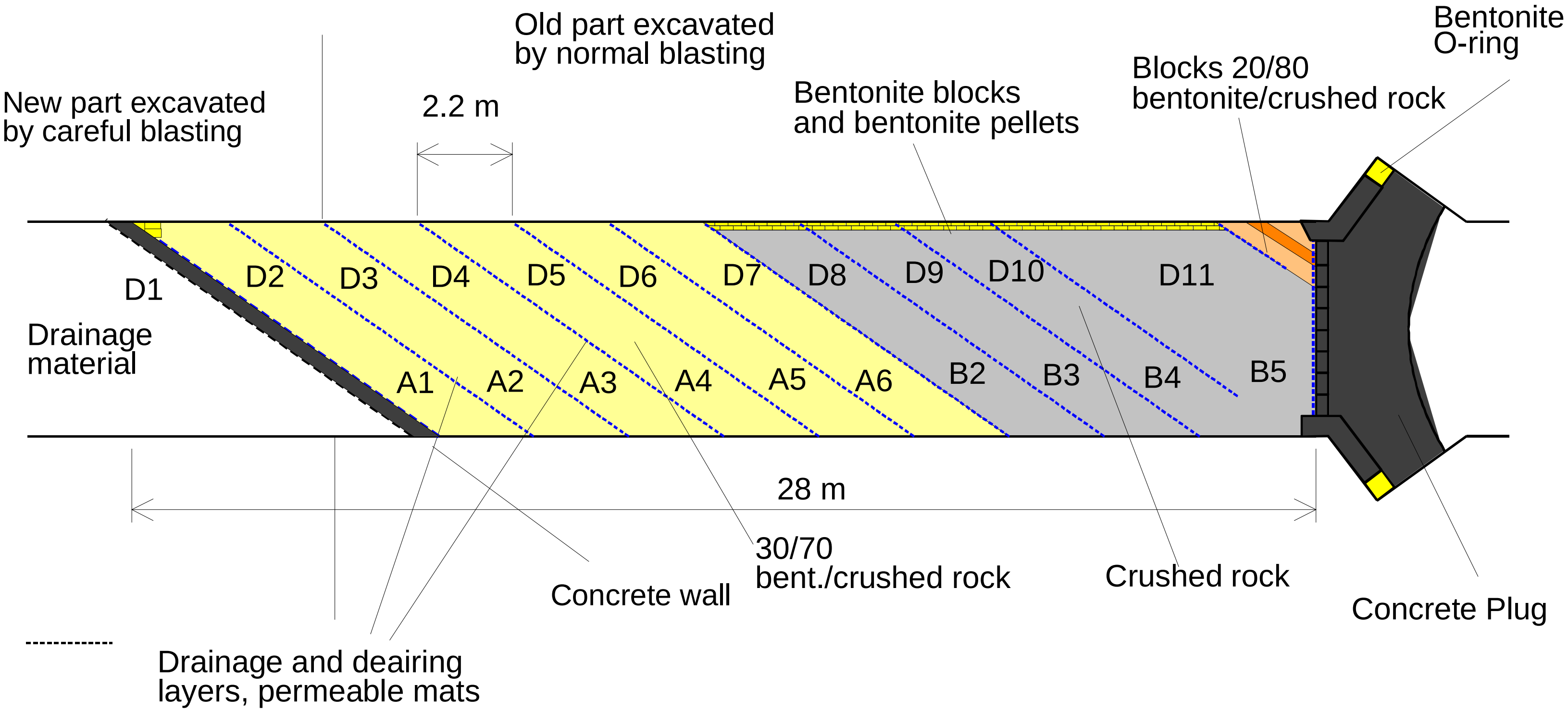


Figure 2-4. *An overview of the test region of the Backfill and Plug Test*

The innermost part of the tunnel will not be used for any tests and was only filled with drainage material. The tunnel sections were filled layer-wise with backfill and compacted using compaction equipment that had been developed and built for this purpose. It was concluded from the *Field test of tunnel backfilling* that inclined compaction should be used in the entire cross section of the tunnel from the floor to the roof and that the inclination should be about 35 degrees.

The inner test part was filled with a mixture of bentonite and crushed rock with a bentonite content of 30%. The composition has been determined after extensive laboratory and field compaction tests. The outer part was filled with crushed rock with no bentonite additive. Since the crushed rock has no swelling potential and will settle with time, a gap of a few decimetres width was left open between the backfill and the roof and was filled with a row of highly compacted bentonite blocks. This was made in order to ensure tight contact between the backfill and the rock. Open joints between these blocks and the roof were filled with bentonite pellets.

The two parts are about 14 meter long each and separated by drainage layers of permeable mats for application of water pressure and measurement of flow through the backfill and near-field rock. They are also used for water saturation of the backfill.

The material in the outer part is filled against a retaining wall of prefabricated bars for temporary support of the backfill before casting of the plug. Since in situ compaction of the backfill could not be made in the uppermost part, this space was instead filled with blocks of bentonite/sand mixture with 20% bentonite content.

The *plug* was designed to resist water and swelling pressures that can evolve. It was equipped with a filter on the inside and a 1.5 m deep triangular slot with an ”O-ring” of highly compacted bentonite blocks at the inner rock contact.

Saturation of the bentonite-containing components is expected to take a few years and the subsequent flow testing about 1 year. The backfill should be completely saturated before flow testing but the plug may be tested in advance. The test procedure will comprise reduction of the pressure in the filters one by one, while recording the flow.

The installation was preceded by characterisation of the near-field rock around the test tunnel, both in the new and the old part. It comprised design, manufacturing and testing of improved compaction equipment, investigation of instruments for measuring water pressure, total pressure, degree of saturation etc., and further investigations of the mechanical and hydraulic properties of unsaturated and saturated backfill materials.

Equipment for sampling of backfill pore water has been installed in the backfill and connected to the outside of the plug with thin tubing in loops that enable circulation of the pore water for reproducible sampling. Analysis of gas composition, water chemistry and microbial diversity can be done in each of the backfill types.

The following main *engineering issues* have been tested:

- Backfill production, preparation and emplacement
- Inclined backfill compaction
- Plug design and construction
- Quality control of the backfill composition and density

The following main *scientific issues* will be investigated:

- The hydraulic behaviour of field compacted backfill
- The hydraulic function of the integrated system of backfill and near field rock
- The mechanical and hydraulic function of the plug and its interaction with the near field rock.
- The water saturation process in the backfill
- The mechanical behaviour of backfill materials and their interaction with the rock
- The diversity, distribution and activity of native and introduced bacteria in backfill.

2.4 Expected outcome

The overall expectations of the *Backfill and Plug Test* are the following:

- Increased knowledge of the behaviour of backfill materials in a repository.
- Developed and tested techniques for backfilling tunnels excavated by blasting.
- Improved understanding of the mechanical and hydraulic behaviour of backfill and the interaction with the rock in a blasted tunnel and how it can be measured.
- Experience from construction of a properly designed temporary plug and understanding of its performance and sealing power.
- A basis for future selection of useful instruments and sampling techniques.
- Improved understanding about the evolution of pore water chemistry and microbiology in different backfill materials.

2.5 Problem areas

The *Backfill and Plug Test* may comprise several problems. The following major ones were identified in the beginning of the project and the actions listed below will be taken to avoid or minimise these problems:

- <u>Problem</u>: To reach a sufficiently high density and tight contact between the backfill and the roof may be difficult.
- <u>Action</u>: Special equipment for compaction close to the roof was built.
- <u>Problem:</u> The flow data at the hydraulic testing of the integrated function of the backfill, near field rock, and plug may be difficult to evaluate.
- <u>Action</u>: The hydraulic testing will be preceded by detailed calculations.
- <u>Problem:</u> Leakage along cables or along the rock surface may jeopardize the possibility to make relevant flow tests.
- <u>Action</u>: All cables were enclosed in tecalan tubes, which are believed to be non-leaking.
- <u>Problem:</u> Piping during artificial infiltration or flow testing may also have a detrimental effect on the results.
- <u>Action</u>: High gradients will be avoided if possible.
- <u>Problem:</u> The backfill with high bentonite content may not be saturated within reasonable time, which may complicate the evaluation or require an extended test period.
- <u>Action</u>: The time will be minimised by artificial saturation using the mats. It is difficult to further accelerate the water saturation without applying high gradients but an extended test period may be needed.
- <u>Problem:</u> Entrapped air in the backfill may prevent complete saturation.

- Action: This will take place in a deep repository. It is avoided as much as possible in the Backfill and Plug Test by discharging air from some of the mats and through filter equipped tubes.
- Problem: The intended hydraulic function of the plug may be difficult to reach.
- Action: If leakage takes place grouting may help. Contact grouting of the interface between the plug and the rock may be done through pre-installed tubes. Furthermore a high water pressure may be maintained by pressurising the filter mat applied to the inner surface of the walls.

2.6 Final test geometry

In the original layout the first five sections consisted of backfill material with a bentonite content of 30% (30/70). The remaining five sections were planned to consist of bentonite free material (0/100). The 30/70 sections and the four first sections of the 0/100 material would consist of six 20 cm thick layers.

The sections containing 30/70 were labelled using the letter A and adding a number. The first section to be backfilled was hence named A1. The sections containing 0/100 were called B with a number.

The compaction at the roof in the first three sections of 30/70 was not satisfactory (see chapter 7 and 8). Since it was considered important to have at least two correctly made 30/70 sections it was decided to replace the first 0/100 section (B1) with 30/70 material (now A6). The final test layout is shown in Figure 2-4. The labelling of the subsequent sections of 0/100 material were not changed but remained B2 to B5.

The location of the backfill sections and permeable mats was pre-determined by the location of the lead-through entries in the backfill tunnel. The weight of each backfill layer was estimated before the compaction. Two sections had to be adjusted so that the intended geometry could be applied. Section A2 was enlarged to consist of seven layers and section A6 was reduced to 5 layers. The last section 0/100 consisted of 12 layers.

Figure 2-5 shows a 3D visualisation of the experimental set-up.

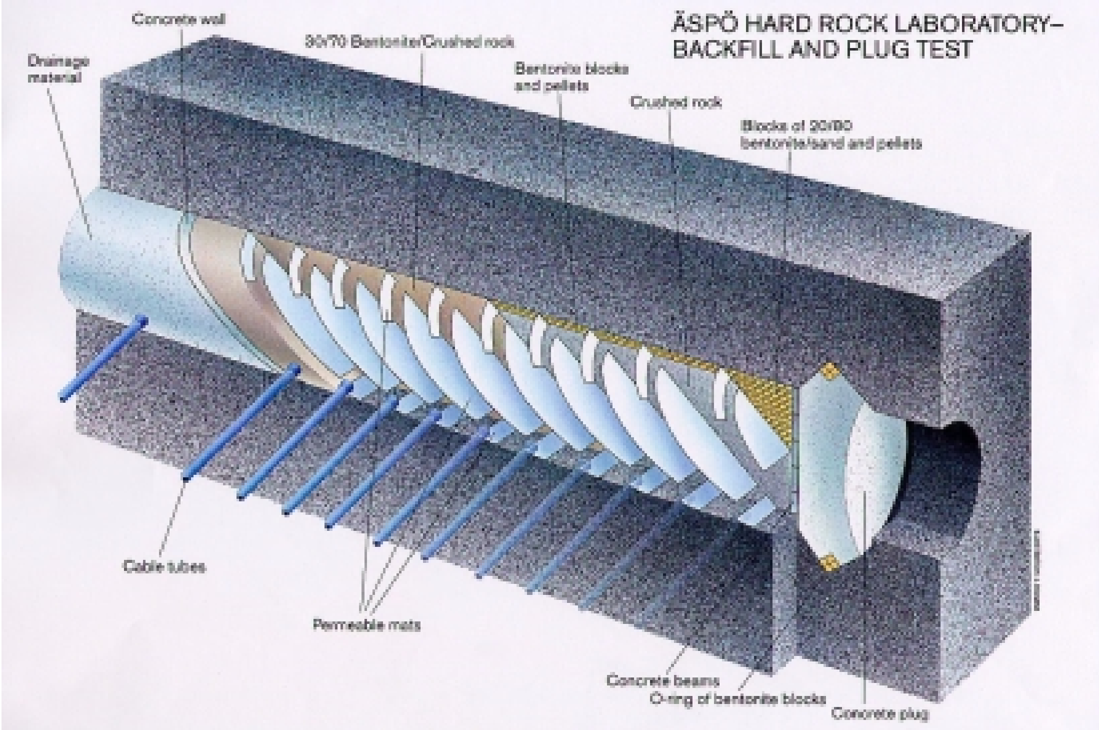


Figure 2-5. *3D-visualisation of the experimental set-up.*

3 Characterisation of the rock

3.1 General

The text in Chapters 3.1-3.6 is from /3-1/.

The TAS Z tunnel was excavated in January 1995 with the objective to study the excavation disturbed zone around blasted and bored tunnels /3-2, 3-3/. A total of eleven rounds were blasted with a cross section of 17,7 m^2 and a total length of 38,5 m. Two different excavation techniques were used:

- Low shock energy smooth blasting (LSES) for the first rounds, i.e. rounds 1–4.

- Normal smooth blasting for the next 5 rounds, i.e. rounds 5-9

In August 1996 the drift was excavated by blasting four additional rounds (16 m) with the objective to test another excavation technique and to make the drift longer. The excavation was made with more careful blasting. The number of contour holes was 61-64 with a c/c distance of 20 cm for 3 rounds and 43 with a c/c distance of 30 cm for one round (compared to 19 with c/c distance of 690 cm for NS blasting). Every second contour hole was charged in the first 3 rounds and every other hole in the fourth round. Every round was excavated in 2 or 3 parts with the contour holes blasted in the second or third part. The drift was extended to get a total length of 55 m.

The following methods were used for characterising the rock volume around the tunnel.

- Geological mapping
- Sampling for physical parameters
- Stress measurements
- Tunnel radar
- Laser scanning
- Water inflow measurement
- Hydraulic testing

3.2 Geological setting and structural model

The geological interpretation was based on tunnel mapping data. Core logging data and results from borehole TV, together with results from the seismic and radar investigations also contributed to the interpretation.

The rock in the backfill tunnel consists of Äspö diorite with veins and inclusions of fine-grained granite and pegmatite. Oxidation of wall rock around fractures is also quite common. The main fracture set trends NW with steep dips. These fractures are also the main water bearing structures

The first 38 m of the ZEDEX tunnel contins two water-bearing NW striking fractures, one at the start (30m) and one at about 63 m (Figure 3-1). The inner part of the tunnel (70-80 m) contains four to five water bearing fractures. The main water inflow to the tunnel comes from these fractures. TV logging showed an orientation of the fractures (170 fractures) with a strike of 139° from Äspö North and a dip of 79°towards the west.

From tunnel mapping most of the fractures appear to have a strike length of a few metres only, although some persist over tens of metres. The fracture infill materials are generally calcite or chlorite and less frequently epidote. Oxidation products and grout are also seen.

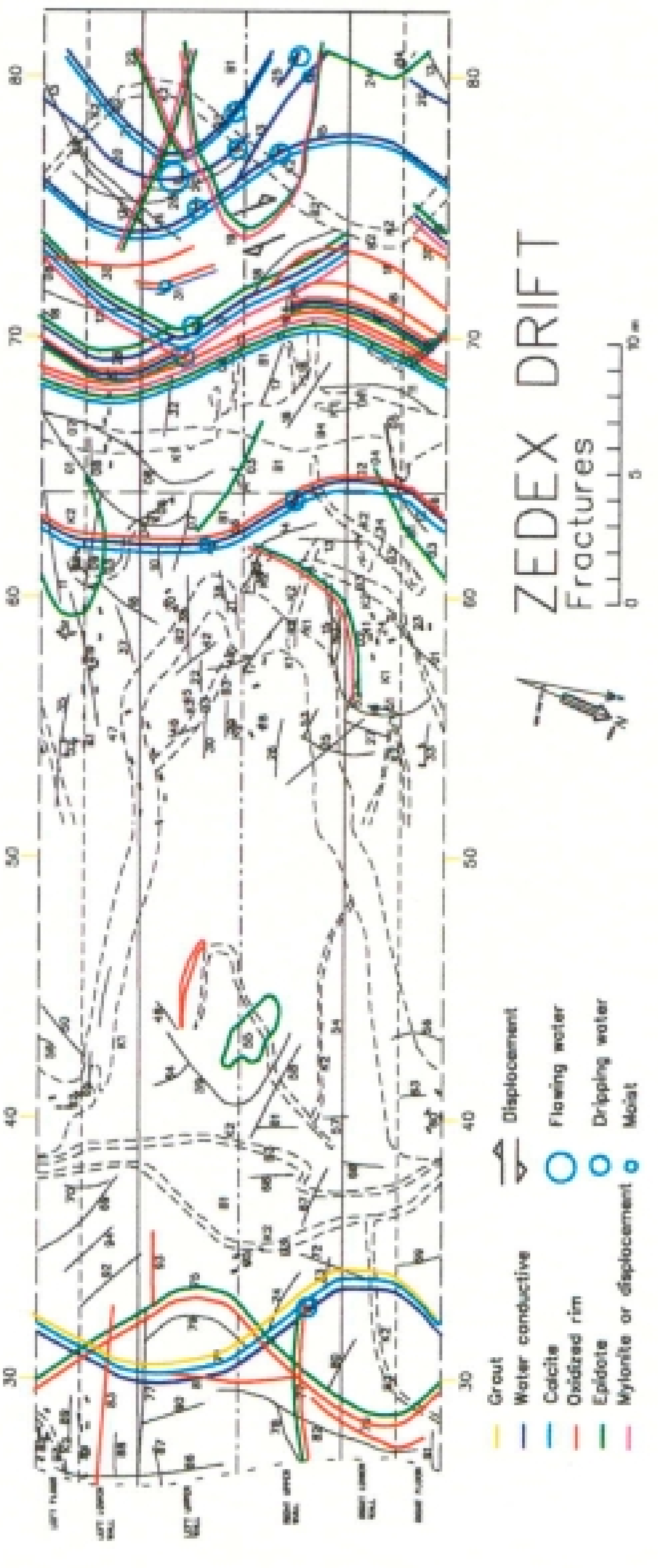


__Figure 3-1.__ Mapping of fractures in the ZEDEX tunnel

Figure 3-2 shows a structural model of the rock at the –420 m level covering the test site and the neighbouring tunnels and drifts. Only the fractures that are water-bearing have been considered. However, there is considerable uncertainty of the length and interconnection of the individual fracture planes.

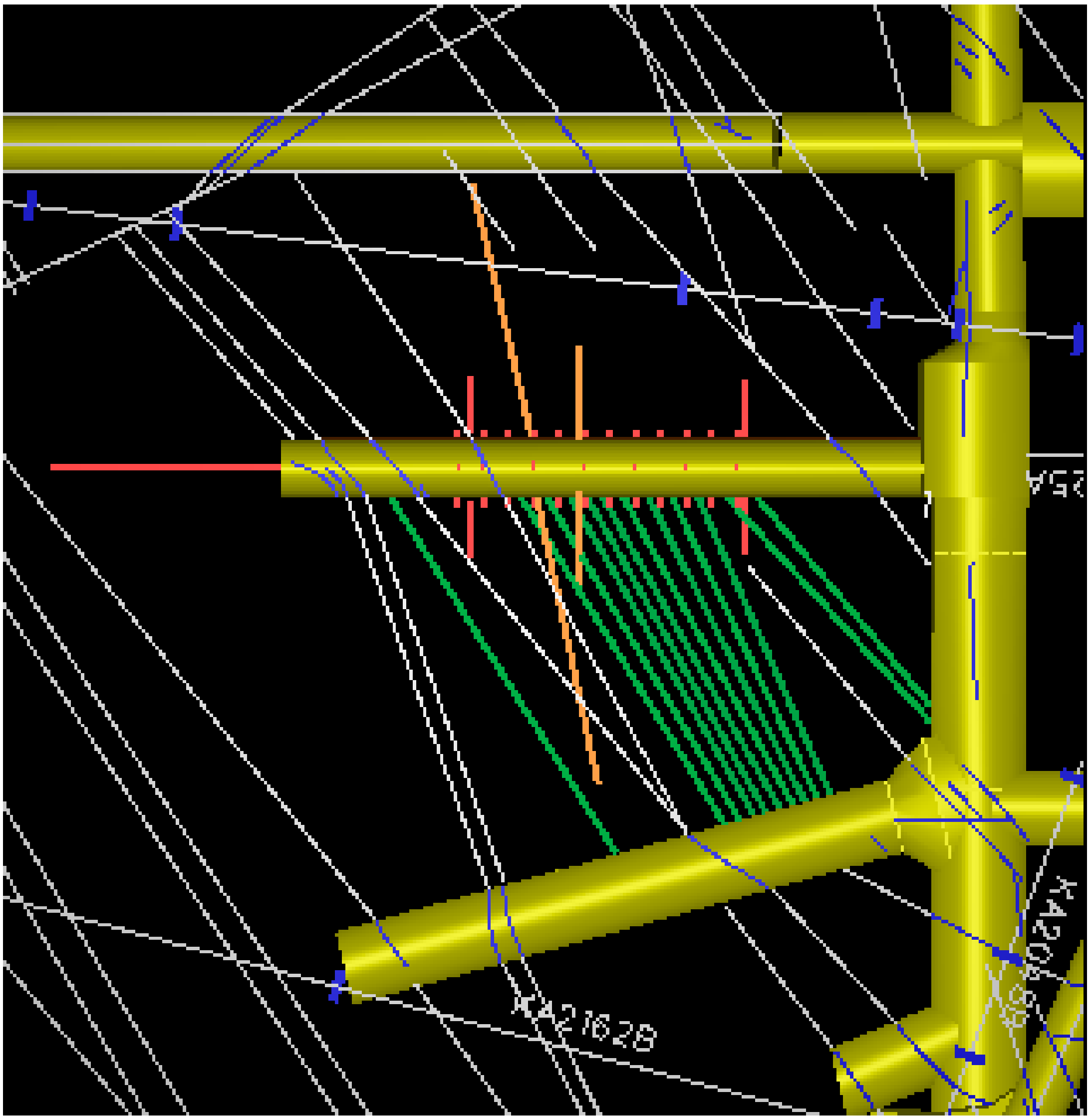


Figure 3-2. *Structural model covering the area between the Backfill tunnel and the demonstration tunnel at the –420 level. Blue lines are water-bearing fractures observed in tunnels and bore holes. Dashed lines are proposed interconnections between observed water bearing fractures in the tunnels. The red and yellow lines are boreholes for monitoring water pressure. The green lines represent the lead through boreholes.*

3.3 Geohydrology

3.3.1 Earlier tests

In the ZEDEX experiment a SEPPI probe developped by the Laboratorie de Mecanique de Lille together with the Laboratoire de Géomécanique de Nancy was used for doing high resolution permeability measurements. The analysed results were re-analysed and compared to the original analyses /3-2/.

The principal conclusions have been summarised /3-1/:

- The geomechanical conceptual model of he ZEDEX study indicated that the effects of the EDZ (Excavation Disturbed Zone) in hard fractured rocks manifest through propagation of discrete fractures which are either pre-existing (and re-activated) or newly developed planes of weakness. Hence, any change of hydraulic properties in hard rock due to EDZ is attributed to the enhanced aperture and/or connectivity of discrete features and not to deformation or alteration of the rock matrix itself.

- As the method was only applied in boreholes drilled after the tunnel excavation precludes a direct comparison of before/after *in situ* properties and as such severely limits the interpretation of the significance of observations. It is not clear weather the measurements reflect primarily the tunnel excavation effect (EDZ) or rather a wellbore skin effect.

- The results suggests that there is no well defined and significant increase in permeability of the rock mass in the damaged zone in the vicinity of tunnel excavation which could be observed systematically in the analysed data even when taking the uncertainties identified into account.

  Since all fractures were excluded in the test arrangements, these measurements only refer to the rock matrix.

3.3.2 Results from hydraulic testing of core-drilled holes

To get information on the hydraulic properties of the surrounding rock with fractures included hydraulic tests were performed in selected boreholes that were drilled for monitoring water pressure.

The tests showed that the hydraulic conductivity in the superficial rock (the measuring section was 0,3 –0,7 m perpendicular from the tunnel surface) was normally 5 X 10^{-8} or lower. For some tests the hydraulic conductivity was substantially higher and in some cases there was visual leakage through superficial fractures. These fractures were mainly located in the floor and probably were induced by the blasting of the tunnel. The blast holes in the floor are normally loaded with more explosives than the rest of the blast holes. This in combination with a less careful scaling of the floor could explain why most of the conductive fractures were found here. The results are shown in Figure 3-3.

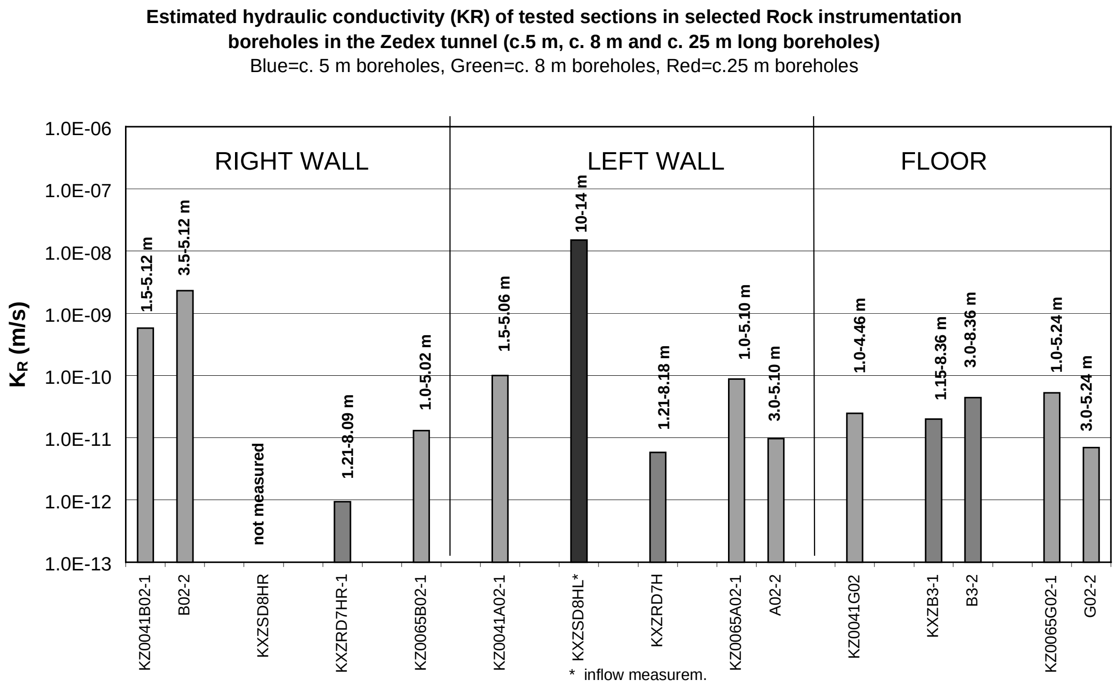


Figure 3-3. *Histograms of estimated hydraulic conductivity from the recovery phase (K_R) for the tested c. 5 m, 8 m and 25 m-boreholes at different locations in the Zedex tunnel. The intervals above the bars refer to the tested sections.*

3.4 Physical properties

Rock mechanical testing of cores from a borehole (KA3191F) drilled axially along and before the excavation of the TBM tunnel gave the values for the physical properties listed in Table 3-1 /3-1/. Measurements were later performed on cores from boreholes A3-A6 and C3-C6. The results from these tests are provided in Table 3-2 /3-1/.

Table 3-1. Properties of Äspö diorite from rock mechanical testing (KA3191F).

	Mean value	Standard deviation
Uniaxial compressive strength (MPa)	195	31
Young´s modulus (Gpa)	69	5
Poisson’s ratio	0,25	0,03
Tensile strength (MPa)	16	3
Internal friction (°)	45	3
Cohesion (MPa)	47	4

Table 3-2. Properties of Äspö diorite from boreholes KXZA-A6 KXZC3-C6

	Mean value	Standard deviation
Density (kg/m^3)	2,76	0,033
P-wave velocity (km/s)	5,57	0,16
S-wave velocity (km/s)	3,26	0,13
Young´s modulus, dynamic (GPa)	73	0,24
Poisson's ratio, dynamic	0,235	0,02
JCS, intact core (MPa)	41	22
JCS, Joint surfaces (MPa)	29	14
Uniaxial compressive strength (MPa)	169	14
Young´s modulus, static (GPa)	61	2
Poisson's ratio, static	0,22	0,04
Internal friction (°)	35	
Cohesion (MPa)	10-20	

3.5 Stress field

During the ZEDEX Extension Project /3-5/ stress measurements were performed in the ZEDEX area in order to obtain stress data which could be correlated with excavation effects. Over-coring stress measurements were performed using the Borre probe /3-6/. Measurements were made in boreholes KXZSD8HR and KXZSD81HR located in the pillar between the ZEDEX and TBM tunnels and in borehole KXZSDSD8HL extending south from the D&B tunnel (see Figure 3-1). The average principal stress magnitudes and directions obtained in the central part of the pillar (KXZSD8HR) and for undisturbed rock from measurements (borehole KXZSD8HL) are presented in Table 3-3 /3-1/.

Table 3-3. Average stress magnitudes and directions obtained in borehole KXZSD8HR and KXZSD8HL /3-5/.

Borehole No.	Stress Component	Magnitude (MPa)	Dip Direction* (Bearing °)	Dip (Plunge °)
KXZSD8HR	σ_1	19,6	336	8
	σ_2	10,7	71	37
	σ_3	10,3	236	52
KXZSD8HL	σ_1	20,1	351	24
	σ_2	9,2	230	49
	σ_3	7,8	96	31

*Bearing is calculated clockwise from the Äspö x co-ordinate (local north).

3.6 Laser scanning

Laser scanning of the tunnel was performed in April –97 /3-7/ with the main purpose to measure the exact geometry of the tunnel. The equipment was a vehicle-mounted TS 360 laser scanner from Spacetec.

The laser measurement resulted in visual images that gave good possibilities to locate and follow very fine fractures in the rock. At favourable conditions fractures with a length of about 10 cm could be observed /3-7/. However the wooden ramp, which was used for the wagon was not stable which caused oscillations and blurs on the images. This caused a slight problem in observing minor fractures and also in deciding where a fracture terminated /3-1/. Profiles from the laser scanning were used when determining a suitable location for the plug.

3.7 Inflow measurements

The Backfill tunnel was divided into 11 sections according to Figure 3-4 and the inflow was measured /3-1/. The result is presented in Figure 3-5

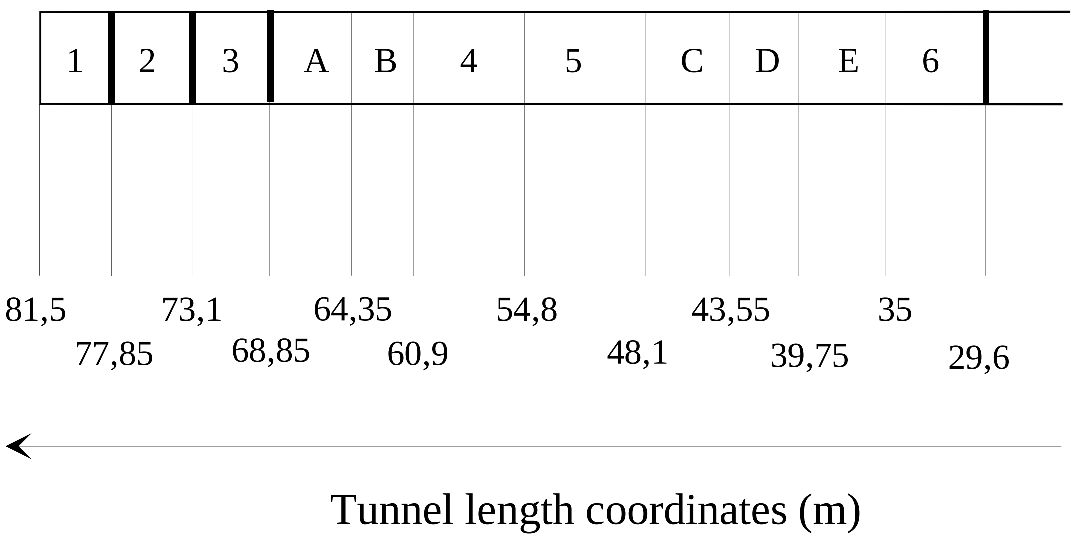


Figure 3-4. *Inflow sections. Bald lines represent concrete dams.*

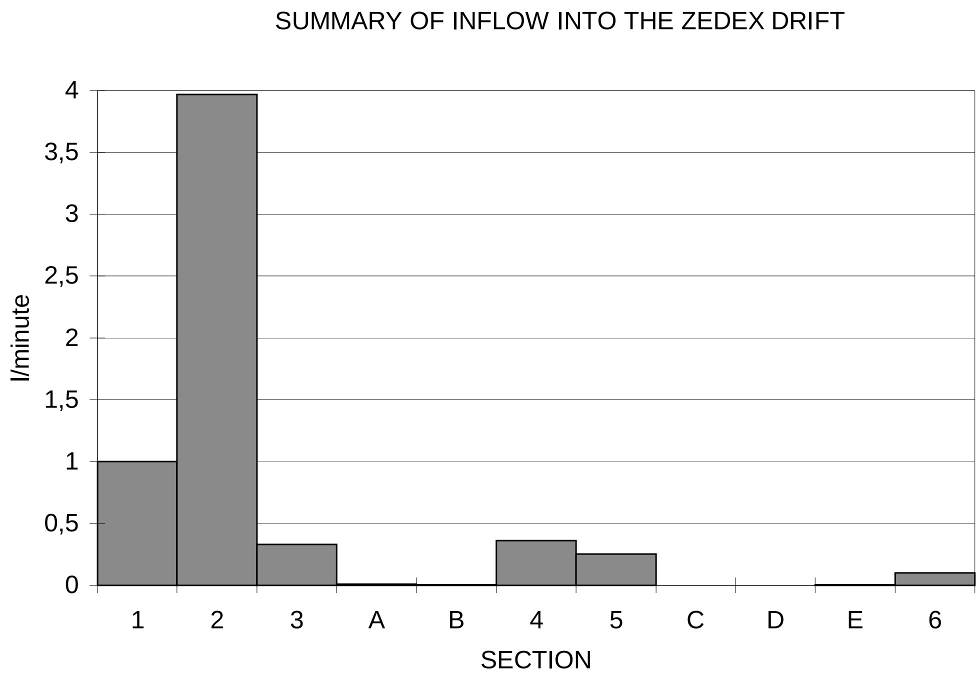


Figure 3-5. *Results from the inflow measurements.*

4 Mixing procedure and backfill material description

4.1 General

Two main types of backfill materials were produced for the test, namely crushed rock without bentonite (0/100) and crushed rock mixed with 30 % bentonite (30/70). In addition, pre-compacted blocks of pure bentonite, blocks containing 20% bentonite and bentonite pellets were manufactured. The bentonite blocks were used for filling the gap at the roof in the 0/100 section while the 20/80 blocks were used for filling the triangular space at the plug (Figure 2-4). The pellets were used for filling the remaining space between the bentonite blocks and the roof and for filling the space at the cable entries.

Two different ways of mixing the material were considered. One was to use a movable mixing station that could be placed at the HRL entry where the crushed rock was stored. The other was to use a stationary concrete mixer in Bockhara some 50 km away from Äspö and transport all mixing components there. The first alternative was chosen for economical reasons.

4.2 Criteria for choice of material composition

As reported in chapter 1 two types of backfill have been chosen for the test. One of these types is a mixture of bentonite and crushed rock that is intended to function both hydraulic and mechanical according to the requirements stated. The other backfill type is crushed rock with no bentonite added, which only is intended to have a mechanical function. The exact composition and demands of the backfills are based on the following criteria:

Criteria of 0/100:

1. Mechanical support in the roof with a swelling pressure of 100 kPa
2. Low compressibility, in order to avoid too much swelling of the buffer material in the deposition holes.

The first criterion cannot be fulfilled since the crushed rock is not swelling. Instead compacted blocks of bentonite will be placed between the roof and the backfill. For the second criterion a modulus of compression of M=10 MPa is required, which corresponds to a dry density of more than 1900 kg/m^3 /1-1/.

Criteria of 30/70:

1. Hydraulic conductivity lower than 10^{-9} m/s
2. Mechanical support in the roof with a swelling pressure of 100 kPa
3. Low compressibility (M>10 MPa), in order to avoid too much swelling of the buffer material in the deposition holes.

For fulfilling all these three criteria a dry density of at least 1650 kg/m^3 is required /1-1; 1-2/.

4.3 0/100

The original idea was to crush blasted rock from the Äspö tunnel to the desired grain size distribution ("Fuller curve" for optimal compaction properties) presented in Figure 4-1. However, normal crushing equipment could not produce the large amount of fines that was desired. Instead, two types of finely crushed granite, 0-2 mm and < 0,075 mm, were added to make up 19 respectively 7% (dry weight). The grain size distributions of the 0-2 mm fraction as well as that of normal crushing are shown in Figure 4-1. The resulting grain size distributions of the mixed material taken as an average from three different sieve analyses is compared to the desired one in Figure 4-2.

The water ratio was adjusted to be about 5% after mixing. Most values obtained during the actual mixing varied between 4 and 6 % as shown in Figure 4-3. The total weight of one mix was about 1200 kg. The recipe for one mix is presented in Table 4-1.

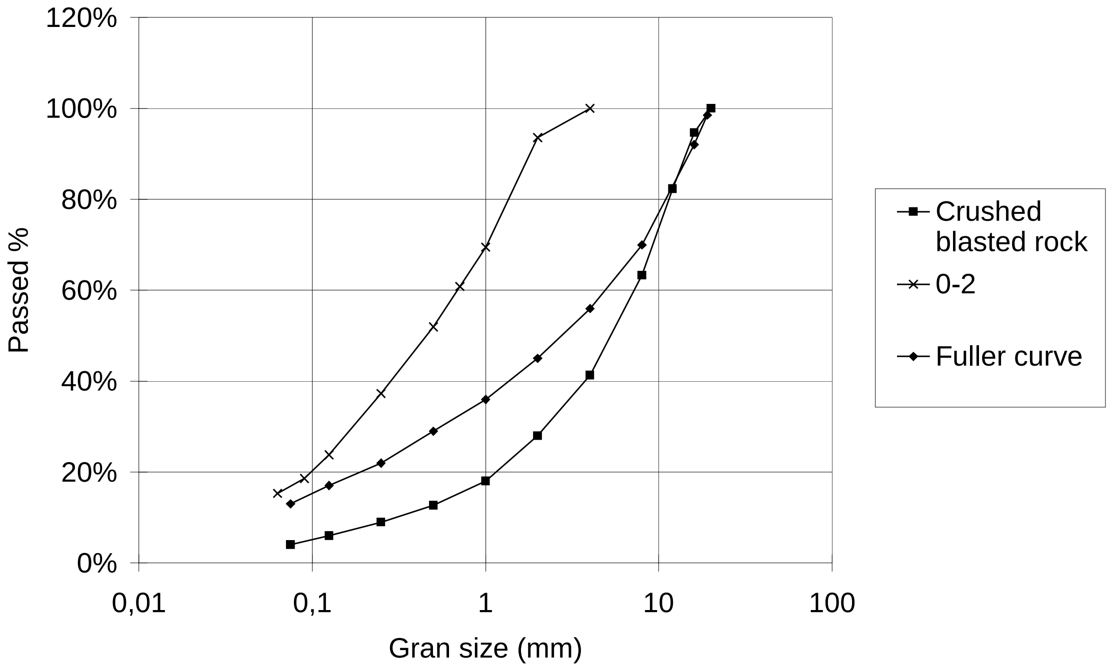


Figure 4-1. *Grain size distribution of the two main components of 0/100 and the desired Fuller distribution.*

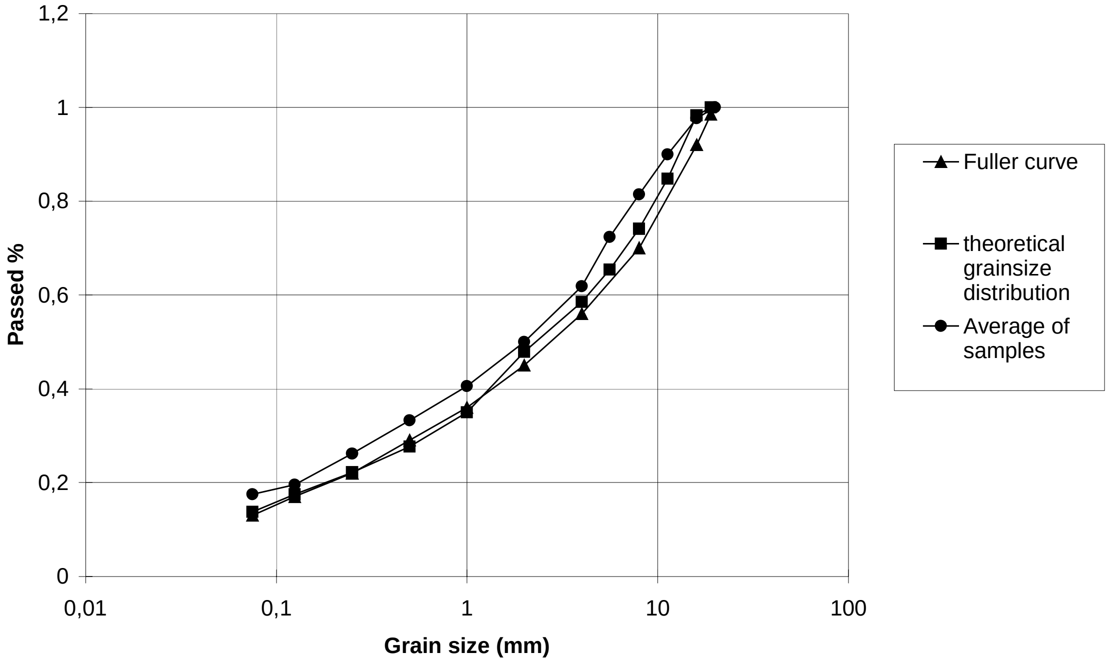


__Figure 4-2.__ Grain size distribution of the 0/100 material

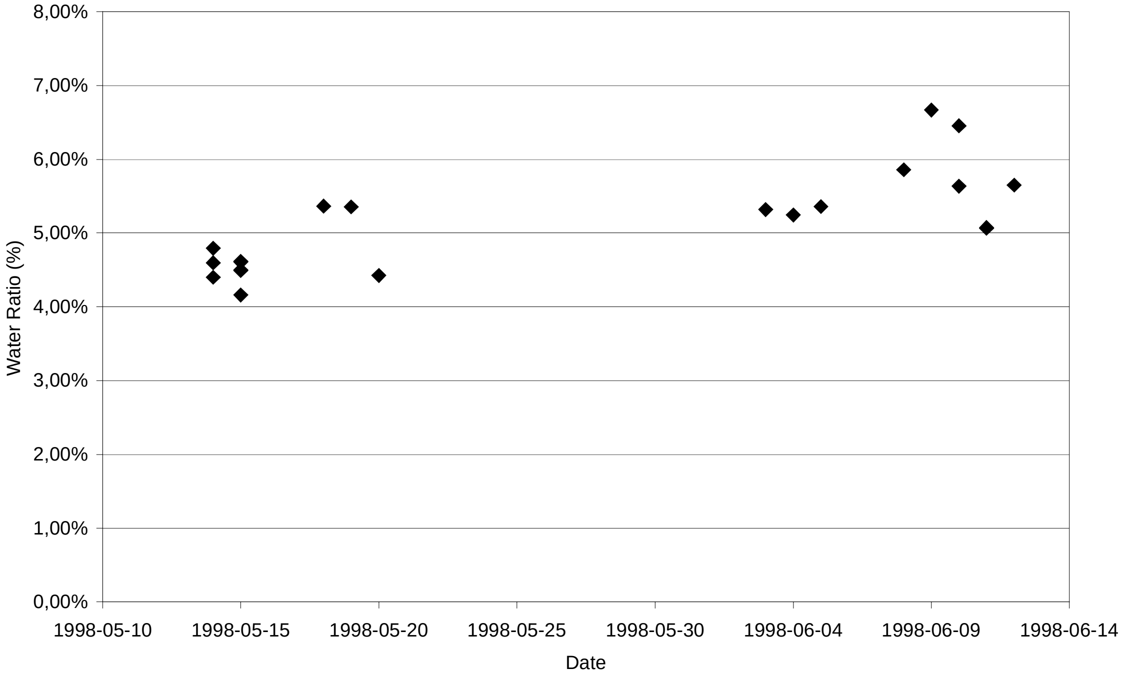


***Figure 4-3**. Measured water ratios of the 0/100 material at different mixing occasions.*

Tabell 4-1. Recipe for one mix of 0/100

Component	kg / mix
< 0,075 mm	86
0-2 mm	218
0-20 mm	870
Water	35

4.4 30/70

The 30/70 mixture consists of 70 % crushed rock (0/100), as described in the previous section, and 30% bentonite (dry weight). The grain size distribution of the 30/70 is shown in Figure 4-4. Water collected from the seepage water pumped up from ÄHRL was added for obtaining the optimum water content. It had a salt content of about 0.6%. The measured water ratio of each mixing batch is presented in Figure 4-5. The weight of each batch was about 850 kg.

Tabell 4-2. Recipe for one mixing batch of 30/70

Component	kg / mix
< 0,075 mm	58
0-2 mm	100
0-20 mm	403
Bentonite	248
Water	39

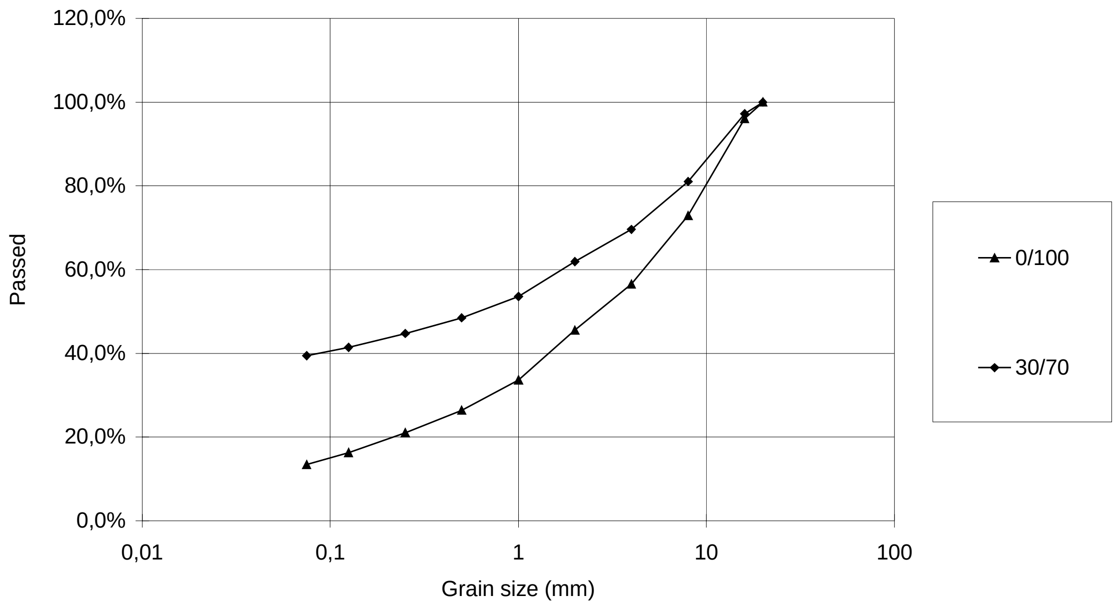


Figure 4-4. *Grain size distribution of 30/70.*

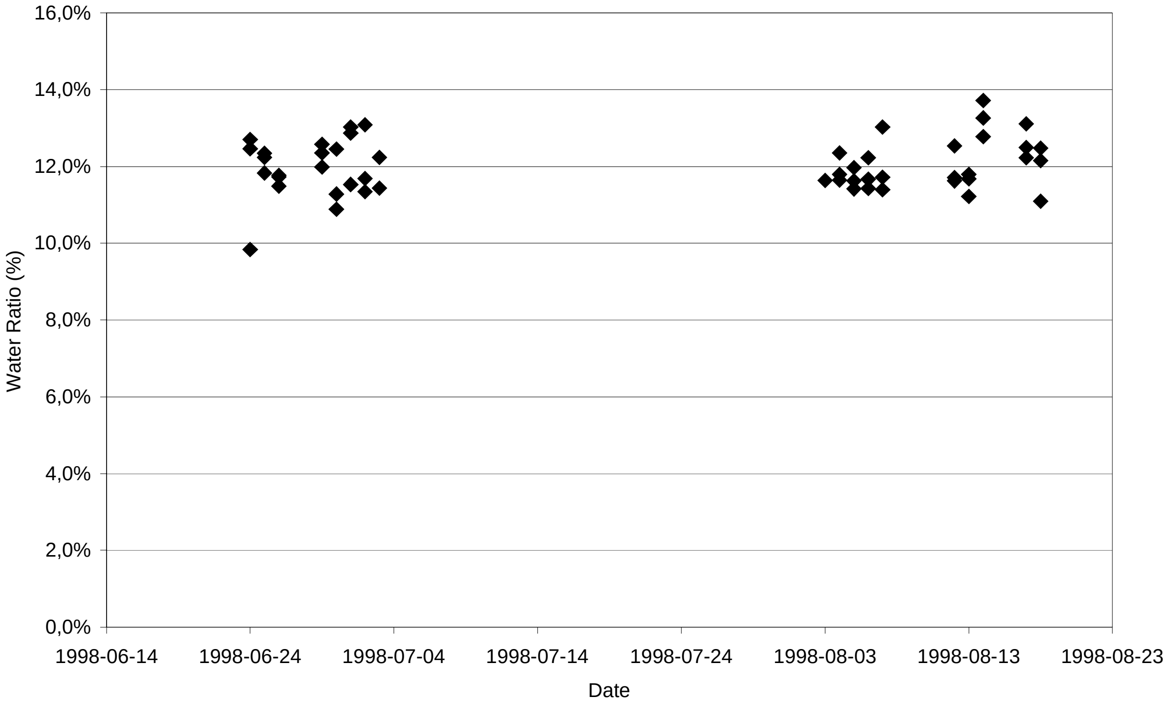


Figure 4-5. *Measured water ratio of the 30/70 backfill at different mixing occasions.*

4.5 Blocks

Blocks of different mixtures of bentonite and ballast (crushed rock) were manufactured for the backfilling of the Backfill and Plug Test. The following types were made

1. Blocks for emplacement close to the roof in the section where bentonite-free backfill material was used. These blocks were made from 100% bentonite.

2. Blocks placed in the triangular section close to the concrete plug. These blocks consisted of 20% bentonite and 80% ballast (sand)

The blocks were compacted uniaxially under a pressure of about 100 MPa using a press with movable upper and lower pistons. The lower piston was also used for expelling the blocks from the form. By compacting the blocks in three steps and lifting the piston from the upper surface of the block between the steps it was possible to minimise damage caused by entrapped air. The technique is described in detail in /4-1/.

Data for the different block types are summarised in Table 4-3.

Tabell 4-3. Data for the blocks used in the test.

Type	Weight (kg)	Dimensions (mm^3)	Water ratio (%)	Bulk density (kg/m^3)	Degree of saturation	Void ratio
1	3461	234 x 115 x 65	12,7	2031	0,649	0,542
2	3867	234 x 115 x 65	4,6	2278	0,541	0,228

4.6 Mixing procedure

The mixing took place close to the adit on the Simpevarp peninsula during May to August 1998. A total of 680 tons of 0/100 and 530 tons of 30/70 were prepared.

The mixing station was a movable concrete mixing station Eltron 1405, a paddle mixer powered by an electric 850 kW motor. A picture of the equipment is shown in Figure 4-6. The station was computer-controlled with preset data of the components. The mixing station is further described in SKB Progress report HRL-96-28 " Field test of tunnel backfilling"/1-11/

The mixing station and two tents for storage of the mixed material was established 300 m south of the ÄHRL adit. The tents were reinforced with boards on the inside, in order to increase the storage capacity. Asphalt was placed on the outside of the tents to stop rainwater from entering. Tarpaulins were placed over the mixed material to prevent drying and to act as an extra protection from rainwater in case the tent should leak.

Figure 4-6. *Mixing station*

A front-end loader was used for loading the mixing station and transporting the mixed material to the tents.

Before start the recipes for the mixing was adapted to the natural water ratio of the components in order to yield the final water ratios 5% and 12%, respectively.

The following mixing procedure was used for 0/100:

- Mixing of Äspö crushed rock and the fraction <0.075 mm for 3 minutes
- Addition of water by spraying during mixing for about 2 minutes
- Addition of fraction 0-2 mm
- Mixing for another 3 minutes

The mixing station is able to mix three different materials (gravel, sand and cement), water and additives. The mixer was programmed to add the material in a prescribed order: sand, cement, water, gravel and additives. Hence, Äspö crushed rock was first taken from the sand container, the < 0.075 mm fraction from the cement container and water was then added from the water tank. After the 3 minutes of mixing the finest fraction 0-2 mm was added from a separate container.

For the 30/70 the following mixing procedure was used:

- Mixing of Äspö crushed rock, bentonite, fraction 0-2 mm, and the fraction < 0.07 mm in this order
- 3 minutes of dry mixing
- Addition of water by spraying
- 3 minutes of wet mixing

To achieve the order stated above the Äspö crushed rock was placed in the sand pocket, the bentonite in the cement pocket and the fraction 0-2 mm in the gravel pocket. The fraction <0.075 mm was added from an external pocket and then the dry components were mixed for three minutes. The water was then added manually, not from the automatic water tank, and the mixing proceeded for another three minutes. This resulted in that the fraction <0.075 mm and the water was presented together as additive in a receipt delivered by the mixer. The amount of water added was therefore noted manually on the receipt. The receipts are stored at Clay Technology in Lund.

During the first days the water ratio of each component was determined, while during the rest of the mixing operation only one sample per day was checked. One sample of 1 kg was taken from each mixing round, numbered and dated. The samples are stored at Äspö.

Planning meetings were held regularly for checking results and adjusting the various techniques. The mixing procedure is described further in /1-11/.

4.7 Mixing rate

The mean mixing rate for 0/100 was 62 tons per working day. Two weeks were required for setting up the equipment and testing of the mixing procedure.

The mean mixing rate of the 30/70 material was 17 tons per day, which was hence considerably lower than for 0/100, the main reason being the difference in batch weight (see chapter 4.3). The mixing time for each mix was also slightly longer for 30/70, which tended to stick to the paddles and to the bottom of the mixing jar. This retarded the process and required cleaning of the mixer every second day. The average mixing rate of 30/70 was 17 tons per day.

The mixing and storage of backfill required the following equipment and personnel:

- one mixing station
- two storage tents
- one person for operating the mixing station
- one bucket loader (tractor) with driver

4.8 Bentonite pellets

4.8.1 General

Bentonite pellets were used for filling the open space at the cable entries, and for filling the gap left at the roof in the 0/100 section, as well as for filling a triangular space at the plug. Pellets were also used for sealing short boreholes and various voids in the walls and floor of the tunnel.

4.8.2 Test at BEPEX

An inventory of different techniques for manufacturing pellets was made and the inventory showed that the most suitable technique was roller compression. BEPEX-Hoskawas laboratory in Germany was chosen for performing pilot tests. The equipment is shown in Figure 4-7. The bentonite is compressed by the screw while the rolls rotate. The right roll rotates clockwise and the left anti-clockwise. The bentonite is compressed to pellets or briquettes between the rolls. It is possible to vary the roll speed and the pressure from the screw to change the density of the pellets.

The objectives of the tests were the following:

- To find out if the investigated method is suitable for manufacturing bentonite pellets
- To determine how the compression force affects the density
- To determine the influence of different water ratios in the bentonite.
- To investigate which density can be achieved.

The average density when the pellets are put in a limited space is much lower than the density of the pellet grains because of all voids between the pellets. This density is called bulk density in the subsequent text.

<u>Test 1</u>

In the first test bentonite with the natural water ratio 10 % was compacted into pillow shaped briquettes or pellets with the dimensions 13 x 13 x 6 mm. The compaction worked well and a pellet density of 2100 kg/m^3 was achieved. The press force between the rolls was 1640 kN/cm. The bulk density obtained when the pellets were poured into a jar was 1160 kg/m^3.

In order to investigate if the latter bulk density could be increased further the pellets were crushed to a particle size between 0 and 5 mm. This did not increase the bulk density, which instead decreased to 1090 kg/m^3.

When the pellets and crushed pellets were mixed 50/50, a bulk density of 1310 kg/m^3 was obtained.

Figure 4-7. *The BEPEX test equipment*

Six different tests were performed.

Test 2

In the second test bentonite pellets with the water ratio 18% were produced. This did not work well because the material stuck to the rolls of the compactor. The reason is believed to be that the applied wetting technique gave heterogeneous distribution of the water. Satisfactory results were obtained by reducing the press force between the rolls to 7.8 kN/cm, which yielded the pellet density 2005 kg/m^3

Test 3

The third test was made to investigate if low pellet densities could be achieved. The press force between the roles was decreased as much as was possible but still high enough to produce coherent pellets. The press force 7.8 kN/cm was used, which resulted in a pellet density of 2004 kg/m^3. The water ratio of the bentonite was 10%.

Test 4

Another test to compact briquettes with high water ratio was made. The bentonite had a water ratio of 14% but the same problem appeared as with the bentonite with 18% water content, the material stuck to the rolls. Like with the wetter bentonite, heterogeneous moistening may have been the reason.

Test 5

In this test new rolls were installed in the compactor, yielding almond shaped pellets with the size 30 x 20 x 12 mm. In this test the pellet density was not measured, but the bulk density was determined to 1012 kg/m^3. The pellets had a tendency to split in two equally large parts. The water ratio of the bentonite was 10%.

Test 6

The same rolls as in the previous test were used but the water ratio was raised to 14%. With this water ratio solid pellets with a density of 2000 kg/m^3 could be produced. The bulk density of the pellet filling was 1160 kg/m^3.

4.8.3 Summary and conclusions of the test

The manufacturing of 13 x 13 x 6 mm^3 pellets from MX-80 with the natural water ratio of 10 % worked well. A pellet density of 2100kg/m^3 or 1910kg/m^3 dry density was achieved, yielding a bulk density of 1160 kg/m^3 or a bulk dry density of 1005 kg/m^3. It is possible to increase the bulk density by filling the voids with fines. When crushed pellets and pellets were mixed a bulk density of 1310 kg/m^3 corresponding to a dry density of 1190 kg/m^3 was obtained. However, this material tended to separate.

Production of pellets with increased water ratios requires better mixing equipment than the one used. In most cases the bentonite with a higher water ratio stuck to the rolls, possibly due to inhomogeneity in water ratio. Higher water ratios seemed to improve the quality of the almond shaped briquettes (30 x 20 x 12 mm^3) but it also decreased the density.

The main conclusion was that the highest density and quality of the pellets was reached with natural water content, i.e. about 10%, and the highest compaction force.

4.8.4 Pellets manufactured for the Backfill and plug Test

Seven tons 13 x 13 x 6 mm^3 bentonite pellets were manufactured for the Backfill and Plug Test at Bepex test facility. MX-80 bentonite with about 10% water ratio was used and the average measured pellet density was 2009 kg/m^3.

5 Preparatory work in the tunnel

5.1 Introduction

Before start of the backfilling operations a lot of preparatory work had to be done in the test tunnel, especially in the rock. The following work will be described in this chapter:

1. General work
2. Plugging of old boreholes
3. Removing old extensometers
4. Boring of holes for monitoring water pressure
5. Installation of packers
6. Installation of pressure cylinders and total pressure cells
7. Sealing of slots earlier used for taking samples of the disturbed zone
8. Arrangement for decreasing water inflow into the inner part of the test tunnel
9. Backfilling and draining of the inner part of the test tunnel
10. Construction of the wall that separates the inner part from the test parts

5.2 General work

Before the backfilling operations began, ventilation and lamps were installed. The tunnel was scaled and the rock walls cleaned by high-pressure washing. Rock slabs that contained shallow axial boreholes were removed. A temporary road was built and the measuring house was installed.

5.3 Plugging of old boreholes

In the rock surrounding the Backfill and Plug Test tunnel a large number of boreholes had been drilled for earlier tests. Some of these boreholes could be used for monitoring water pressures in the rock during the Backfill and Plug Test. Most of the holes, however, had to be plugged. They had various diameters, 56, 76 and 86 mm, respectively. All holes that were plugged are listed in Appendix 1.

Boreholes longer than approximately 5 m were filled with cement in the inner part. The outer 4- 5 m were sealed with compacted bentonite. Boreholes with smaller lengths than 5 m were sealed with compacted bentonite alone.

The bentonite used for sealing was compacted to small cylindrical blocks with a height of 5 cm. The blocks were pushed into the boreholes with light rods. After installation, a rubber plug was applied in the borehole and the rest of the hole (about 50 cm) filled with concrete. The concrete acts as mechanical support.

The boreholes that were to be plugged are located in all sections of the tunnel. The only borehole that has not been sealed is KXZA5. The reason is that it is 41 m long and the water inflow to the hole so strong that bentonite blocks could not be inserted. The water comes from a fracture zone that intersects the backfill tunnel outside the plug and the hole is thus not connected to the test area. Because of this it has been decided that the entire hole can be plugged with cement. This will be done before start of the flow testing.

A packer for injecting the cement in the inner part as well as the outermost part of the boreholes was needed for all holes. Expanding cement was used for ensuring that the contact rock-concrete was good.

The bentonite blocks needed for the plugging were compacted at Höganäs Bjuf AB in Bjuv.

The plugging of holes longer than 5 m was made in the following manner:

- After draining the borehole a mechanical packer was installed at the outer end of the planned cement filling and cement pumped into the hole. The packer was left in the hole.
- After the cement had hardened the hole was emptied of water. The compacted bentonite blocks were pushed into the borehole with jointed rods.
- A rubber plug was pushed into the borehole for providing the expansive bentonite with confinement. In the holes directed upwards a locking device was used instead of a rubber plug.
- A mechanical packer was installed and the outermost part of the hole (0.5 m) was filled with cement in order to support the rubber seal. The outer packer was also left in the hole

A schematic drawing of a plugged borehole is shown in Figure 5-1.

Boreholes shorter than 5 m were plugged in the same way except for the innermost cement filling, which was excluded.

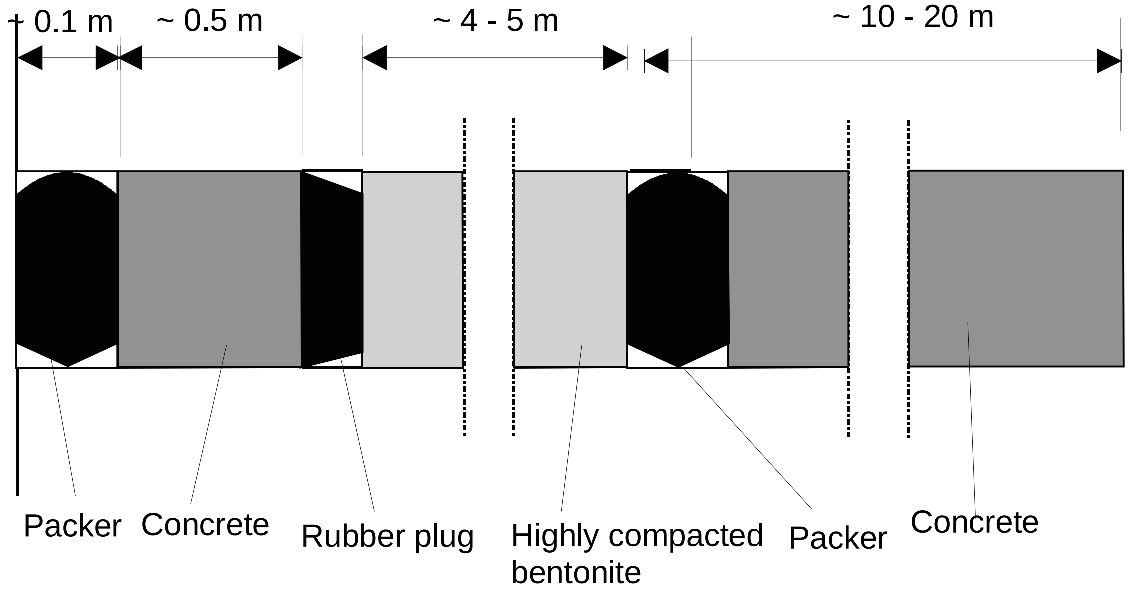


Figure 5-1. *Schematic drawing of a plugged borehole*

5.4 Removing extensometers

Before start of the Backfill and Plug Test the tunnel had been used for a variety of experiments. One was to measure the movement of the rock in the vicinity of the tunnel with extensometers /3-5/. Four extensometers had been installed in boreholes KZB5, KZB6, KZB7, and KZB8. B5, B7 and B8 were bored radially from the backfill tunnel while B6 was drilled from the TBM tunnel before the Backfill tunnel was excavated. The borehole extended into the Backfill tunnel and the end of the hole was consequently removed by blasting when the tunnel was excavated and thus came to connect the two tunnels.

All the boreholes were located in the same section of the tunnel. The positions of the boreholes are shown in Appendix 2. The extensometers consisted of 6 - 8 steel rods in plastic tubes. The rods ended in anchors that extended from the the plastic tubes. The volume left in the boreholes had been filled with grout that fixed the anchors in their positions.

The extensometers had to be removed and the holes sealed for avoiding hydraulic short-circuits. This was especially important for the KXZB6 borehole since this hole was connected to the TBM tunnel outside the test area.

The steel rods could be pulled out after the protecting casing had been removed. KXZB6 was emptied by using both percussion drilling and core drilling. The drilling started with percussion drilling until the first anchor was reached. It was taken out by over coring. The plastic tubes were not fragmented and had to be pulled out by heated metal hooks. However, this procedure was only used for KXZB6 because of the long time it required. For the rest of the holes, only the outer 30 cm were emptied and then injected with cement grout.

The percussion drilling was made by the company “Södermans sprängning” while the core drilling was made by NCC. The injection of cement was performed with the same technique as for plugging boreholes.

5.5 Boring of holes for monitoring water pressure

The water pressure in the rock surrounding the backfill tunnel will be recorded throughout the test period. The pressure is monitored in boreholes drilled radially, in the walls, roof and floor of the tunnel. It can be recorded at different distances from the periphery of the drift by use of split packers.

Most of the measuring sections were placed close to the tunnel in the disturbed zone. Eleven boreholes from earlier tests were used. Two 25 m long holes, 8 new 5 m long holes and 44 new 1 m holes were core-drilled for the monitoring of water pressure in the Backfill and Plug Test. The designations of the boreholes and their co-ordinates, inclination and bearing relative the tunnel are given in Appendix 3.

The drilling took place from November 1997 to January 1998.

The following drilling machines were used:

- 1 m holes: Pixie HB 400
- 5 m and 25 m holes: DIAMEC 251

The 1 m holes are placed in 11 sections. Each section consists of one hole in the floor, one in the roof, one in the right and left walls, respectively. The sections has a spacing of 2.2 m along the test part of the tunnel. The boreholes in the roof were given the same bearing as the tunnel and an inclination of 45° relative the tunnel. The boreholes in the walls were placed 1 m above the tunnel floor and inclined 45° with respect to the wall surface. The bearing was perpendicular to the tunnel axis. The boreholes in the floor were given the same bearing as the tunnel and 45° inclination, since the expected thickness of the disturbed zone is about 30 cm. The minimal packer length was 0.5 m. The inclination of the boreholes enabled measuring of water pressure in the disturbed zone. The direction of the boreholes in the walls also made it easy to de-air the holes after installing the packers.

The 5 m holes are also located in sections which consist of 4 radial holes, one in the roof, one in the floor, one in the right and left walls, respectively. One 5 m section was placed at each end of the test part of the tunnel.

Two 25 m holes were drilled: one vertically in the roof and one at the front.

No casing was required for the 1 m holes. For the 5 m and the 25 m boreholes casing were used since water-bearing structures with high pressure and flow could be intersected.

The drill cores were examined by the well site geologist who made a tentative mapping of lithology and structures. The cores were placed in boxes and are stored at the Äspö HRL.

5.6 Packer installation

5.6.1 General

The packers for monitoring water pressure were installed by Geosigma. The locations of the measuring sections and their designations are given in Chapter 11.3. The design of the packers is shown in Chapter 11.4

The bentonite packer technique worked well as long as they could be installed in one piece. Since the bentonite expanded very fast, there was not enough time to stop and mount a second section. The maximum section length that could be installed was thus less than the tunnel diameter. A picture of an installation of a 5 m packer is shown in Figure 5-2.

5.6.2 Installation procedure

The bentonite rings were mounted on the packers just before they were installed since the high humidity in the tunnel could otherwise cause cracks in the rings. The bentonite rings were stored in plastic bags until the emplacement of the packer.

Figure 5-3 shows the short packer. The packers were installed in the following way:

- The packer was placed in the borehole so that the filter on the steel tube was in contact with the bottom of the hole.
- The packer was fixed by expanding the rubber slightly.
- The position of the expansion shell bolts was marked and the holes for the bolts drilled.
- The expansion shell bolts were installed. A gap corresponding to the expected compression of the rubber (about 2-3 cm) was left between the steel and the rock.
- The rubber was further axially compressed and radially expanded with special tools.
- The expansion shell bolts were tightened.

The design of the packers is reported in more detail in chapter 11.4.

Figure 5-2. *Installation of bentonite packers*

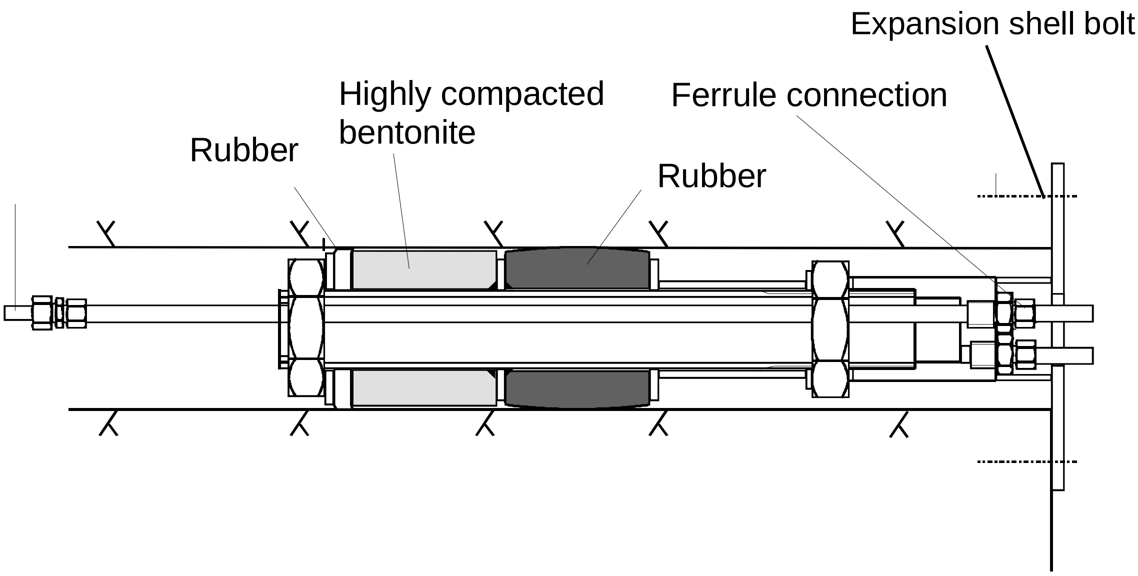


Figure 5-3. *The short packer*

5.7 Installation of pressure cylinders and total pressure cells

As a part of the instrumentation four pressure cylinders and five total pressure cells were installed adjacent to, or in the rock.

<u>Pressure cylinders</u>

The design is described in chapter 10.9.

Two of the pressure cylinders were installed in the roof and two in the floor. The positions are specified in chapter 10. When the hydraulic testing is completed, the cylinders will be activated. The pressure exerted by the cylinder and the deformation of the backfill will be recorded and the obtained information used for determining the mechanical characteristics of the backfill, particularly the compressibility.

Before installation the rock was excavated for countersinking of the cylinders.

The excavation was made by core drilling. A number of 20 cm diameter holes were drilled, which were then expanded to 50 cm by use of a large core drill.

The pressure cylinders were fixed in the rock with concrete. The boreholes in the roof were equipped with rock bolts to make sure that the concrete was actually anchored in the rock. The pressure cylinders were equipped with flanges to make sure that the cylinder was fixed in the concrete. A drawing showing the arrangements for the casting is given in Appendix 4.

Four rock bolts were installed in the rock around the excavation. Two steel beams were attached to the bolts and used as support for the form. The steel beams holding the form and the cylinders at the roof were left in place until the backfilling had progressed far enough to take the load of the cylinder in case it would separate from the concrete (Figure 5-4).

The cylinders in the floor were protected from the material in the temporary road with a plastic sheet.

Total pressure cells

The total pressure cells (Glötzl, of the same type as for the rest of the backfill) were installed for determining the contact pressure between the backfill and the rock wall.

The pressures cells were placed so that natural depressions in the walls of the drift could be utilized.

The total pressure cells in the roof were installed at the same time as the pressure cylinders. The ones in the floor were not mounted until the backfilling had proceeded to the respective installation. In this way, the risk of damaging the instruments was minimised. A picture of a total pressure cell installed in the floor is shown in Figure 5-5.

Figure 5-4. *Pressure cylinder mounted in the roof above the bentonite blocks.*

Figure 5-5. *Pressure cell mounted in the floor.*

5.8 Sealing of sawn slots

In earlier tests some samples of rock had been taken from the walls and floor of the tunnel by use of a large-diameter diamond saw disc. It was important that all the slots were properly sealed before the backfilling of the tunnel began. Some of the deep notches extended about 2 m in the axial and tangential directions and would thus be able to short-circuit two permeable sections.

The slots in the floor were simply filled with fine-ground expanding cement after they had been carefully cleaned. The deep notches were filled from the bottom and up (see Figure 5-6). A form was mounted and the main excavation was filled with normal construction concrete.

Figure 5-6. *Sealing of a slot*

5.9 Attempts to decrease inflow to the inner part of the tunnel

5.9.1 Description of activity

The ZEDEX tunnel was excavated in two steps. 38.5 meters were excavated in the first step (see chapter 3.1). This part was very dry compared to the rest of the Äspö tunnels. In the second step, the tunnel was extended to 54.5 meters (four blasting rounds) using very careful blasting technique. The water inflow in this 16 m long part of the tunnel was about 5.3 l/min (see Figure 3-5), which was unacceptably high. Experience from earlier field tests showed that backfilling with 30/70 material cannot be made with water dripping on the surface of the compacted layers section. This raises the water ratio, which makes the material difficult to handle and destroys the compaction properties. Even if it is possible to apply the backfill in a section with high inflow, the water pressure at the contact between the rock and the backfill may cause piping. In the field test it was possible to backfill sections with inflow of about 1 l/min, but this required continuous backfilling. In the Backfill and Plug Test the backfilling was interrupted by instrumentation, surveying and density measurements. The maximum acceptable inflow in the tunnel was therefore settled to 0.5 l/min.

In order to reduce water inflow, an attempt was made to drain the rock surrounding the inner tunnel. Boreholes were drilled through the water bearing fractures intersecting the tunnel. Tubes were installed in the boreholes for discharging water from the test area.

A standard hand-operated percussion boring equipment (Atlas Copco BBC V) with a pneumatic feeder (Atlas Copco BMP 33) was used for drilling the drainage holes.

Four-point bits gave a borehole diameter of 45 mm to 48 mm, the latter being required for using the Äspö HRL borehole camera (boreholes HZ0069A04, HZ0069A03 and HZ0069A02). The locations of the holes and their designations are shown in Appendix 5.

The measurements of the flow from the boreholes were made by measuring the time for a specified volume of water to flow out of the borehole. The boreholes were sealed with rubber plugs and the water led out through tubes.

The measurement of the water inflow into the inner part of the tunnel (excluding the water from the boreholes) was made with the same type of equipment that was used for recording the water inflow into the tunnel before the boring /3-1/. A water pump was placed at low elevation so that no water could pass (see Figure 5-7). The water was pumped to a plastic tank.

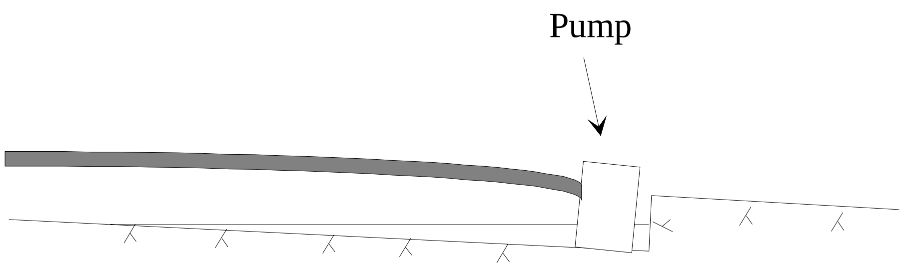


Figure 5-7. *Detail of the water inflow measurement system*

5.9.2 Results

The measured flow from the boreholes is shown in Figure 5-8, the total flow being 27.4 l/min. The main discharge was obtained from the boreholes in section 69. The main flow, about 17.5 l/min, was from hole HZ0069A04. The large difference in flow from the boreholes confirms the assumption that water moves in channels in the water bearing fractures. It is remarkable that the inflow increased by a factor 6 due to the new boreholes.

After boring and draining the holes, the remaining inflow to the inner part of the tunnel was measured (Figure 5-9). The mean inflow from 97-10-02 to 97-10-10 was 2.08 l/min.

5.9.3 Grouting

Since the inflow to the tunnel could not be reduced to a sufficiently low level, the boreholes were grouted. This resulted in an inflow of 2.4 l/min to the inner part of the tunnel, which was still too high.

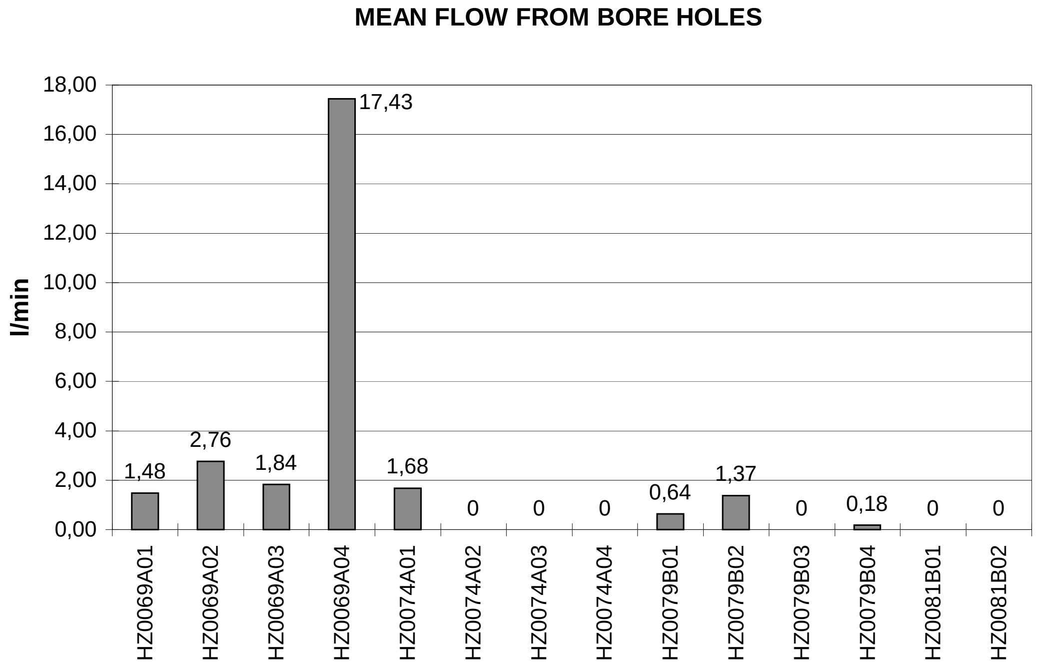


Figure 5-8. *Summary of the flow from the boreholes.*

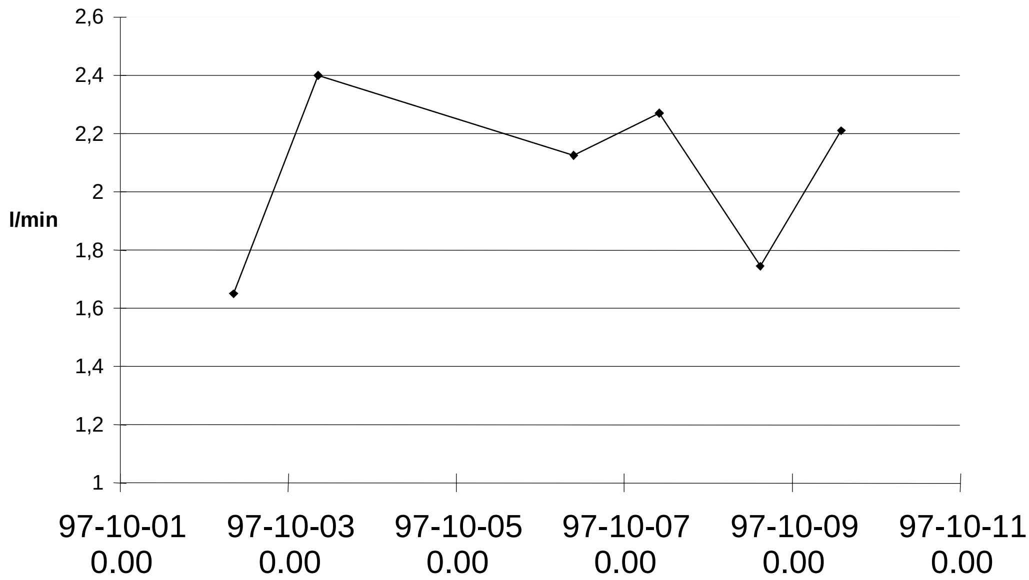


Figure 5-9. *Remaining inflow to the inner part of the Backfill tunnel.*

5.9.4 Effect on test geometry

The inner wet section of the tunnel could thus not be included in the Backfill and Plug Test area and was instead filled with material from which the water was led to the adjacent Demonstration tunnel. A concrete wall was made in order to separate the drainage material from the test. The backfilling and drainage of the inner part of the tunnel is described in the following section.

5.10 Backfilling and drainage of the inner part of the tunnel

5.10.1 General

Since the water inflow to the inner part of the tunnel was unacceptable, it was excluded from the test area. In order to avoid a high water pressure behind the test section during the installation of the test, it was decided that the inner part should be drained. The inner part was therefore backfilled using crushed TBM-muck and two pumps were installed. The reason for using TBM muck was that it has a sufficiently high hydraulic conductivity.

Two drainage sumps made of concrete well rings, approximately 0.5 m high and with an inner diameter of 1.0 m, were installed. In each ring a pump (BEST 2-aut from ROBOTA) was installed. One of them served as backup.

The concrete rings were covered by steel lids, which in turn were covered by permeable mats. Cuts were made in the rings to make space for the cables and tubes and also for allowing water to enter.

The water and the electric cables were led out through Tecalan tubes with outer/inner diameter 16/10 mm.

5.10.2 Installation

The concrete ring arrangement is shown in Figure 5-10. The rings were placed on a bed of drainage material, which was at least 0.1 m thick. The bed was in contact with the first layer of permeable mats, which were folded below the drainage material.

Since the tubes passed through the connection flange and the lead-through to the Demo tunnel, the pumps with tubes were mounted as a unit together with the flange and the rest of the tubes from the inner section. When the flange was in position, the pumps were placed, one in each drainage sump (see Figure 5-11). Cuts were made in the concrete rings for the Tecalan tubes. The pumps were placed so as not to hinder the float switches.

The steel covers and permeable mats were then placed on top of the concrete sumps.

The backfilling around the cement rings was made carefully for avoiding movements and much of the work had to be done manually. Effective compaction was avoided close to the sumps.

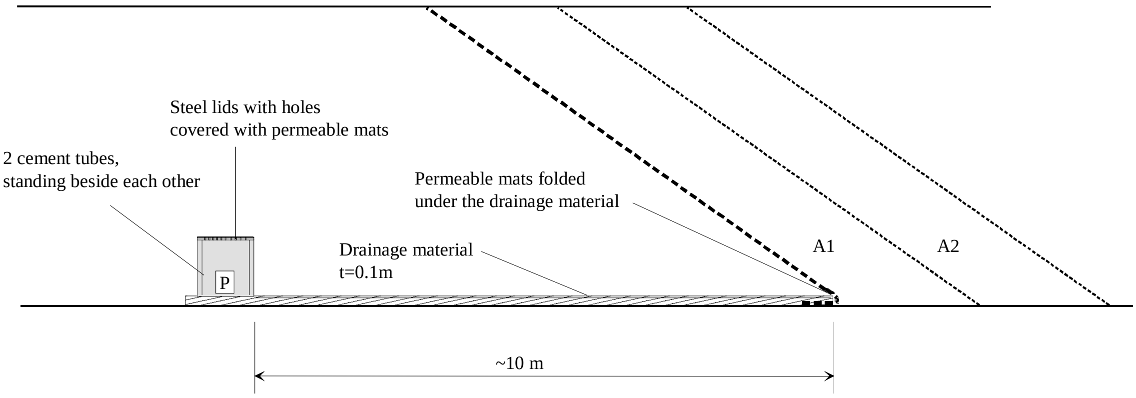


Figure 5-10. *Schematic view of the sumps (not to scale)*

Figure 5-11. *Sump with pump.*

5.11 Wall separating the inner part from the test part

In order to effectively seal the inner part of the tunnel from the test volume, and to create a good support for the first layer of 30/70, the slope of the drainage fill was covered with concrete. It was given the same inclination as the backfill layers (35°).

The rock fill behind the concrete cover was drained during the test installation. To obtain an effective drainage, the concrete was cast on an inclined layer of coarse drainage material on which geotextile had been applied. Tubes led the water from the rock fill to the Demonstration tunnel (see chapter 9). Since the difference in elevation is approximately 4 m and the tube length approximately 40 m, the water had to be pumped from the drainage layer.

Permeable foam rubber was placed between the roof and the backfill to ensure good contact with the drainage layer and to prevent concrete from entering the slot between the backfill and the roof. Great care was taken to obtain tight contact between the roof and the concrete. The first layer of permeable mats for the flow testing was applied on the concrete wall.

In the first two layers of 30/70 in the test section a gap was left at the roof and it was filled with bentonite blocks and pellets for effective sealing of the roof. The described arrangement is shown in Figure 5-12.

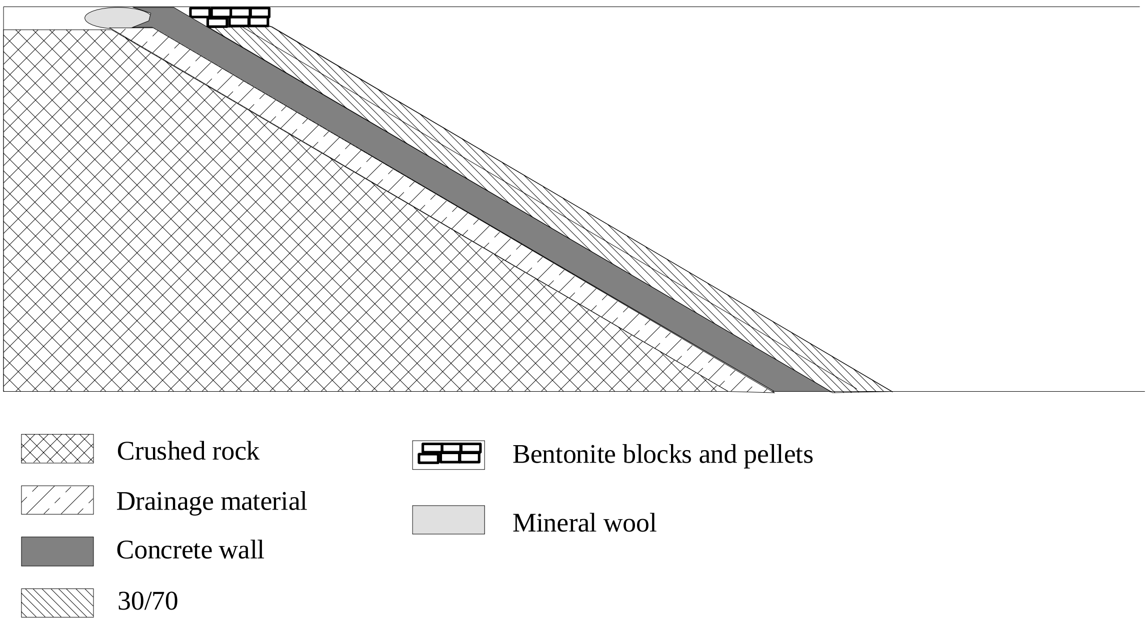


Figure 5-12. *Overview of the arrangements for the concrete wall*

6 Development of backfilling equipment

6.1 General

The technique of backfilling by applying inclined layers was first tested in the project "Field Test of Tunnel Backfilling" /1-11/. A major conclusion from this test was that the technique was suitable but needed improvement. Thus, while the density in the central part of the backfill was high it was significantly lower near the roof and walls. It was necessary to design equipment that could compact the backfill material close to the roof and walls of the tunnel. The design had to ensure improved reliability in operation. The compactor in the field test of tunnel backfilling was powered with an electrical motor and this caused problems with the power cable and difficulties with the transmission to the eccentric element. The improved equipment was hence powered by a hydraulic motor. It was also necessary to find a new carrier that is flexible enough to move the compaction equipment over the entire inclined surface of a backfill layer.

The conclusions led to that two compactors described later in this chapter were developed and designed.

6.2 Carrier

Three different carriers were tested, a Pinguin P 90, a Mecalac 12 CX and a Volvo BM 6300 the latter being selected for the test since it had the best stability and strength (Figure 6-1).

The carrier is small enough to move in the tunnel and to bring the vibratory plate over the entire slope while exerting normal force on it. Also, it can turn the plate so that it rests on the slope irrespective of its inclination. The boom of the carrier is long enough to reach over the entire slope surface and strong enough to carry the compaction equipment with the boom fully extended. The hydraulic system of the carrier is powerful and able to simultaneously move and power the compactors.

The hydraulic system of the carrier was adjusted to fit the requirements of the two compactors. The maximum pressure of the hydraulic system of the carrier was raised to 25 MPa and a constant flow valve was installed. The oil levels in the eccentric elements of the compactors were optimised with respect to required driving pressure.

A balance for weighing the backfill (Tamtron PKV 100) was installed in the carrier.

Figure 6-1. *The Volvo BM 6300*

6.3 Slope compactor

6.3.1 General

The slope compactor was used for compacting the entire surface of the backfill layers except for the part very close to the roof. A photo of the slope compactor is shown in Figure 6-1 and a schematic drawing is shown in figure 6-2.

6.3.2 Desired properties

Experience from previous field tests had set the the following requirements:

- The amplitude should be the same over the entire area of the vibrating bottom plate.
- The total height of the vibrating plate has to be low in order to make it possible to compact close to the walls and roof.
- The total weight of the vibrating plate has to be so low that a relatively small and flexible carrier can handle it.
- A hydraulic motor should be used for powering the vibrating plate.
- The reliability of the vibrating plate has to be high in spite of the mechanical strain caused by vibrations and rough handling.
- The direction of the vibrating movement and the centrifugal force has to be perpendicular to the surface of the backfill

Typical data are:

- Total weight: 900 kg
- Area of compaction plate: 7700c cm^2
- Vibrating weight: 414 kg
- Vibrating frequency: about 43 Hz (2700 rpm)
- Amplitude: 2.7 mm (peak to peak 5,4 mm)
- The driving and transmission systems are designed for 15 kW power.

6.3.3 Design

A schematic drawing of the slope compactor is shown in Figure 6-2. The numbers in the figure refer to the text below. The upper frame is red in the figure, the vibrating part is green, the safety arrangements (catching hooks) are blue and the eccentric element, which is hidden the picture, is represented by a dotted line.

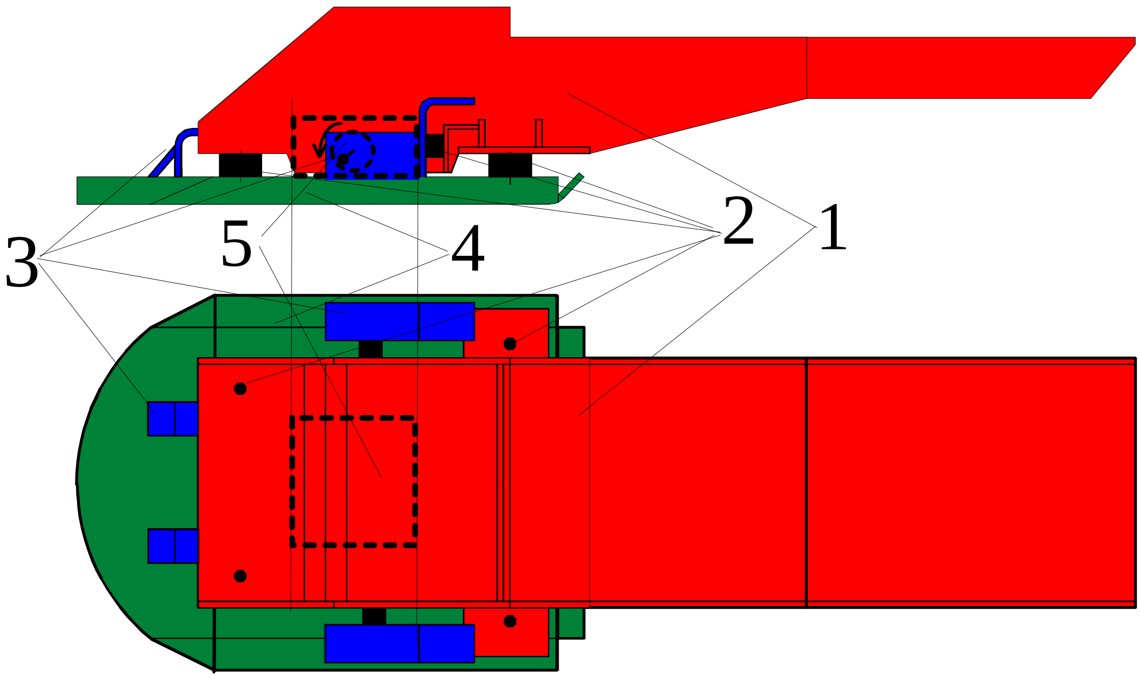


Figure 6-2. *Schematic drawing of the slope compactor*

<u>Eccentric element (# 5 in Figure 6-2)</u>

The eccentric element was welded to the bottom plate and placed inside the upper frame. In Figure 6-2 its location is indicated by dotted lines. A Dynapac LG 700 eccentric element specially prepared for perpendicular eccentric force and with 30% higher eccentric force, was chosen. The special design with a hydraulic motor operating the eccentric element required special adoption to the shaft and rubber isolations.

Bottom plate (# 4 in Figure 6-2)

The bottom plate was designed so as to reach out to the walls and roof while being stiff enough. The centre of gravity in the horizontal plane and the resultant of the eccentric forces coincide. The front of the vibrating plate was given a round shape to facilitate close approach to the roof and walls of the tunnel.

Upper frame (# 1 in Figure 6-2)

The function of the upper frame is to serve as a connection to the carrier so that the vibrating plate can be moved over the inclined surface. Static pressure can also be applied to the vibrating plate through the attachment and the frame. The attachment was placed behind the vibrating plate on an extension of the upper frame (see Figure 6-2) so that the attachment and the rotor tilt unit would not prevent the vibrating plate from coming close to the rock.

Figure 6-3. *Photo of the slope compactor*

Rubber isolators (# 2 in Figure 6-2)

The rubber isolators separate the upper frame and the vibrating parts and transfer manoeuvre forces to the vibrating bottom plate. To obtain good isolation the critical frequency for the support of the upper frame was designed to be lower than the working frequency of he vibrating plate.

The hydraulic system

The hydraulic motor was designed for an oil flow of 45 l/min at a rotation of 2600 rpm. It has an output of 15 kW, which is the required power. The hydraulic system of the carrier supplies the hydraulic motor of the vibrating plate with an oilflow of 45 l/min.

6.3.4 Test of function

A performance test of the slope compactor was made at the manufacturing company. The compactor worked satisfactorily, but the isolation of the upper frame was found to be too stiff. The slope compactor was tested with three isolators instead of four, this improved the isolation of the upper frame considerably. The change in total spring constant was required to achieve good isolation of the upper frame from the vibrating plate.

The slope compactor was then tested on a gravel pile using a caterpillar as carrier and power source. The hydraulic system of the carrier was not stable enough to keep the hydraulic flow sufficiently high for powering the vibrating plate at a constant frequency. The power output from the carrier varied when other hydraulic functions were used. The test showed that the vibrating plate worked well together with a carrier but also demonstrated how difficult it is to control the oil flow from the hydraulic system of a carrier.

6.4 Roof compactor

6.4.1 General

The backfill adjacent to the roof could not be compacted satisfactorily with the slope compactor. In order to solve this, another vibrating compaction machine, the roof compactor, was designed and manufactured. The development and testing of the roof compactor was made during 1997 and 1998. A principal drawing is shown in Figure 6-4 and a photo of it in Figure 6-5.

6.4.2 Desired properties

The dynamic pressure (defined as the pressure peak exerted on the soil surface created by the roof compactor) was calculated to be about two times the dynamic pressure of the slope compactor. The high dynamic pressure is created by a high momentum and relatively small size of the vibrating plate. The orientation of the compactor varied during the compaction of the layers. The roof compactor was designed to move small amounts of backfill material up to the tunnel roof.

<u>Typical data are:</u>

Vibrating mass: 240 kg.

Total weight of the roof compactor: 400 kg.

Vibratory amplitude: 3 mm (total movement 6 mm)

Vibratory frequency: 43 Hz

The vibratory movement of the roof compactor is one-dimensional. The eccentric forces are symmetrical and the point of gravity of the vibrating mass produces a centrifugal force of 56 000 N.

6.4.3 Design

In the schematic drawing in Figure 6-4 the vibrating part is green-coloured while the isolated support is marked red.

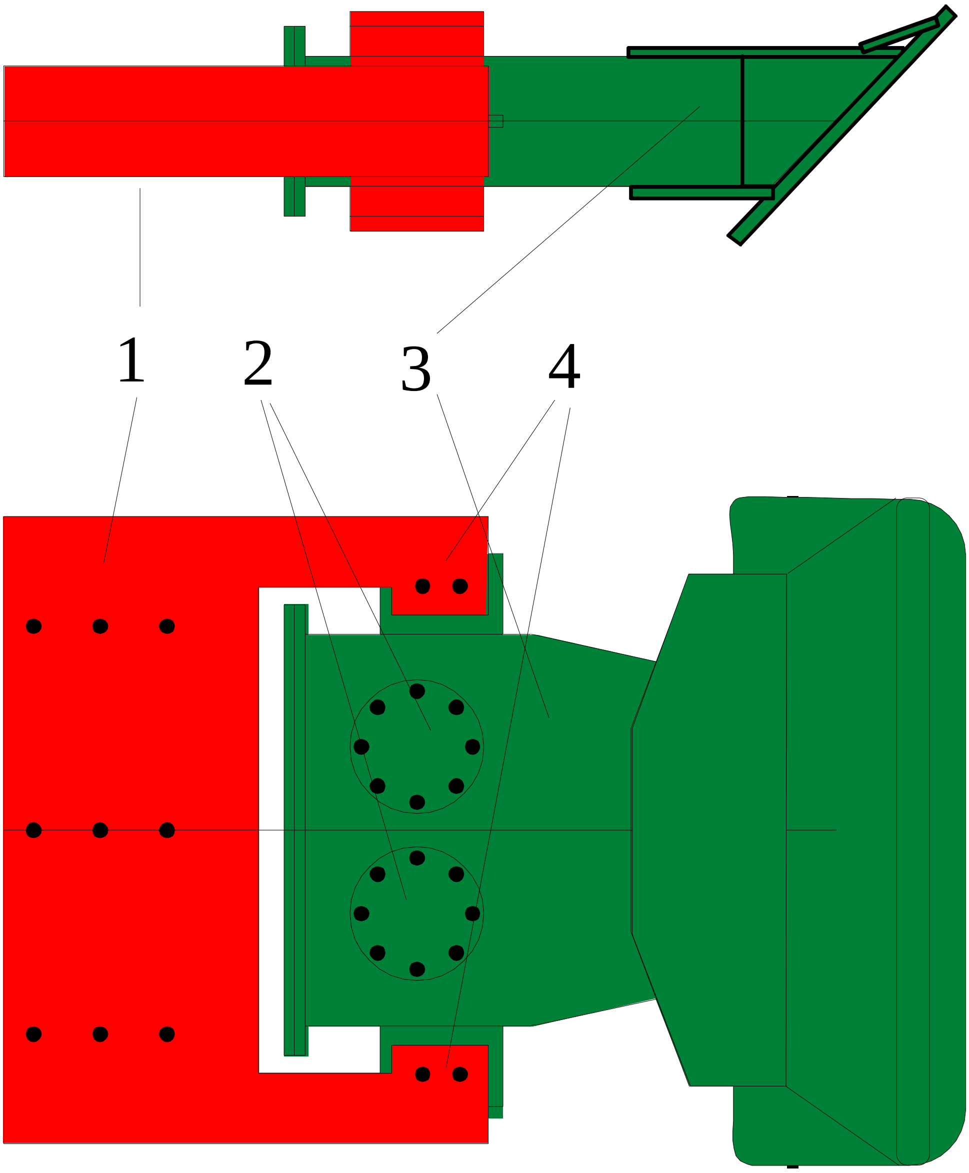


Figure 6-4. *Schematic drawing of the roof compactor (The arm connecting it to the carrier is not illustrated)*

Vibrating part (# 3 in Figure 6-4)

For making it possible to come close to the roof the eccenters (# 2 in Figure 6-4) were placed 500 mm behind the pressure plate of the compactor. The part of the compactor located between the pressure plate and the excenters was designed as a pillar shaped so that it was suitable for the stress conditions in the vibrating part.

Since it was required that the vibration motion should be one-directional it was necessary to have two eccentric elements rotating in opposite directions. The gears were placed in the middle of the shaft and the excenters were placed on both sides of the gears to achieve gravitational symmetry. The shafts were rested in spherical cylinder bearings. The bearings and gears had to be lubricated and the rear part of the vibrating plate was therefore designed as a gearbox house, containing 2,7 litres of oil. The excenters produce an oil mist, which lubricates the bearing and the gears. It also helps transporting heat from the bearings and the gears to the gear house shell, which is air-cooled.

Isolated support (# 1 in Figure 6-4)

The vibrating part was mounted on rubber isolators in a U-shaped support frame. The four rubber isolators mounted at the ends of the U-frame were placed on the symmetry axis on which the center of gravity was located. The other isolators were placed at the rear end of the U frame for stabilising the vibrating part.

Rubber buffers were placed in the end of the isolated part to transform high static forces from the carrier. The purpose of these buffers was to prevent the vibrating part from hitting the support metal to metal.

Driving systems

A hydraulic motor of the same type that is used in the slope compactor is the driving source for the roof compactor. The motor is powered by the hydraulic system of the carrier. A constant flow control valve gives the hydraulic motor a constant oil flow of 45 litres per minute corresponding to a rotation speed of 43 Hz independent of other hydraulic functions of the carrier.

Flexibility

The roof compactor was designed for easy modification and maintenance:

- The push plate was attached through a screw joint, thus making it exchangeable.
- The eccenters were bolted to the shafts.
- There was extra space making it possible to put in bigger rubber isolators.
- The isolated frame was bolted to the adapter arm, thus making it exchangeable.

Figure 6-5. *The roof compactor*

Figure 6-6. *The pusher*

6.5 Pusher

The tool for pushing backfill material in place was developed as part of the project "Field Test of Tunnel Backfilling" /1-11/. Based on experience from this and later tests a new prolonged version was manufactured (see Figure 6-6).

6.6 Pellet blowing machine

The pellet-blowing machine was designed and manufactured to fill the void close to the roof that is left when backfilling 0/100 and bentonite blocks. A picture of the pellet-blowing machine is shown in Figure 6-7. The principle of the machine is that an air source, "a blowing machine", adds air through a pipe system. A cell feeder, or feeding-out valve, portions the pellets into the air-stream that transports the pellets to the desired position.

The capacity is 5000 kg/h with a possible transport length of 25 m horizontally and 5 m vertically, including two bends of the tube.

Figure 6-7. *The pellet blowing machine.*

6.7 Safety assessment

Safety assessment of the slope and roof compactors was made before the work started.

There are two major risks concerning personal safety:

- Pieces may come loose from the compactors and fall down the slope during compaction.
- Workers could get run over by the carrier, since it is difficult for the operator to watch the area around the carrier.

Actions taken to decrease risks:

- No one will be allowed in the carrier's working area. Warning signs are placed outside the tunnel during the backfilling.
- After the compaction of each layer the rubber isolators and their attachments are checked. After each day the torque of the screw joint reinforcements is checked.

6.8 Service and reparations during backfilling

Before the backfilling began, a risk analysis considering failure of the different parts of the slope and the roof compactor was made. Based on this analysis, necessary spare parts were required. A contract was signed with a company in Oskarshamn for quick repairs. The slope compactor needed to be repaired two times during the course of the backfilling.

The rubber isolators of the slope compactor were worn out from time to time. They were exchanged by the operator of the carrier.

The following repairs and adjustments were made.

Roof compactor:

- The rubber units were insufficient for transferring the static force from the carrier and isolate the U-frame from the vibrating part. This resulted in certain minor deformations of the U-fame and the vibrating part. The units were replaced by bigger ones.
- Cracks occurred in catching hooks probably due to contact with the roof. The hooks were welded and the shape changed to minimise rock contact.
- The mounting plate of the adapter arm was deformed at the edges due to deformation or creep during manufacturing. The edges were straightened, and the plate was reinforced.
- Cracks developed on the inside corners of the isolated U-frame due to contact with the vibrating part at high static load. The corners were given a radius of 10 mm and the remaining cracks were welded.

Slope compactor:

- The hydraulic hoses were worn and had to be replaced such that they would last longer.
- The transmission coupling had been broken and was replaced

6.9 Evaluation

6.9.1 General

After the compaction was completed, the roof and slope compactors were transported to Oskarshamn and examined. The conclusions from the investigations are given in the subsequent chapters.

6.9.2 Slope compactor

<u>Control</u>

1. All of the catching hooks were deformed and some of them had to be repaired. The catching hooks need to be re-designed for enabling them to take higher loads from the carrier.
2. The rubber isolators broke quite frequently. It is important that the isolators are exchanged as soon as they brake to prevent damage of the rest. The design of the compactor should be changed for saving the isolators and reducing the vibrations in the carrier, see above.
3. The hydraulic tubes were worn by being in contact with the compactor but it is not yet known what caused the wear. The positioning of the tubes should be changed so that they are not worn when static load is applied. The worn tubes should be exchanged.
4. No oil leakage could be detected. On replacing the oil it was found to be black, which shows that the temperature had been high. No metal fragments could be found in the oil. The excenter should be more effectively cooled but this will be quite complicated. An alternative procedure would be to check and replace the oil more frequently. Should the oil contain metal fragments, the excenter must be opened and the gear wheel and the bearings controlled and, if necessary, be exchanged.
5. The flexible connection of the power transfer between the hydraulic motor and the excenter was worn. A protection of the coupling should be designed and attached. The coupling with socket head cap screws should be exchanged. It would also be convenient to exchange the shaft packing.
6. Worn surfaces had corroded. The entire compactor should be re-painted with a brighter colour for better visibility.
7. The weldings of the attachment appeared to weak and should be improved.

General experiences

1. The effective operating time for application and compaction of the backfill was about 75 hours

2. The compaction effect was as expected.

3. The accessibility to roof and walls was very good: The momentum at the carrier decreased the mobility of the carrier expansion arm.

4. Accessibility at the floor was good.

5. The compactor should be re-painted for increasing visibility. The working area should be equipped with strong lamps.

6. It was deemed necessary to reduce the vibrations transferred from the compactor to the carrier since there was a risk for damaging the carrier.

7. The temperature of the carrier's hydraulic system rose to a high level after compaction. An extra cooler should be installed in the carrier.

8. It would be an improvement to insert the hydraulic couplings into a clean container while not in use to prevent them from becoming soiled.

9. The repair contractor asked for a better-balanced lifting point.

6.9.3 Roof compactor:

Control

1. The compaction plate was heavily worn and the welds were almost gone. They should be removed and be redone.

2. The isolated U-frame and the vibrating part had hit each other, resulting in quite heavy deformations. A proposal of how to handle high static loads will be made.

3. All the rubber isolators were damaged. A new design proposal on how to better handle the static forces should be made.

4. The conclusions under paragraphs 3, 4, 6 and 7 for the slope compactor are valid also for the roof compactor.

General experiences concerning the roof compactor

1. Net operating time was estimated at 15 hours.

2. The conclusions under paragraphs 3, 5, 6, 7, and 8 for the slope compactor are valid also for the roof compactor.

3. The life length of the rubber isolators was acceptable but they should absorb more of the vibrations transferred to the carrier.

4. The shape of the compaction plate, the attachment of the adapter arm and the design of the catching hooks are satisfactory.

5. It is difficult to connect the roof compactor to the carrier since the angle of attachment is unfavourable. The operator of the carrier cannot see the attachment when the compactor is lying down. A suitable stand would shorten the time for attaching it.

6.9.4 Safety

Some safety aspects:

1. The catching hooks cannot be allowed to be deformed.

2. It is important that nobody is present inside the working area during compaction.

7 Backfilling description

7.1 Overview of the backfilling sequence

Starting from the concrete wall six sections of 30/70 bentonite/crushed rock, each consisting of 5 – 7 compacted layers, were backfilled (see Figures. 7-1 and 7-2). The layers were given an inclination of approximately 35 degrees. The sections consisting of 6 layers had an axial extension of 2.2 m in the tunnel and were separated by permeable mats.

The 30/70 backfill part is followed by four sections of crushed rock (0/100). A gap of 20 to 30 cm was left at the roof and filled with blocks of highly compacted bentonite. The space that remained between the roof and the bentonite blocks was filled with bentonite pellets by use of the blowing machine.

At the base of the fourth 0/100 section a retaining wall, separating the backfill from the plug, was placed. The wall consists of concrete beams, which were put in place as the backfilling progressed, i.e. when a beam was in place the backfilling continued as much as possible until the next beam was placed. This was repeated as long as it was possible to bring material in for compaction The remaining space was partly filled with compacted blocks containing 20 % bentonite and 80 % sand and partly with bentonite pellets. A permeable mat was placed between the retaining wall and the backfill. A schematic drawing of the subdivision of section B5 is shown in Figure 7-3.

The backfill and the rock were instrumented as described in chapter 11. From each section the cables from the instruments and the permeable mats were led through steel tubes in the bore-holes to the Demonstration Tunnel (chapter 9).

A few shallow blast holes remaining from the excavation of the tunnel were filled with a mixture of MX-80 powder, pellets and water as the backfilling progressed.

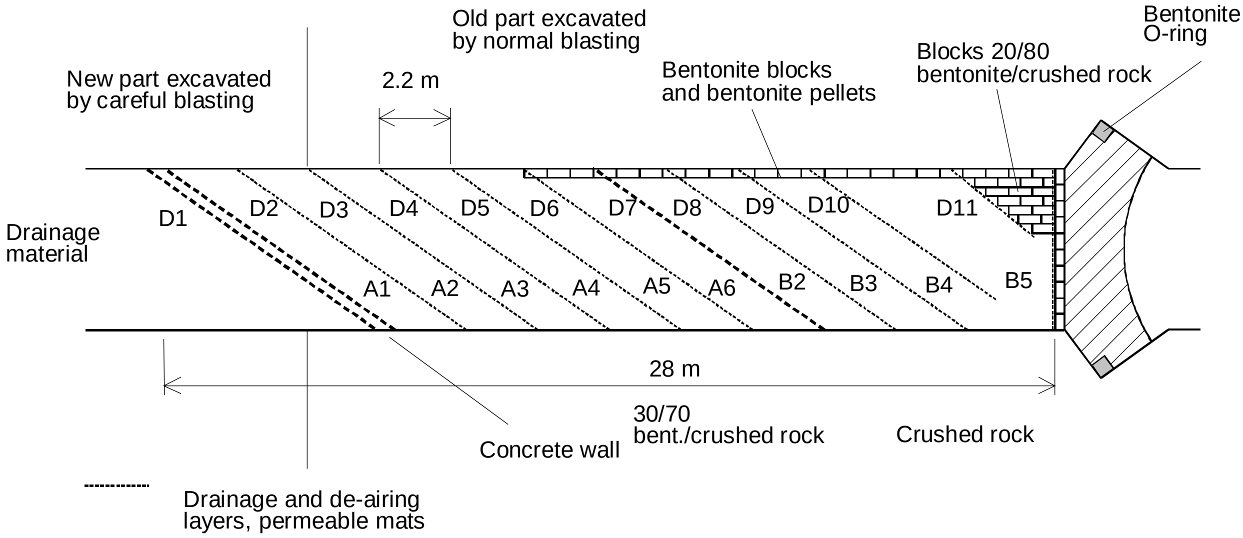


Figure 7-1. *Designation of the test sections and the permeable mat sections.*

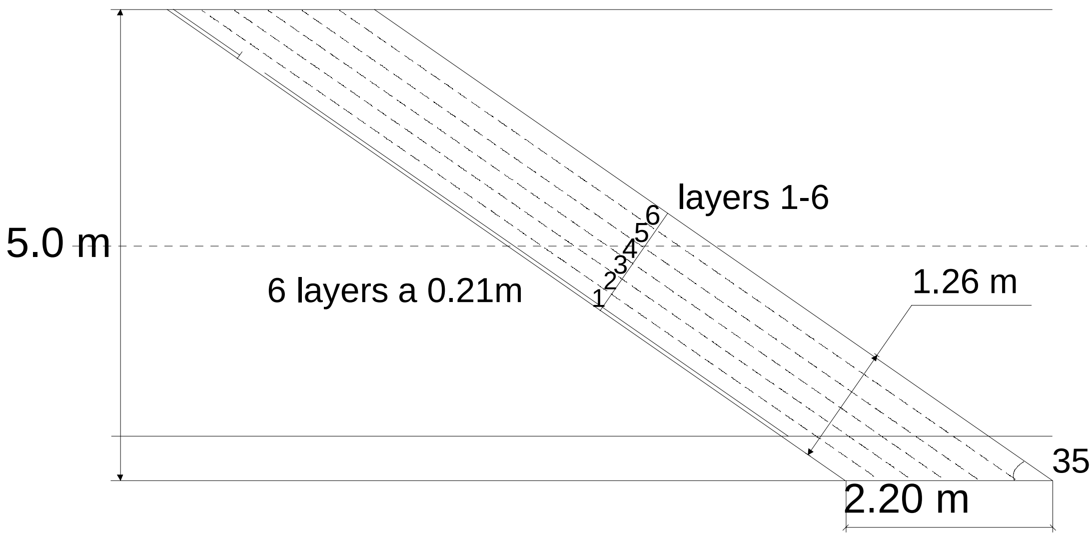


Figure 7-2. *Subdivision of a backfill section into backfill layers*

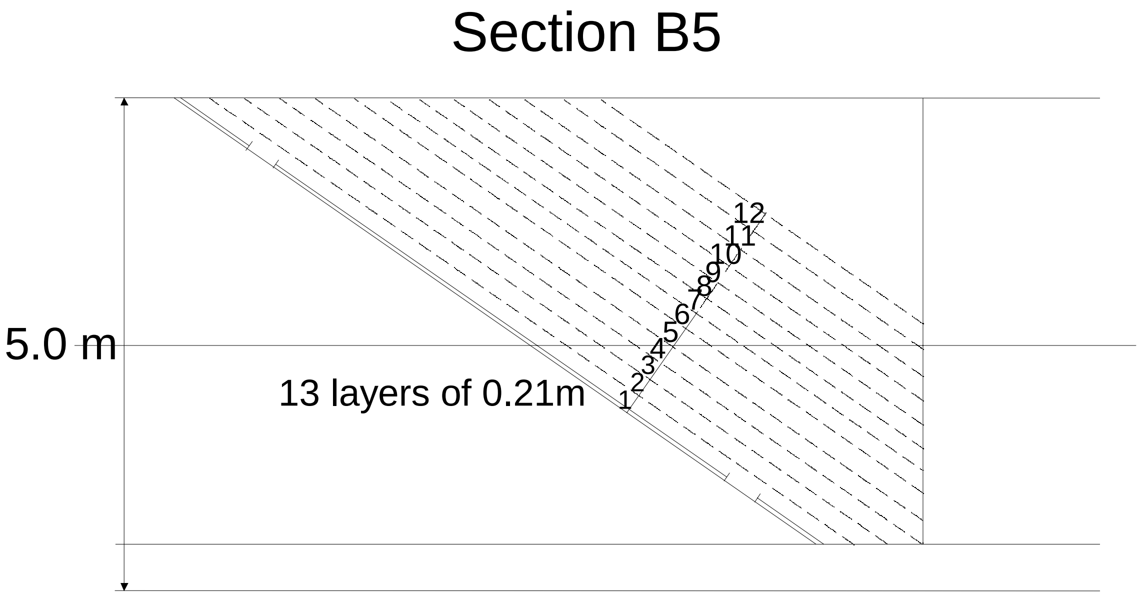


Figure 7-3. *Subdivision of backfill section B5. Tthe space above layer 12 was filled with 20/80 blocks and bentonite pellets.*

7.2 Overview of the –420 level

The test is located in the so-called ZEDEX-tunnel at the –420 m level. An overview of the test site at the –420 m level is shown in Figure 7-4.

The area codes below refer to Figure 7-4.

<u>Area A (test tunnel):</u>
The Backfill and Plug Test tunnel

<u>Area B (just outside the test tunnel):</u>
A container for storage of backfill material was placed outside the test tunnel in January 1999 and removed in June 1999. The area to the north of the container was used for intermediate storage of the backfilling tools. The material container was re-filled with backfill material brought down from tents on ground close to the adit.

Area C (north part of the Demonstration tunnel)

The through-connections of the cable packages enter the Demonstration tunnel here. A 0.9 m wide scaffold with a walking level 1.5 m above the floor was used as working platform for the cable handling. A cable ladder for the cables and tubes leading from the inner hole to the tunnel opening and then along the roof to the measuring container has been installed.

Area D (small shaft opposite to the Demonstration tunnel)

The measuring house is placed in this small niche.

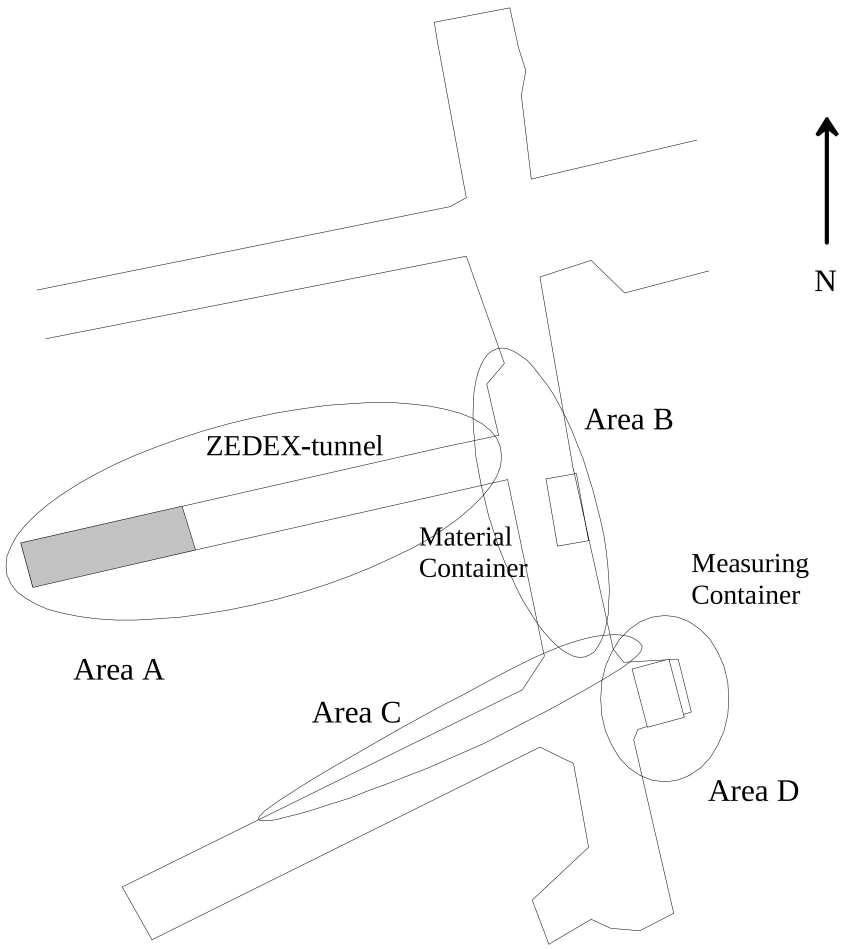


Figure 7-4. *Overview of the –420 level, showing the test tunnel and different areas used in the project.*

7.3 Installation and Instrumentation of test sections

Each test section except the one closest to the plug (B5) consists of 5–7 layers of compacted backfill material. The thickness of each compacted layer was intended to be 37 cm in the axial direction of the tunnel. This corresponds to a thickness perpendicular to the surface of 21 cm. In practice, the thickness varied somewhat within and among the layers. An example of the cable arrangement within one layer is shown in Figure 7-5. Before a section was backfilled, the lead-through for cables and instruments was installed. The bundle with cables and tubes was collected at the lead-through exit in the Backfill and Plug Test tunnel (see Figure 7-6 and chapter 9).

Figure 7-5. *Cable arrangement in a 30/70 layer.*

In order to fasten and protect the cables and minimise leakage along them in the layers, they were separated and fixed with nails and plastic staples in a 5 cm deep ditch (Figure 7-7). Extra sealing was made by applying bentonite powder between and around the tubes and cables. Finally, backfill material was used for covering the bentonite and cables and levelling the surface. The cable ditches were placed in special cable corridors to keep the density measurement areas free from cables and tubes (see Figure 7-8). The instruments were put in recesses made in the backfill and fines from the backfill material were placed around the sensors to minimise strain during compaction

Figure 7-6. *Exit of an installed cable package in the test tunnel.*

Figure 7-7. *Installation of cables on the surface of a backfill layer*

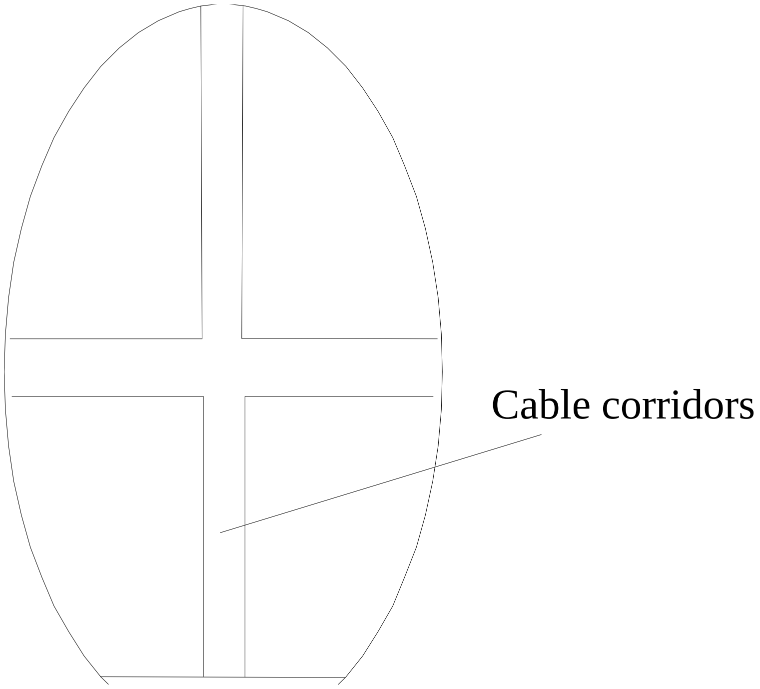


Figure 7-8. Areas where cables and tubes were led at the surface of the compacted layer

7.4 Description of placement and compaction of one backfill layer

The backfilling sequence of each layer was different for the two different backfill types.

Backfilling of 30/70
A schematic drawing showing the procedure is shown in Figures 7-9 and 7-10.

The following steps were taken during the backfilling sequence:

1. The temporary road (coarse gravel) was removed from the floor and the floor cleaned

2. The backfill material for the layer was transported into the tunnel. The mass required was about12 tons. The backfill material was dumped on the previous layer.

3. The material was brought into position with the pusher (see Figure 7-11). This operation gave the desired shape of the layers, which were made slightly concave to simplify compaction close to the walls.

4. At the left rock wall, around the cable lead-through the backfill material was replaced with granulated bentonite and bentonite pellets (see Figure 7-6).

5. Backfill material was pushed towards the roof and compacted with the roof compactor in order to achieve a high density at the top of the backfill and to ensure good contact with the roof (see Figure 7-12). In some cases, when the density at the roof was not sufficiently high, the procedure of pushing more material towards the roof with the pusher and compacting it with the roof compactor was repeated.

6. The rest of the layer was compacted with the slope compactor. Starting about 1.5 m above the floor it was moved horizontally across the layer from one wall to the other (see Figure 7-10). The rotor-tilt unit was continuously used for adjusting the angle between the layer and the compactor to prevent cutting into the material. After one horizontal sweep the compactor was moved up for the next horizontal sweep. The overlap varied between 20 and 50%. This was repeated to the top of the layer so that the areas treated with the slope compactor and the roof compactor overlapped. The compactor was then moved in the same way, side to side, down the layer to the starting position. This procedure was repeated three times. The compactor was subsequently moved from top to bottom along the rock wall to further improve the compaction close to the walls. As a last step, the compactor was turned 180° for compacting the area at the floor (see Figure 7-13). The result of the compaction varied with the local thickness of the layers, geometry of the tunnel, backfill and rock instrumentation. The operator of the carrier compensated this with extra rounds of the compactors. Hence the compaction time for different areas in one layer and for different layers varied somewhat.

A slightly concave shape of the inclined layers was optimal since it allowed the carrier to move the vibrating plate in a simple way and since the compaction work could be effective near the walls of the tunnel. The density of the backfill in this region could therefore be raised and the impact of topographic irregularities of the rock walls reduced. Furthermore, the concave shape also resulted in a steeper inclination of the layer near the roof, which made it easier for the vibrating plate and roof compactor to reach and compact the material here.

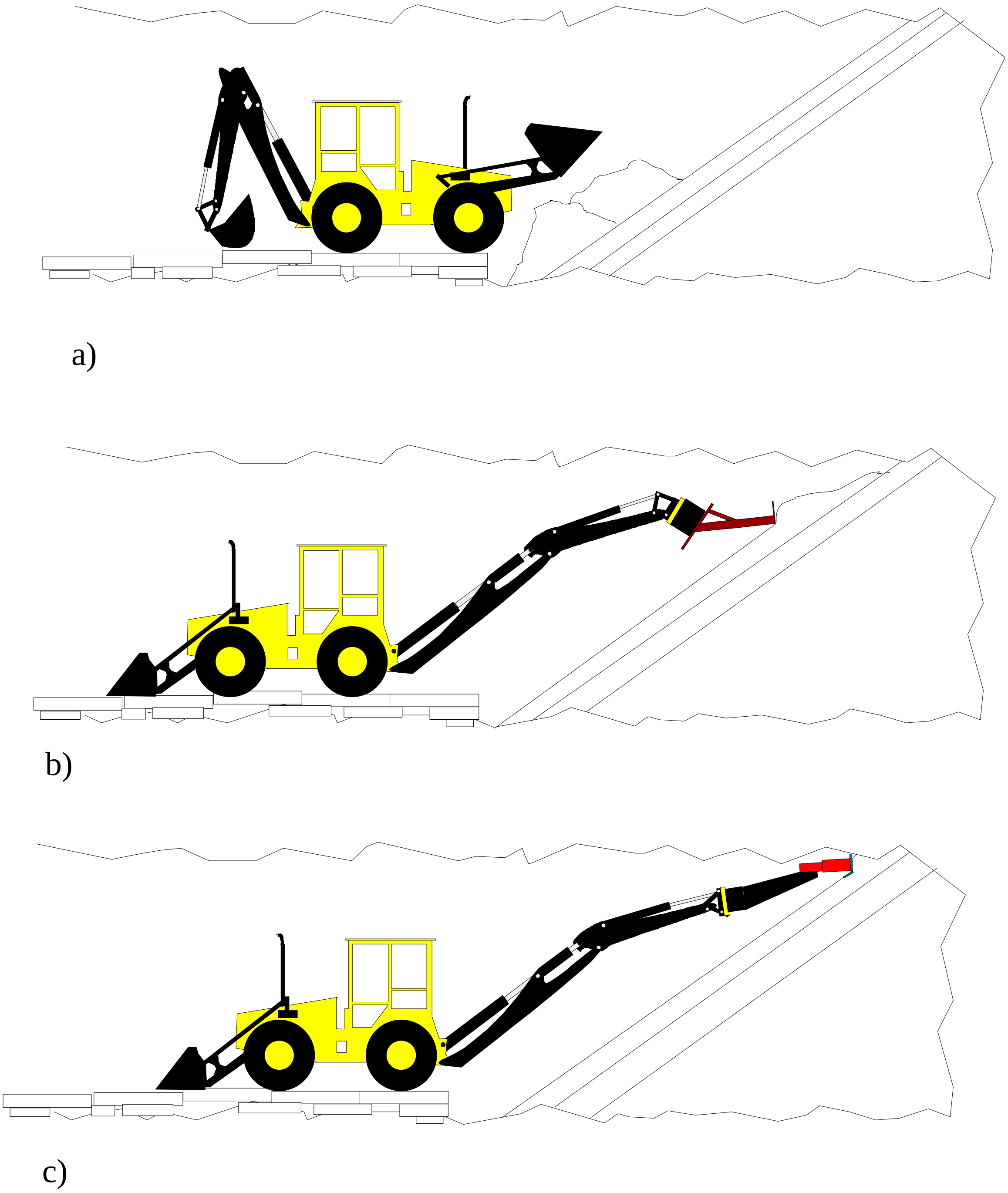


Figure 7-9. *Backfill cycle for one layer 30/70*

a) Placement of the backfill

b) Pushing the backfill in position

c) Compaction at the roof

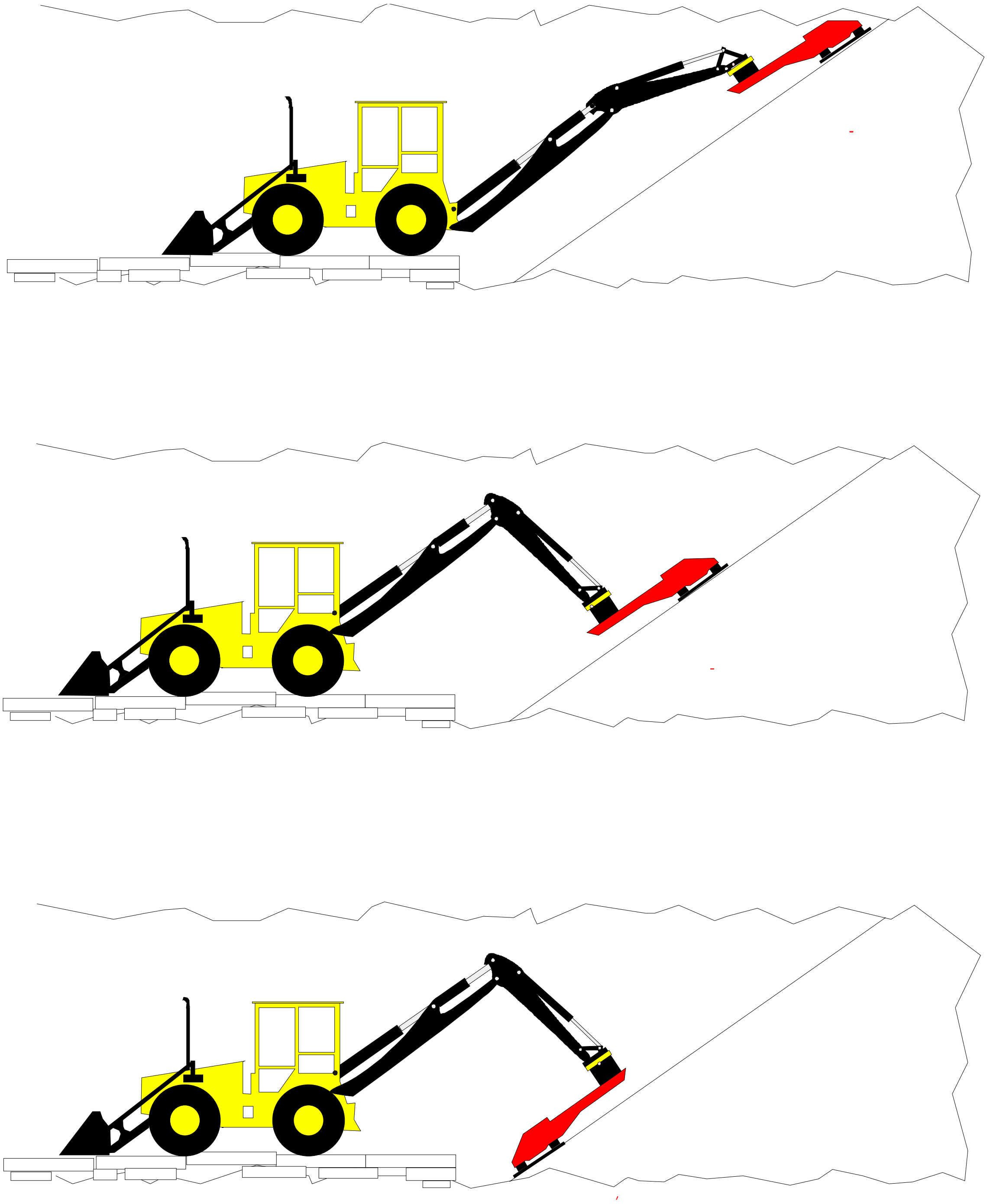


Figure 7-10. *Backfill cycle for one layer 30/70. Compaction with the slope compactor*

Figure 7-11. *Pushing the material in place.*

Figure 7-12. *The roof compactor in action.*

Figure 7-13. *The slope compactor*

Backfilling of 0/100

1. Step 1- 4 was identical with the procedure for backfilling 30/70, except that the weight of each layer was 17 tons.

5. The layer was compacted with the slope compactor the same way as for the 30/70 in step 6.

6. The gap that remained at the roof was made wider so that two bentonite block layers fitted the gap.

7. Two layers of bentonite blocks (250 x 117 x 65 mm^3) corresponding to 100-150 pieces were placed in the gap (Figure 7-15).

8. Bentonite pellets were placed in the remaining gap between the roof and the bentonite blocks (Figure 7-14). This was made every second layer. The emplacement was made with the pellet-blowing machine (see chapter 6.6). The emplacement of pellets is shown in Figure 7-14. 100-160 litres of pellets were used for two backfill layers.

Figure 7-14. *Emplacement of pellets*

Figure 7-15. *Blocks and pellets placed in the slot at the roof.*

7.5 Material handling

The backfill material was stored in tents close to the adit of the Äspö HRL. It was transported to the test site with trucks and stored in a container built for this purpose (see Figure 7-4). In order to prevent the water ratio of the material from changing when it was stored for longer periods of time, the container was covered with tarpaulin.

When moving backfill material into the tunnel the operator of the carrier filled the bucket with backfill material, recorded the weight and stored the data in a computer mounted on the carrier. The weighing equipment gave the weight of each layer as a receipt. The weight of the layers is given in chapter 8.

7.6 Permeable mats

Permeable mat layers separate the test sections. For each permeable mat section one mat was placed at the roof, one at the floor and one covered the central area of the test tunnel (Figures 7-16 and 7-17). To insure good hydraulic contact between the rock and the mats at the floor and roof they were folded along the rock surface so that about 20 cm was in contact with the rock.

In the first three permeable mat section each mat consisted of three layers of geotextile: two layers of Fibertex F-4M and one layer of 2/F/500. This three-sheet permeable mat had a damping effect on the compaction energy and hence decreased the density of the subsequenty applied layer of backfill material. In order to reduce this it was decided that the rest of the permeable layers should consist of two Fibertex F-4M mats. They were nailed to the under-lying material with three-inch nails. The mats were connected to Tecalan tubes that were used for saturating and de-airing the mats in the initial stages of the test and for measuring water pressure and water flow during the flow tests. The positions of the water inlet and de-airing points of the tubes are shown in Figure7-18.

Figure 7-16. *Permeable mats*

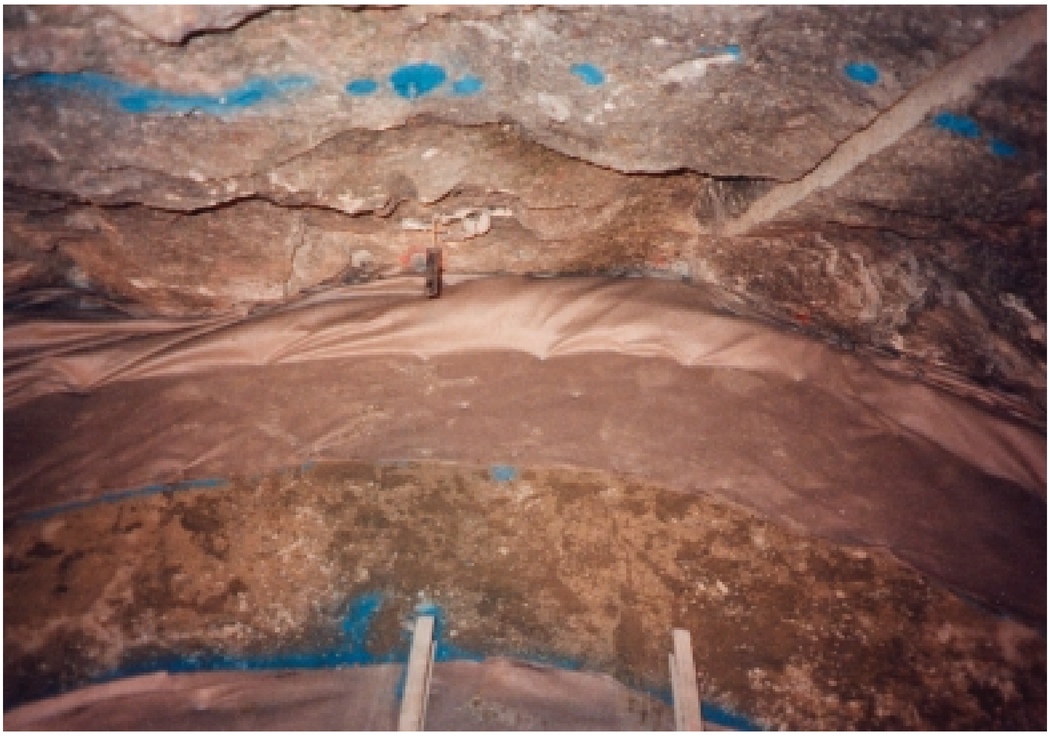


Figure 7-17. *Permeable mat at the roof*

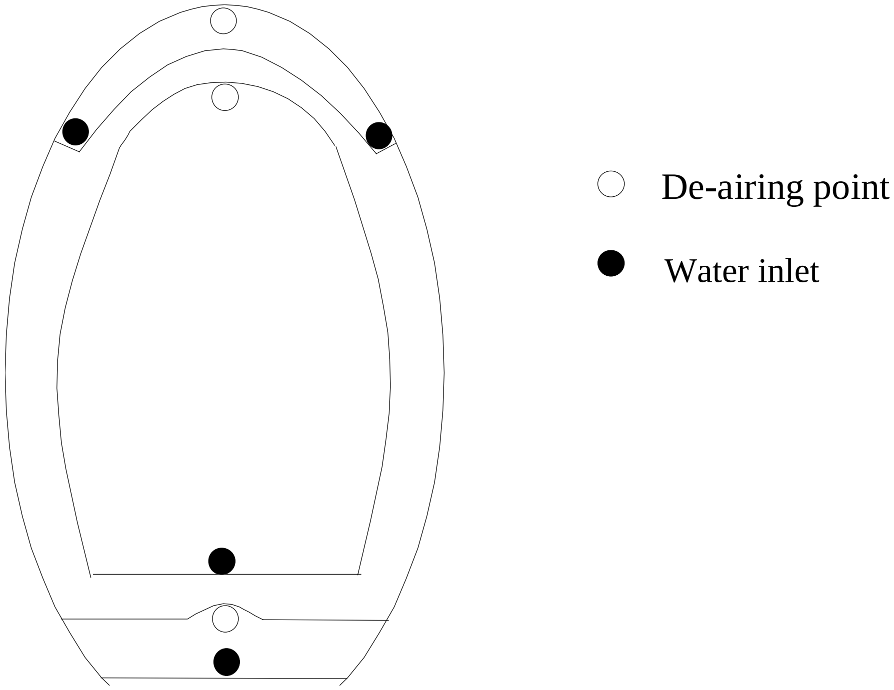


Figure 7-18. Schematic drawing of the permeable mats. The view is perpendicular to the surface of the layer.

The tubes were fixed to the underlying material with nails and hooks. The ends of the tubes were secured with several nails to ensure that it stayed in position in the permeable mat during the backfilling of successively applied and compacted layers. The tube ends were placed in small filter mat units and were placed between the permeable mats. The application of the backfill layers in contact with the permeable mats was made carefully in order not to displace the mats and the connections.

7.7 Backfilling against the retaining wall

Prefabricated concrete beams were applied to form a retaining wall that separated the backfill from the plug. Before the start of the backfilling, part of the plug was cast to provide support for the concrete beams and act as a foundation of the “bentonite O-ring” (see chapter 12). When the backfilling had proceeded to the wall, corresponding to layer 2 section B5, the first of the 6 concrete beams was put in place. The permeable mat on the inside of the wall was applied in segments, beam by beam. After putting in a beam, backfill material was applied and compacted in 35° layers starting from the beam (Figure 7-19). This was repeated up to layer 12 in section B5 (see Figure 7-3). The remaining space was filled with 20/80 blocks and pellets, see Figure 7-20. The space remaining between the rock, roof and concrete beam No 6 was cast as the final step of constructing the concrete ring.

Figure 7-19. *Backfilling against the concrete beams*

Figure 7-20. *Filling the remaining space at the roof with 20/80 blocks and bentonite pellets.*

8 Measurements during backfilling

8.1 General

The density and water ratio of each layer were determined during the backfilling. Two types of measuring principles were used for determining the density. Equipment, technique and results from the measurements are presented in this chapter.

8.2 Equipment and Technique

8.2.1 Nuclear gauges

Two types of Campel Pacific nuclear gauge have been used for measuring the density. The performance of the Campel Pacific MC-3 Portaprobe (referred to as density meter A) and the MC-s-24 Direct Readout Strata Density / Moisture Gauge (referred to as density meter B) is based on the use of radiophysics. A gamma source produces radiation that passes through the soil. The density is calculated on the basis of the amount of radiation that is absorbed by the soil. Schematic drawings of the gauges are shown in Figures 8-1 and 8-2.

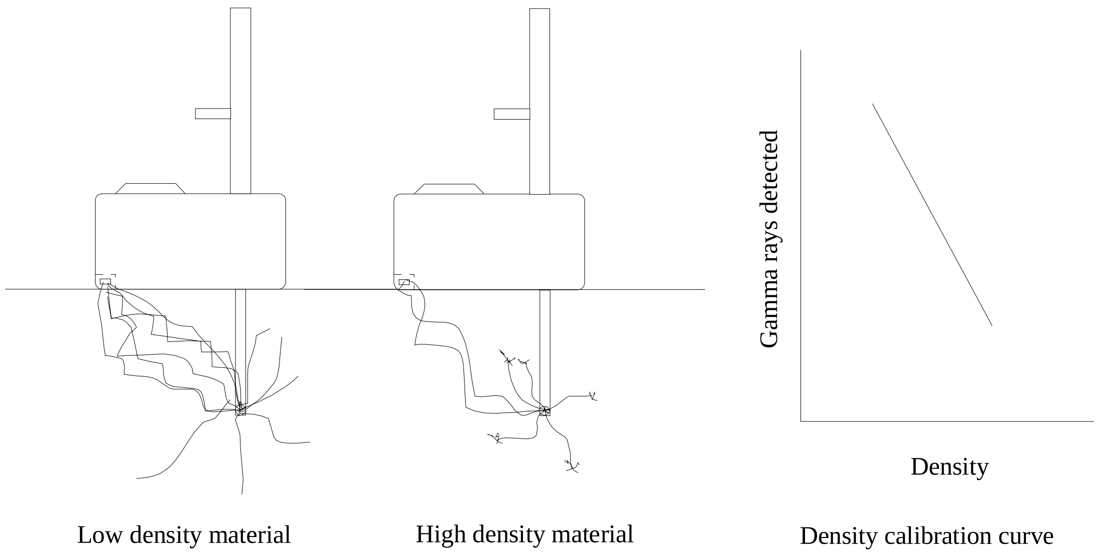


Figure 8-1. Pacific MC-3 Portaprobe (density meter A)

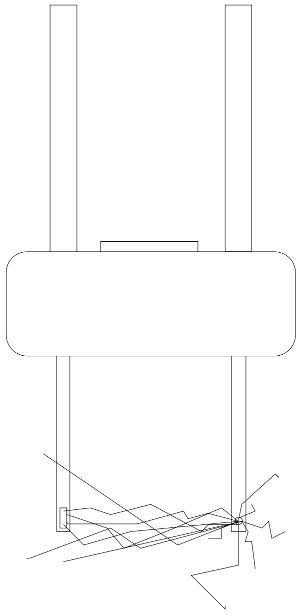


Figure 8-2. *MC-s-24 Direct Readout Strata Density / Moisture Gauge (density meter B).*

A slide hammer and a plate were used for making a hole perpendicular to the compacted surface and the rod with the gamma source was pushed down into it. The recording time was 1 minute for both gauges. Density meter A measures an average density between the source and the soil surface. Density meter B measures the density at the depth where the source is placed. The gauges are described in detail in /1-11/.

8.2.2 Penetrometer

A penetrometer was used where it was not possible to use the nuclear gauges, i.e. close to the roof and walls of the tunnel.

The principle of the penetrometer is to measure the resistance when a steel rod is pushed into the material. The average resistance for different densities of a certain material with a certain water ratio can be calibrated. This is a rough method and an average of at least 5 measurements should be used for estimating the density.

8.2.3 Water ratio

The water ratio was determined in four evenly spaced parts of each layer. Samples with an approximate weight of 1 kg were taken and transported to the geotechnical laboratory at Äspö. The samples were weighed, dried at 105 °C for 24 hours and then weighed again for determining the loss of water. The water ratio was calculated as the weight of water divided by the dry weight of the sample.

8.3 Scope of measurements

The density was measured and the water ratio determined after compaction of each layer. The measuring program of each layer is described in this chapter.

8.3.1 Density measurements

The density after compaction was measured at the following points (Figure 8-3).

<u>With density meter A</u>: 8 points outside the cable corridors.

<u>With density meter B</u>: Measurements were made only when time was available. The measurements were not made according to a pre-set schedule. One set of measurements was made in layer 6 in section A5, one set in layer 6 in section B2, and four sets were made in layer 7 section B2.

<u>Penetrometer</u>: Measurements were made in 40 points in every compacted layer along the roof with a spacing of 0.1 m and a distance of approximately 10 cm from the rock surface. The penetrometer was only used in the 30/70 material. Measurements were also made in about 10 points from the crown and downwards with a spacing of about 5 cm. The radial measurements were made where the tangential measurements had indicated low-density values.

8.3.2 Measurement of water ratio and sampling

The water ratio was checked frequently at the preparation of the mixed backfill. In addition, it was measured in the backfill layers according to the following program:

Four samples were taken arbitrarily from the backfill material for each layer or after compacting it. The water ratio of all these samples was determined.

Two additional samples were taken in each layer and stored for future investigations, one in the top of the layer and one in the bottom for investigation of possible material separation.

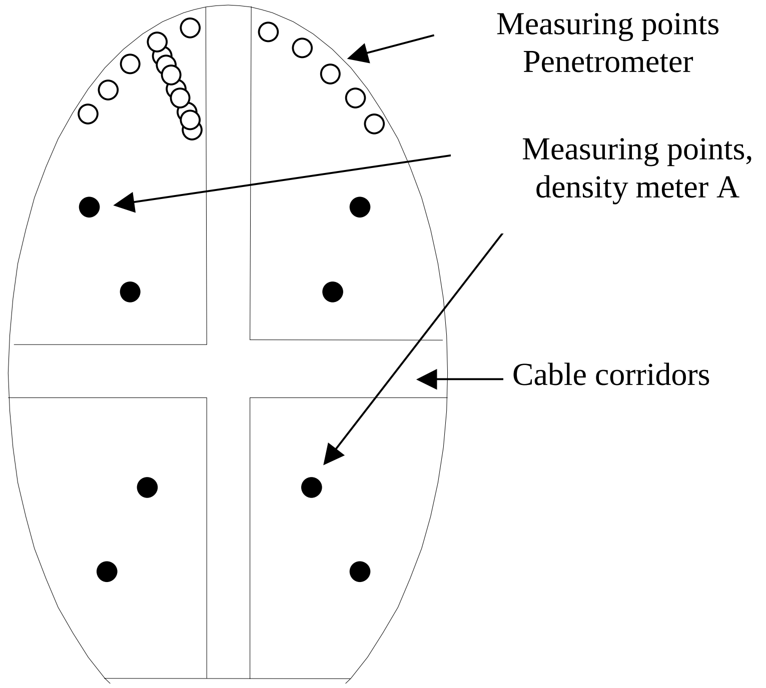


Figure 8-3. *Standardised pattern of density measurements in the backfill layers.*

8.4 Results from the backfilling of 30/70

8.4.1 Density in the centre of the tunnel

The average values of the dry densities measured in the 30/70 sections are shown in Figure8-4. The variations in density within the sections are shown in Figure 8-5. Normally, the density of the first two layers was lower than the average. This was probably due to the energy consuming properties of the permeable mats. In the two first sections the permeable mats consisted of three layers of drainage mats, two 4-mm mats and one 8-mm while only 4-mm mats were used in the remaining permeable sections. This decreased the damping effect and raised the density. For layer 5 and 6 in section A1 the compaction work was more extensive. Instead of the normal three passages with the slope compactor about six passages were made. This resulted in a higher density but also increased the mechanical stress in the slope compactor.

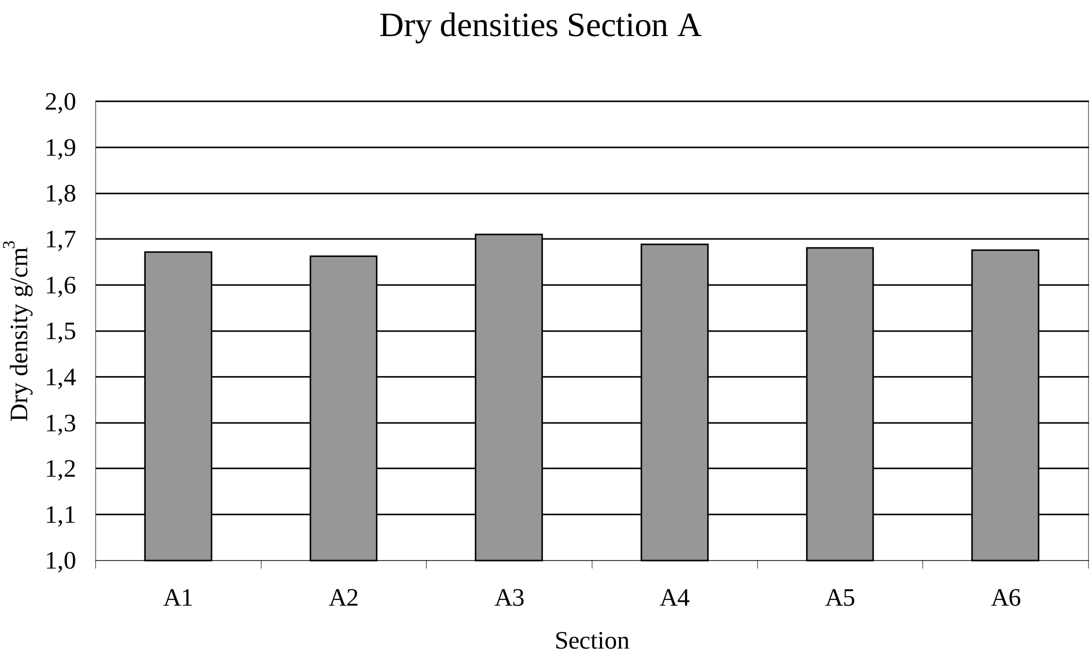


Figure 8-4. *Average measured dry density in the 30/70 sections.*

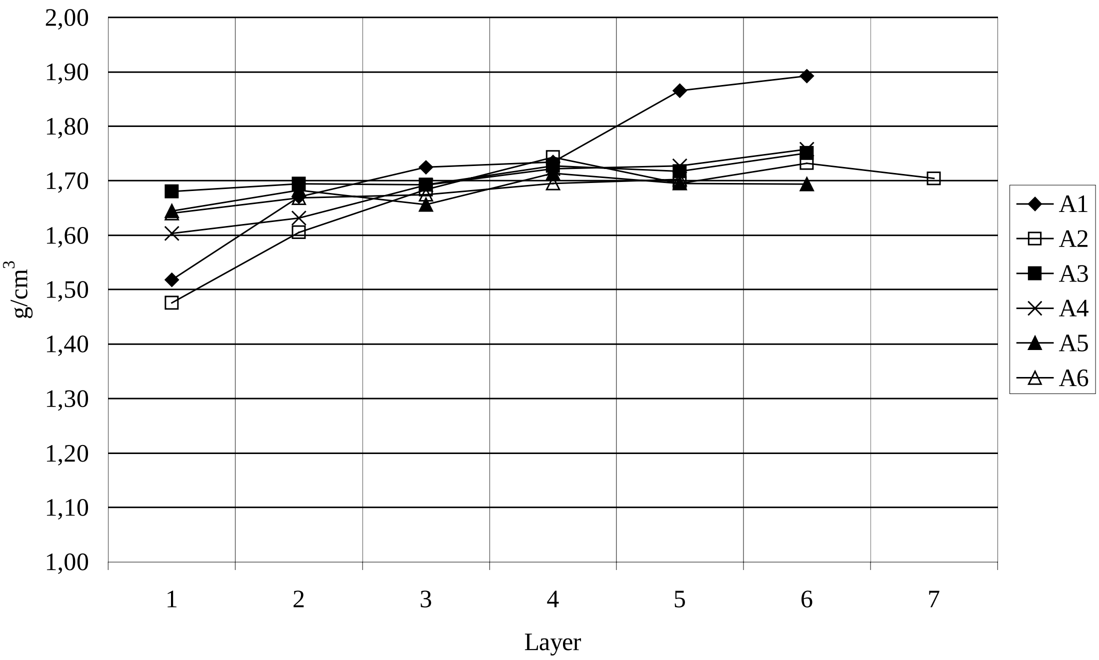


Figure 8-5. *Average measured dry density in all layers in the 30/70 sections.*

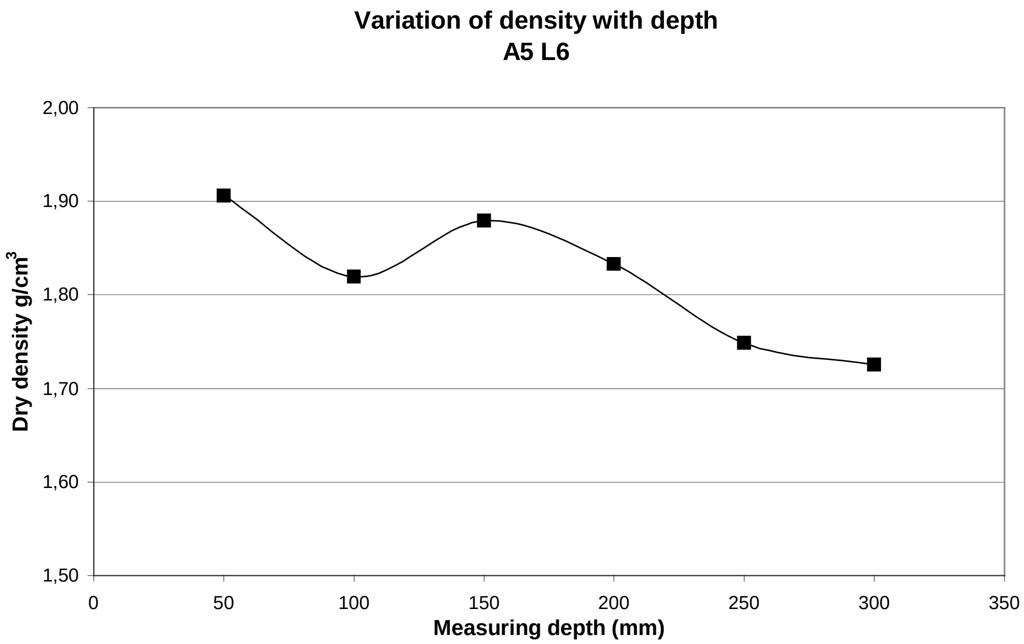


Figure 8-6. *Variation of density with depth in layer A5 L6.*

One set of measurements was made with density meter B in section A5 layer 6 (A5L6). The results are presented in Figure 8-6. In "Field Test of Tunnel Backfilling" /1-11/ it was observed that the density decreased with depth. The same tendency can be seen in Figure 8-6.

8.4.2 Density at the roof

The density at the roof varied depending on the topography of the rock surface and the presence of instruments near the roof. The compaction had to be made carefully to avoid damaging the instruments and this yielded a low density. The general trend was that the density at the roof increased during the course of the backfilling, which was partly due to the operator's growing experience. In the last two sections A5 and A6 most results from the measurements at the roof were above the measuring range. This can be seen in Figure 8-7 which shows the mean density evaluated from tangential penetrometer measurements.

The columns in the diagram represent the fraction of the measurements that represented values above and below 1500 and 1200 kg/m^3 (left axis). These densities correspond to the measuring range of the penetrometer. The right axis shows the median density of the material close to the roof for each layer. The density at the roof in the layers in section A1 and the first layer in section A2 was not measured. The density of these layers was too low to make measurements with the penetrometer. No open slots between the roof and the backfill were observed.

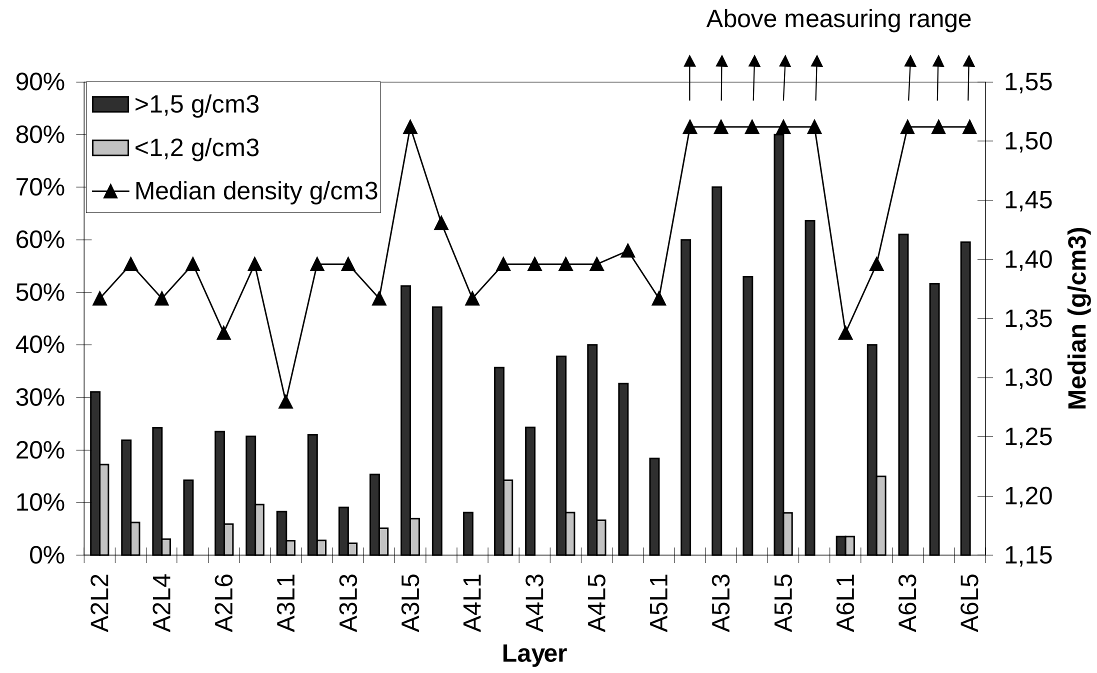
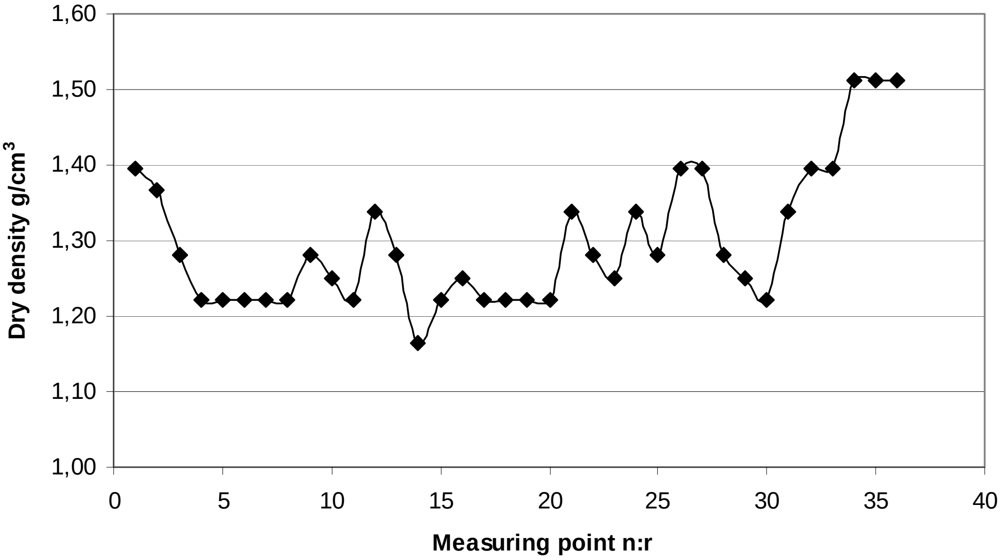


Figure 8-7. *Summary of measured densities close to the roof in the 30/70 backfill.*

In Figure 8-8 examples from penetrometer measurements along the periphery of the roof in two different layers (A3L3 and A5L3) are presented. These plots represent the lowest measured densities (A3L3) and the highest ones (A5L3).

The density of the first layer of most sections was lower than that of the other layers (e.g. A6L1). This was caused by problems with the upper permeable mats that were folded and attached to the irregular roof on the last layer of the previous section.

The radial measurements showed that the density increased with the distance from the roof. Two examples from A3L3 and A5L3 are shown. In general, the density exceeded 1500 kg/m^3 at 40 cm distance from the roof. Note that the radial measurements were made in the areas with the lowest density.



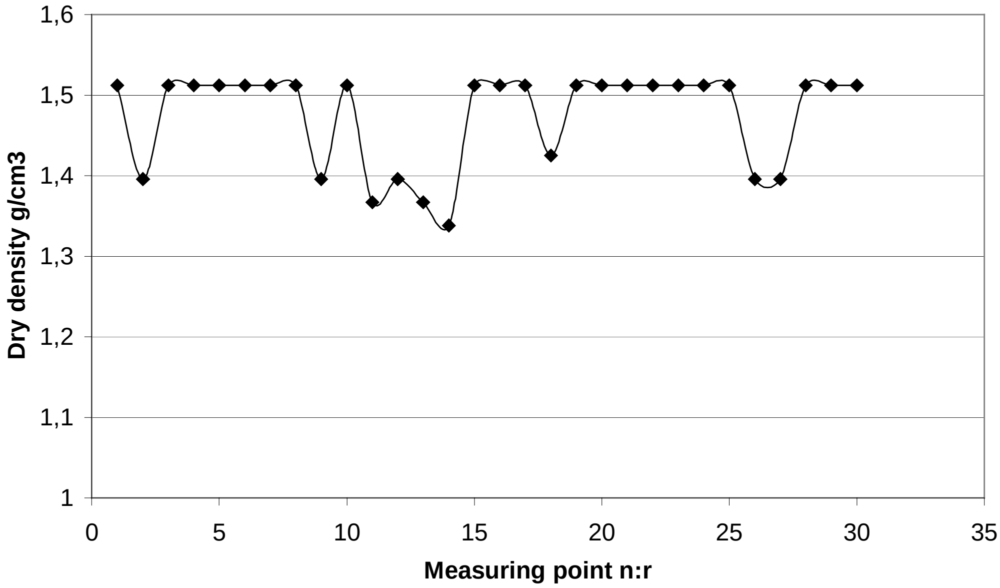


Figure 8-8. *Results from the penetrometer measurements at the roof in A5L3 (upper) and A3L1 (lower).*

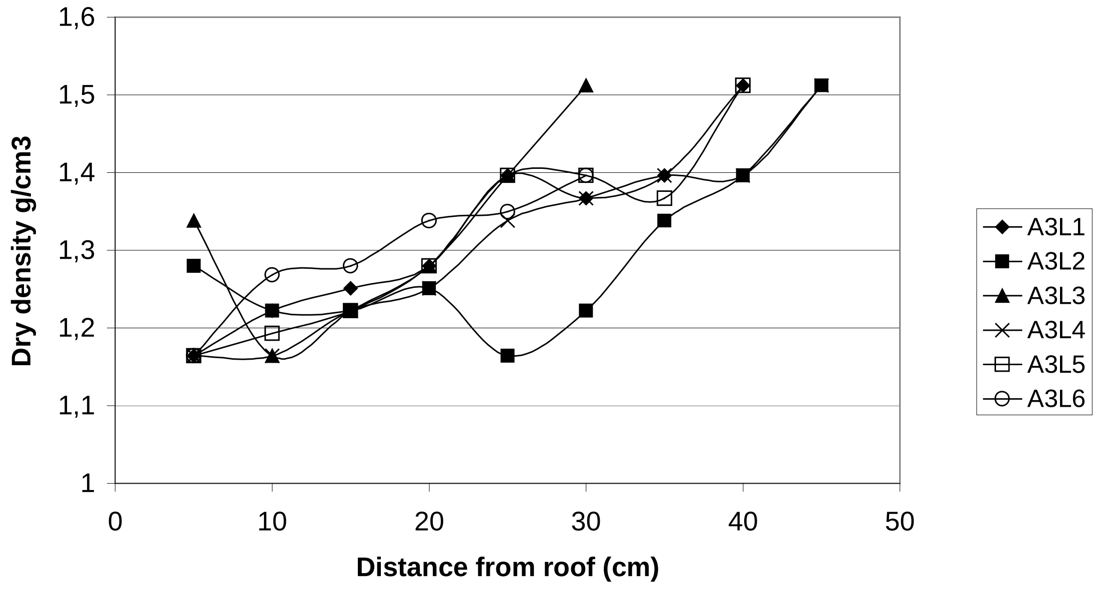


Figure 8-9. *Variation of density with distance from the roof in section A3 at places chosen for having low density.*

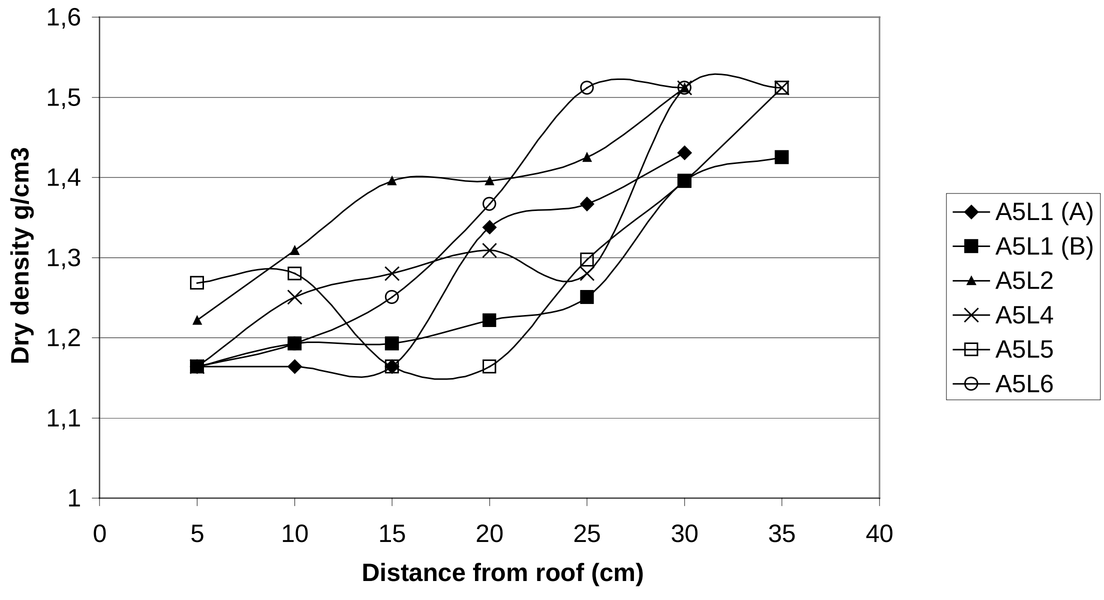


Figure 8-10. *Variation of density with distance from the roof in section A5 at places chosen because of the low density.*

8.4.3 Water ratio

The average water ratio of the layers of 30/70 varied between 10 and 13.5 % (see Figure 8-11). The samples were taken from the layers in the tunnel and the weight was in general between 1 and 1.5 kg. Each presented value corresponds to the mean value of four samples. They agree rather well with the water ratios measured during mixing (see Figure 4-5). The water ratio of the last section was lower than the rest. This material had been in the tent for the longest time period and had probably dried somewhat.

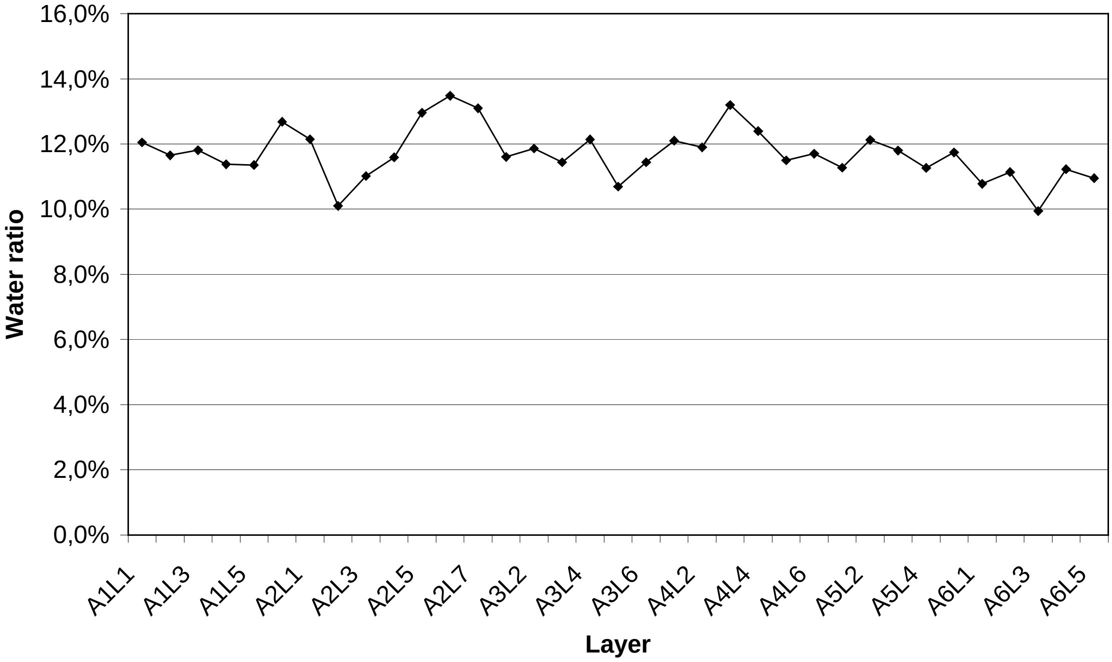


Figure 8-11. *Variation in water ratio in the 30/70 layers*

8.4.4 Weight of backfill material

The total weight of 30/70 backfill delivered to the tunnel was 503 tons. It made up the 36 layers, which yielded an average of 14.0 tons per layer. The weight per layer is shown in Figure 8-12. As the backfilling proceeded, the coarse material used for the road was removed from the tunnel. When the instruments in the floor had been installed, the floor was covered with 30/70 material that was weighed. 11.4 tons out of the total 503 tons were used for covering the floor. When the coarser material was scraped off some of the 30/70 material disappeared with it. This has not been accounted for.

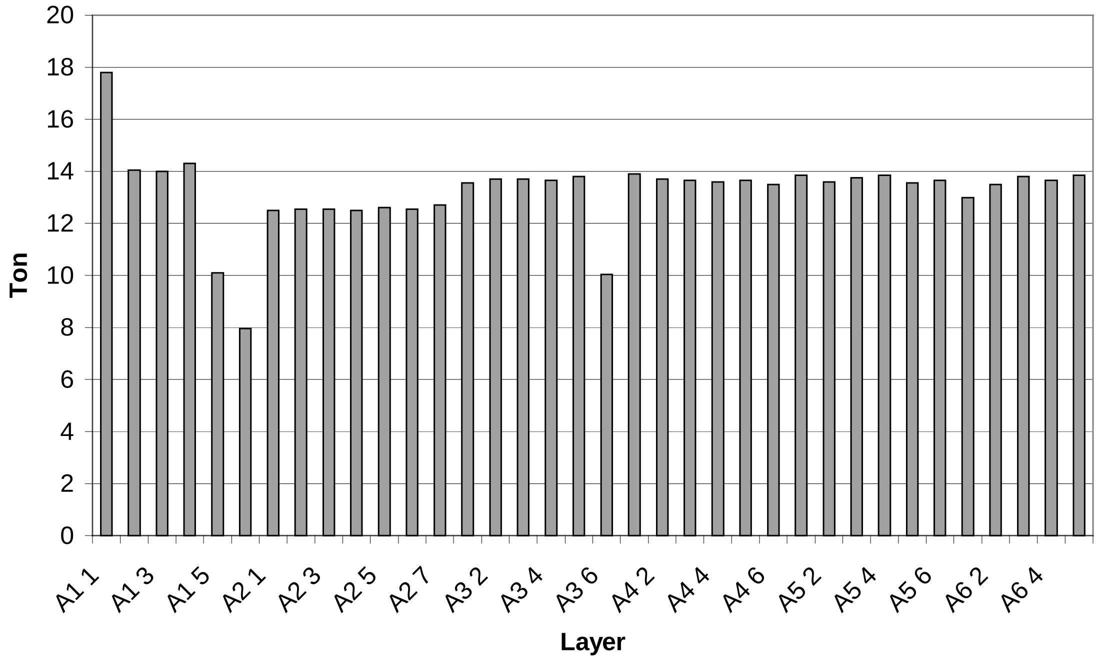


Figure 8-12. *Weight of each layer of the 30/70 backfill material.*

8.5 Results from the backfilling of 0/100

8.5.1 Density in the centre of the tunnel

The average values of the measured density in the 0/100 sections are shown in Figure 8-13. The mean measured dry density was 2170 kg/m^3. The variation in density within the sections can be seen in Figure 8-14.

In layer B2L7 four sets of measurements were made with density meter B (Figure 8-15). No general trend concerning the variation of density with depth can be identified although it seems that the top part of each layer has a lower density than the rest of the layer, a phenomenon that was also observed in earlier tests /1-11/.

8.5.2 Bentonite density in the gap at the roof

The gap at the roof was filled with bentonite blocks with a dry density of 1800 kg/m^3 and pellets with the dry bulk density 1050 kg/m^3. The gap was filled with one or two block layers and pellets were applied in the remaining space. The maximum dry density was 1800 kg/m^3 (two block layers, no pellets) and the minimum density 1430 kg/m^3 (one bentonite block layer and one block height filled with pellets). The average dry density (based on measurements on photos) was estimated at 1650 kg/m^3.

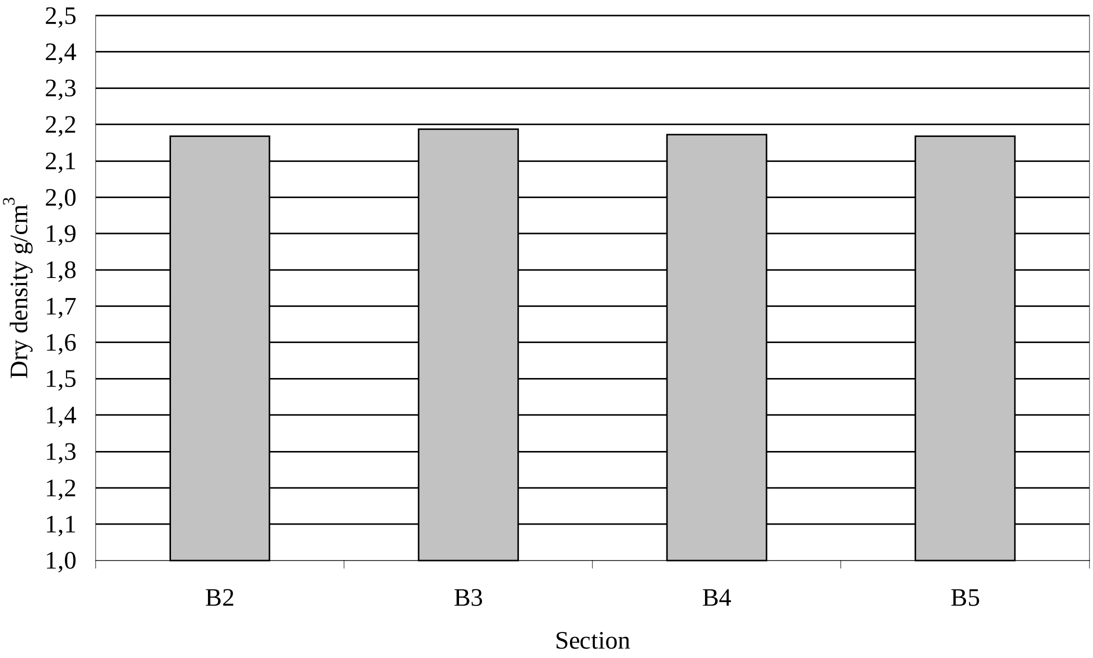


Figure 8-13. *Average dry densities of the 0/100 sections.*

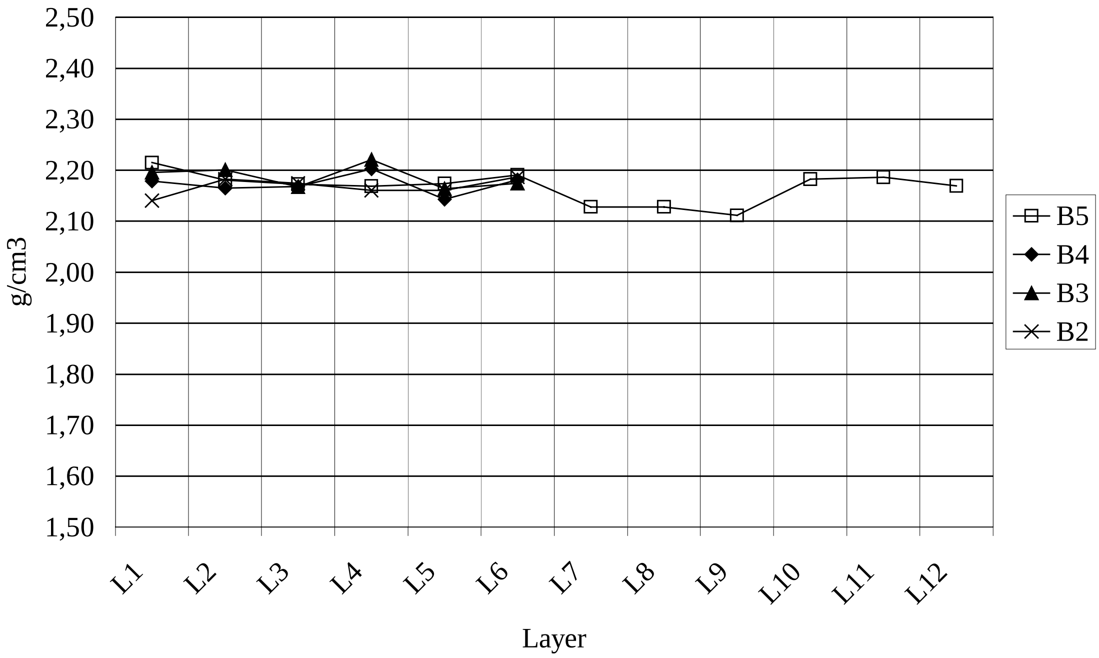


Figure 8-14. *Variation of density within the 0/100 sections*

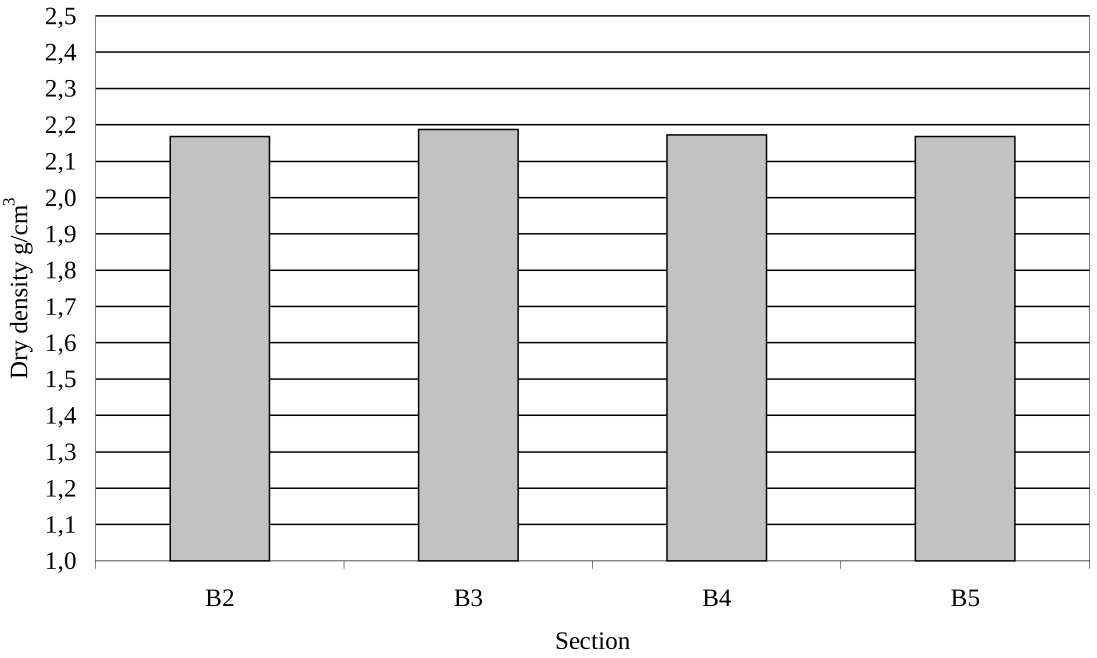


Figure 8-15. *Variation of density with depth in layer B2L7 of the 0/100 backfill*

8.5.3 Water ratio

The water ratio of the 0/100 backfill material varied between 5% and 6% (see Figure 8-16).

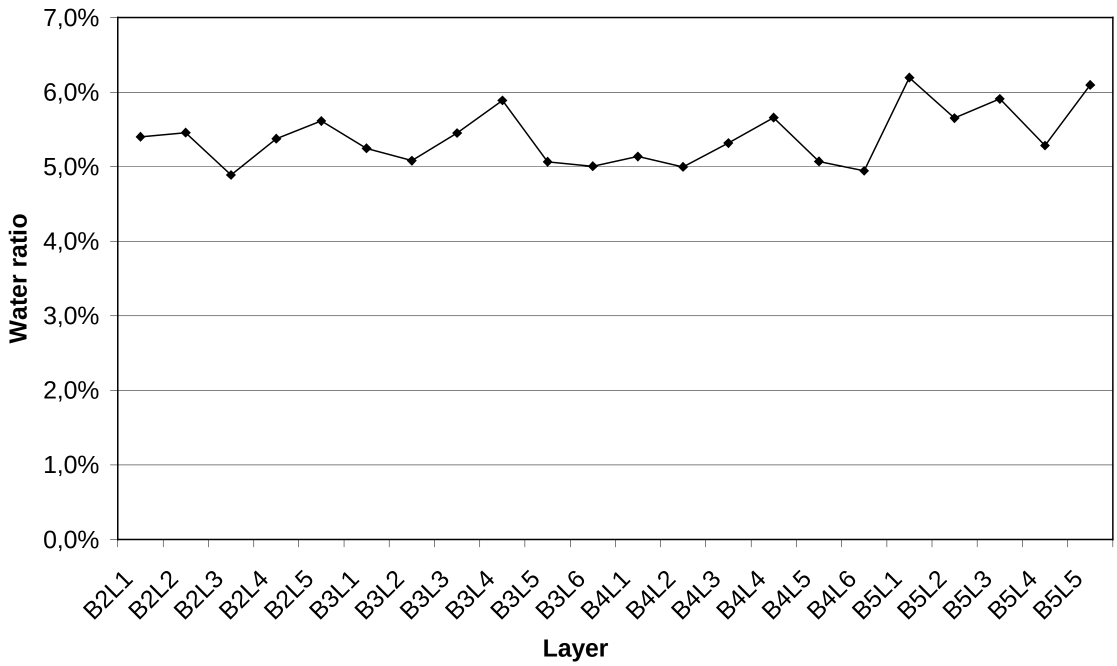


Figure 8-16. *Variation of water ratio in the 0/100 backfill*

8.5.4 Weight of backfill material

A total of 466 tons of 0/100 was transported into the tunnel. Figure 8-17 shows the weight of each layer, the average being about 17 tons. The backfill mass of the layers in section B5 was reduced in order to keep the thickness of the layers constant as the area of the layers decreased. The mass of layer 6 in Section B3 was also reduced.

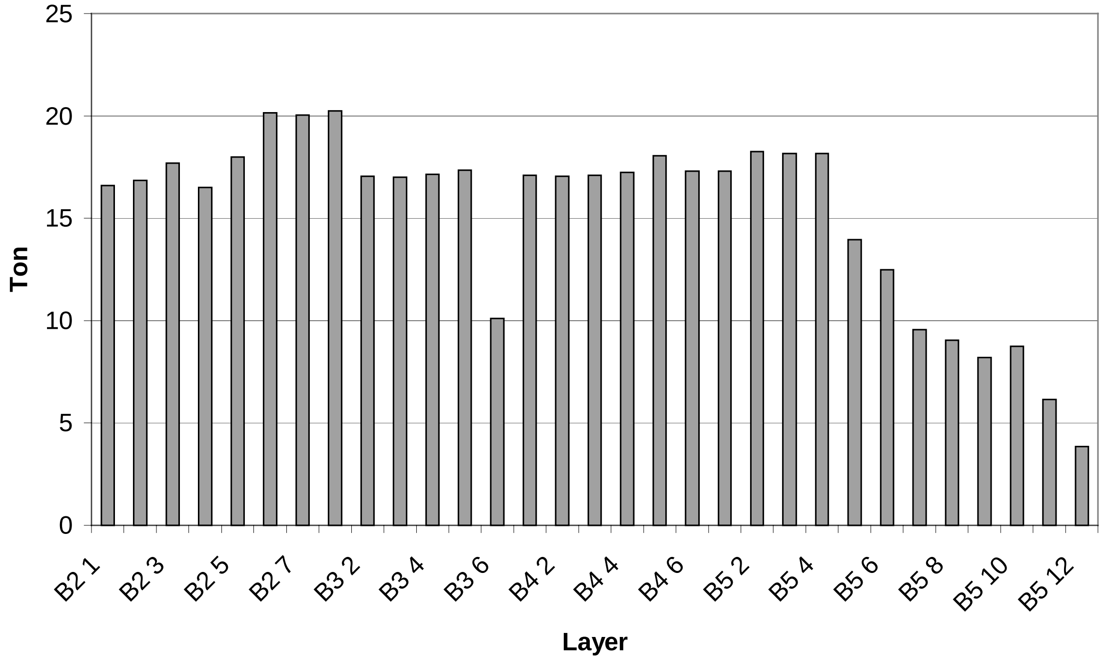


Figure 8-17. *Weight of the 0/100 layers.*

8.6 Density and swelling pressure of 20/80 blocks and pellets at the plug

The upper triangle between the roof and the plug (Figure 7-1) with the estimated volume 5.7 m^3 was filled with blocks made of 20% bentonite and 80% sand. In order to moderate the swelling pressure from the blocks a part of the space was filled with bentonite pellets and a small part at the roof was left unfilled. The design was planned to yield a net swelling pressure of about 300 kPa after swelling of the blocks and compression of the pellets, filter and backfill. It is estimated, based on swelling pressure measurements on mixtures with 10-30 % bentonite content /1-2 and 8-1/, that the dry density of 20/80 must be about 1900 kg/m^3 to yield this swelling pressure.

The following volumes (estimate) and weights of blocks and pellets were installed in this space:

Blocks 20/80 (ρ_d = 2210 kg/m3):

1996 blocks with 3.5 kg weight and 4% water ratio were originally placed on the upper layer of compacted 0/100.

Volume: 3.09 m3

Dry weight: 6 850 kg

Pellets (average ρ_d at emplacement = 1135 kg/m^3)

2 112 kg pellets with the water ratio 10% were placed

Dry weight: 1 920 kg

Volume: 1.69 m^3

Empty space:

4% gap between blocks yields the volume 0.12 m^3

Estimated remaining space at the roof is thus 5.7–3.09–1.69–0.12= 0.8m^3

Space for swelling

It is estimated that 300 kPa swelling pressure of 20/80 will cause a compression of the backfill and filter with 1 cm and compression of the pellets to ρ_d = 1235 kg/m^3. These compressions yield an extra space for swelling of 20/80 of

Filter: 0.05 m^3

Backfill: 0.08 m^3

Pellets: 0.14 m^3

Sum: 0.27 m^3

Extra blocks (20/80)

In order to reach the desired dry density of the 20/80 (1900 kg/m^3) 379 extra blocks were applied in the remaining space (dry weight 1276 kg)

Final dry density

The average dry density of 20/80 will thus after swelling be

ρ_d = (6 850 + 1276 kg)/(5.7 – 1.69 + 0.27 m^3) = 1910 kg/m^3

Summary

The empty triangular space between the backfill, the roof and the plug is estimated to have the volume 5.7 m^3. It was filled with blocks and pellets with the following properties:

- Altogether 2375 <u>blocks</u> of 20/80 were placed. The average dry density after swelling and homogenisation will 1900 kg/m^3 and yield the estimated swelling pressure 300 kPa.
- 2112 kg bentonite <u>pellets</u> were placed. The average dry density will after compression and homogenisation be 1240 kg/m^3 and the estimated swelling pressure will be 300 kPa.

The estimated properties are very preliminary, since it was difficult to measure the volume and the homogenisation takes a long time and will not be complete. It is thus probable that there will be strong local deviations from these estimated average properties.

8.7 Comparison with earlier tests

The "Field Test of Tunnel Backfilling" involved backfilling of a part of the TBM tunnel in the Äspö HRL with material containing 0, 10, 20 and 30% bentonite, respectively /1-11/. The average densities obtained in this field test and in the Backfill and Plug Test are expressed in terms of "Proctor density" in Figure 8-18 and as dry density in Figure 8-19. The density representing 100% Proctor is different for the material used in the Field tests (1940 kg/m^3) and Backfill and Plug Test (2010 kg/m^3). This shows that the densities are higher in the Backfill and Plug Test with 30/70 material but lower for the 0/100 material.

The compaction in the Field Test of Tunnel Backfilling was very effective in the central area of the tunnel (about 1800 kg/m^3 for 30/70). In the Backfill and Plug Test very high (1900 kg/m^3) densities were achieved in layer 5 and 6 of section A1. Achieving such a high density required about twice the compaction time and yielded a severe risk of damaging the compaction equipment. The higher risk for damage is due to that the material became very dense at 1700 kg/m^3 density. Further compaction on this hard surface results in vibrations propagating through the compactor to the carrier, causing a much higher wear of both compactor and carrier. After compaction of these layers the strategy was changed. The material was compacted until the driver of the carrier felt the vibrations in the carrier. This usually required three passages with the slope compactor. The average measured dry density was then 1690 kg/m^3.

The "percent Proctor" reached in the centre of the tunnel in the Field Test of Tunnel Backfilling was higher than achieved in the Backfill and Plug Test.

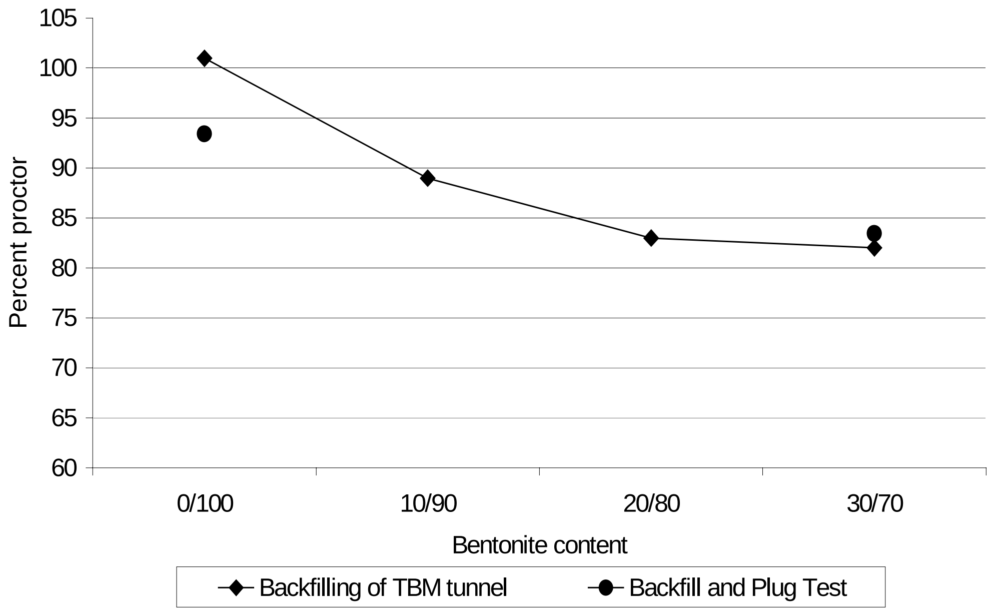


Figure 8-18. Comparison of measured "percent Proctor" with earlier tests.

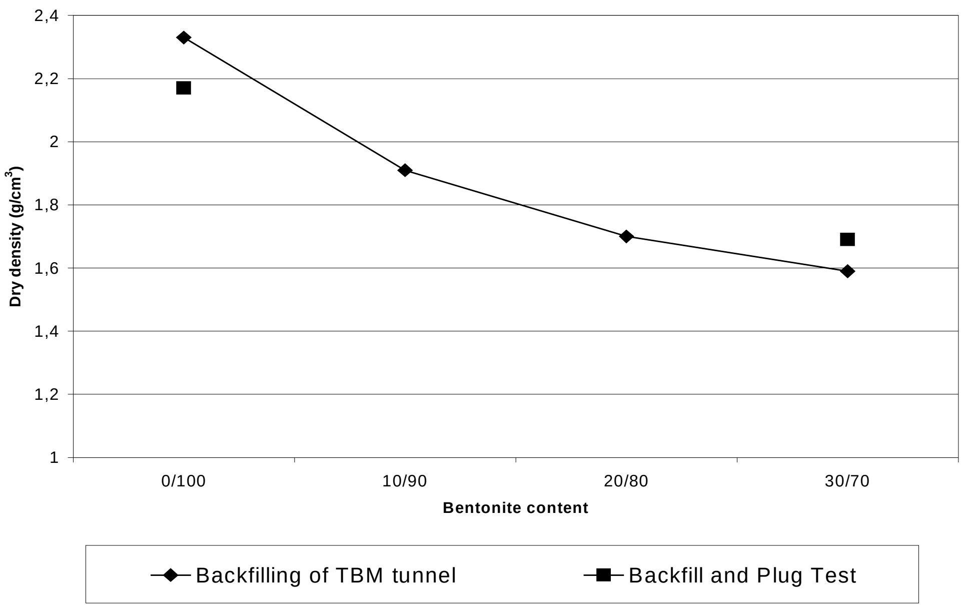


***Figure 8-19**. Comparison of measured dry densities with earlier tests*

9 Cable lead throughs

9.1 Introduction

It was important to make the lead-throughs for all cables and tubes watertight for maintaining a high water pressure in the test tunnel. The design and installation of the lead-throughs are described in this chapter.

The cables and tubes were led in twelve pipes in boreholes drilled in the rock between the Backfill and Plug Test and the neighbouring Demonstration tunnel (see Figure 9-1) and then from the Demonstration tunnel to the measuring container located in the D-tunnel.

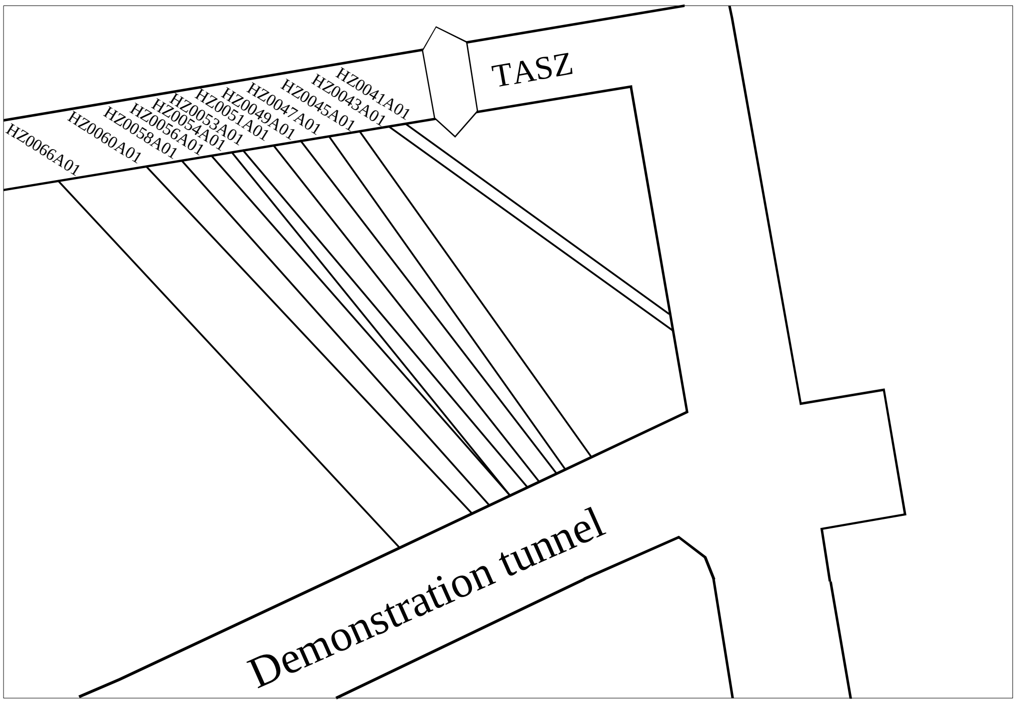


Figure 9-1. *Location of the lead-through boreholes*

9.2 Design

Two different principles of conveying information from the test site to the measuring container were used: electrical signals by use of cables and hydraulic pressure by use of tubes. All electrical cables were led in tubes. The tubes were led through steel pipes via steel cones (Figure 9-2) to the Demonstration tunnel.

Each tube was led through two Swagelok ferule connections with cutting rings mounted in a steel plate that was located in the Backfill tunnel (Figure 9-3). The steel plates were 38 mm thick and made of stainless acid proof steel SS2343-28. Dimension standard is DIN2527 PN 25. The two upper tubes in Figure 9-3 show the state before the ferrule connections were tightened and the other three tubes show the installed connections.

The connections ensured that no leakage could occur between the tube and the steel plate. The steel plate was screwed to a steel cone, which in turn was welded to the pipe leading to the neighbouring tunnel (see Figure 9-2). The steel cones were manufactured from 6 mm 2343 stainless steel sheet.

The diameter of the boreholes is 127 mm. The 2.5 m long part of the borehole closest to the test tunnel was widened to 150 mm diameter. The steel cone and flange were placed in the rock wall of the Backfill and Plug Test tunnel in order to facilitate the compaction of the backfill.

The steel pipes have the dimension 88.3x4.05 mm^2 (DIN 17175 ST 35,8/I) and were equipped with spacers to keep the pipes from being in contact with the borehole. The pipe lengths are specified in Appendix 6.

A bentonite packer system was used to cut off the flow paths along the boreholes. Bentonite rings were placed over 0.6 m length of the steel pipe close to the flange and steel plate (see Figure 9-5). The saturated density of the bentonite in the packer after swelling was estimated to 1950 kg/m^3 corresponding to 3.5 MPa swelling pressure. The bentonite rings were kept in place by steel rings welded to the pipe. Rings of PUR-rubber were placed between the bentonite and the steel rings, and between the cement and the inner steel ring to ensure that no bentonite or concrete could migrate past the metal rings. The variation in density in a bentonite block is given in Appendix 7.

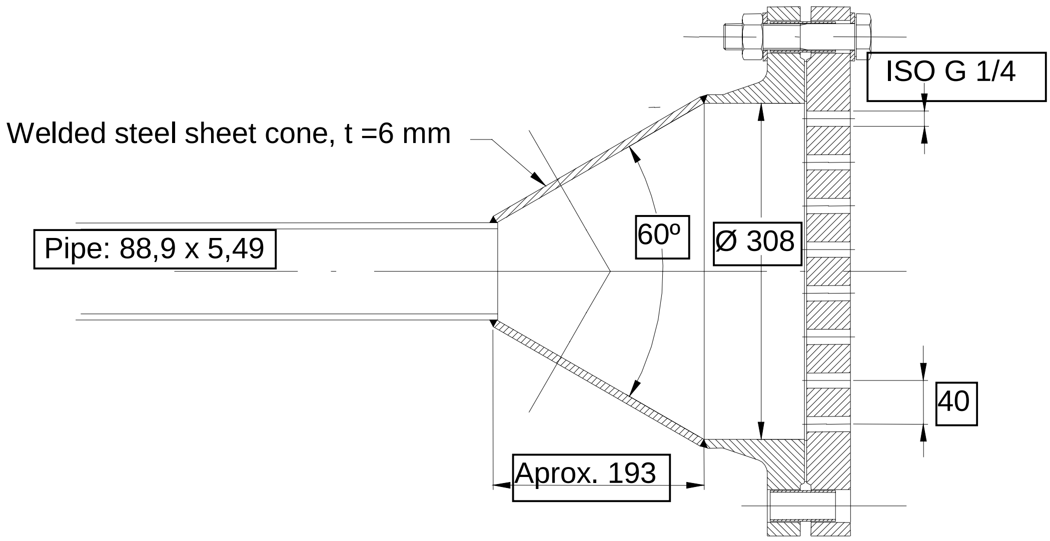


Figure 9-2. *Drawing of a cross section of the steel cones.*

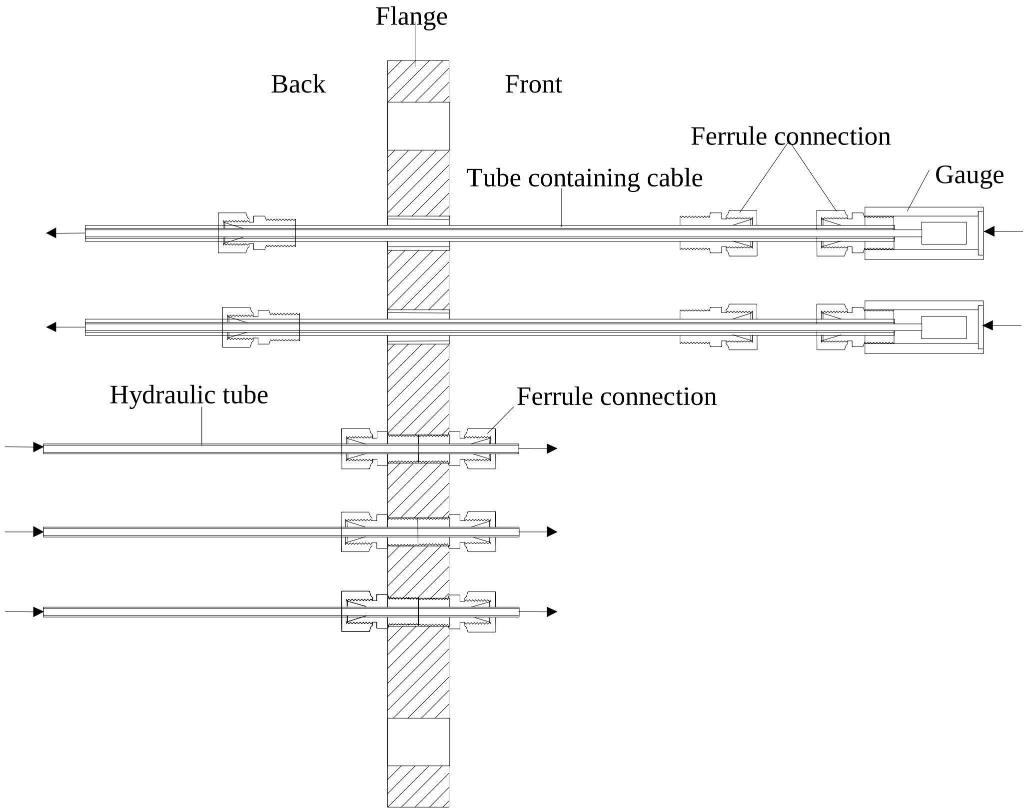


Figure 9-3. *Lead-through connections in a steel plate.*

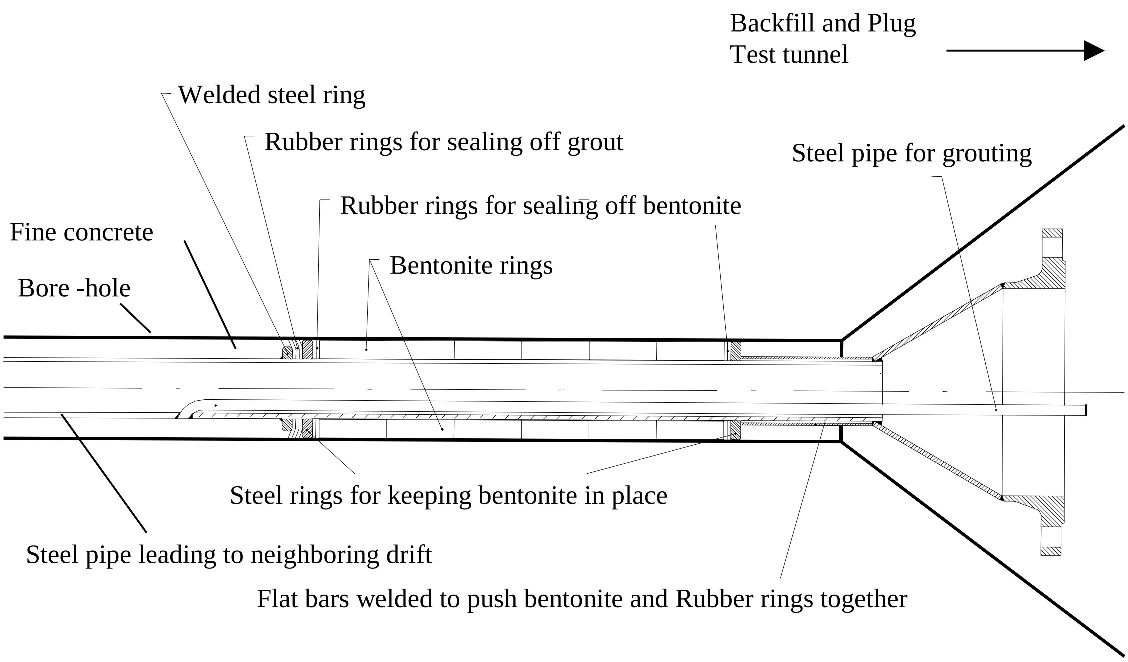


Figure 9-4. *Overview of the lead- through arrangements at the test tunnel*

The remaining volume in the borehole between the pipe and the rock was filled with fine cement-based grout delivered by BEMIX (see chapter 9.5). The purpose of the grout was to serve as a backup seal and to prevent water-bearing fractures intersecting a borehole from being short-circuited.

9.3 Boreholes

The boreholes for the lead-troughs were placed and directed so as to avoid intercepting major water-bearing fractures. The local structural model /3-1/ was used for determining the locations of the boreholes. Water inflow in the boreholes would make the installation of the bentonite packer system and casting of concrete difficult. The holes were bored from the Backfill tunnel to get them accurately positioned.

Pilot holes (drill bit diameter 64 mm) were drilled and then grouted and re-drilled to eliminate water leakage. The boreholes were widened with a 127-mm drill bit over the entire length of the hole. A 1.5 meter part of the borehole closest to the backfill tunnel was finally widened to 150 mm diameter.

At inspection it was found that most of the 150-mm boreholes were eccentric and their axes not parallel to the axes of the 127 mm drill bit bore holes. To make the installation of the pipes and bentonite packer system possible, the widening with the 150 mm drill bit was extended by 1 m to 2.5 m.

Rock was excavated to form a cone around the borehole ends (except for HZ0041A01) in the Backfill tunnel by boring and hydraulic fracturing (DARDA technique). Blasting was not used, since this would have extended the disturbed zone into the part of the borehole where the bentonite seal was located.

Before the pipes were installed in the boreholes a 9 meter long 99 mm dummy pipe with spacers giving the diameter 120 mm was pushed through the holes to ensure that the holes were straight enough for the installation of the lead-throughs. It was concluded that the lead-through pipes could be installed with spacers in all holes except for the following three: HZ0047A01, HZ0051A01 and HZ0066A01. In two of those the spacers were not installed (cf. Appendix 6) and in hole HZ0066A01 the pipe diameter of the part closest to the demonstration tunnel was decreased to 76 mm and no spacers were attached. This was possible because of the small number of cables and tubes. The geometry of the 150 mm widening and the excavation for the cone for each lead through were also checked, which led to a reduction from 148 to 145 mm in the outer bentonite ring diameter.

9.4 Installation of pipes and bentonite seals

The pipes were delivered to the Backfill tunnel in 3 m pieces, inserted into the hole and welded together. The spacers were welded to the pipe according to Appendix 6. Since the boreholes were curved it took some force to insert the pipes. The outermost parts of the pipes were made of acid proof stainless steel.

A small mobile tent was used for preventing water from the roof to affect the welding. An adjustable steady rest was manufactured so that the jointed pipes were straight.

Before the last section of the steel pipes was inserted into a borehole, a steel ring with 125 mm diameter was welded to the end of the pipe. Three rings of PUR rubber were placed in the hole as shown in Figure 9-5. A detailed drawing of the inner part of the packer is shown in Figure 9-5. The purpose of the rings was to prevent the cement that was to be filled in the pipe from leaking into the bentonite packer. After this, a steel ring with 140 mm outer diameter was inserted. This ring centred the pipe at that location. Since the axes of the 127 mm and 150 mm bore holes were not parallel, the pipe was extended into the tunnel so that it could be centred by hand. Two PUR rings with the function of stopping the bentonite from swelling past were installed. Then the bentonite rings were pushed in place. Despite the fact that the outer diameter of the bentonite rings had been decreased there were quite some problems with inserting them. Some of the 150 mm boreholes were very curved. Special equipment had to be arranged to get the bentonite rings properly in place. The first step in getting the rings in place was to use a “slide hammer”, which is a heavy pipe sliding on the 89 mm pipe combined with a plastic ring between the bigger pipe and the bentonite ring. Outside the bentonite and PUR rings a 125 mm steel ring and a 140 mm steel ring, with the same function as above, were inserted. A simple jack construction was used for, via axial spacers, compressing the bentonite between the two 140 mm steel rings. When the intended volume was achieved the spacers were welded to the pipe and thereby the volume of the system was fixed.

The length of the pipe was adjusted so that it just reached the conical cavity in the rock. The pipe was then welded to the cone. The length of the injection pipe was adapted so that the mounted coupling and valve were inside the cone.

9.5 Casting of casing

The space between the casing tube and the borehole was injected using BEMIX injection grout and conventional injection equipment. At the borehole end in the Demonstration tunnel the space between the pipe and the rock was filled with fast hardening concrete. A short tube was installed in each hole for de-airing purposes.

The BEMIX grout was injected through the steel pipe (see Figure 9-4) until it came out from the de-airing tube in the Demo tunnel. The grouting was ended by shutting the valve of the pipe.

Before the cable packages were installed the valves were replaced with plugs.

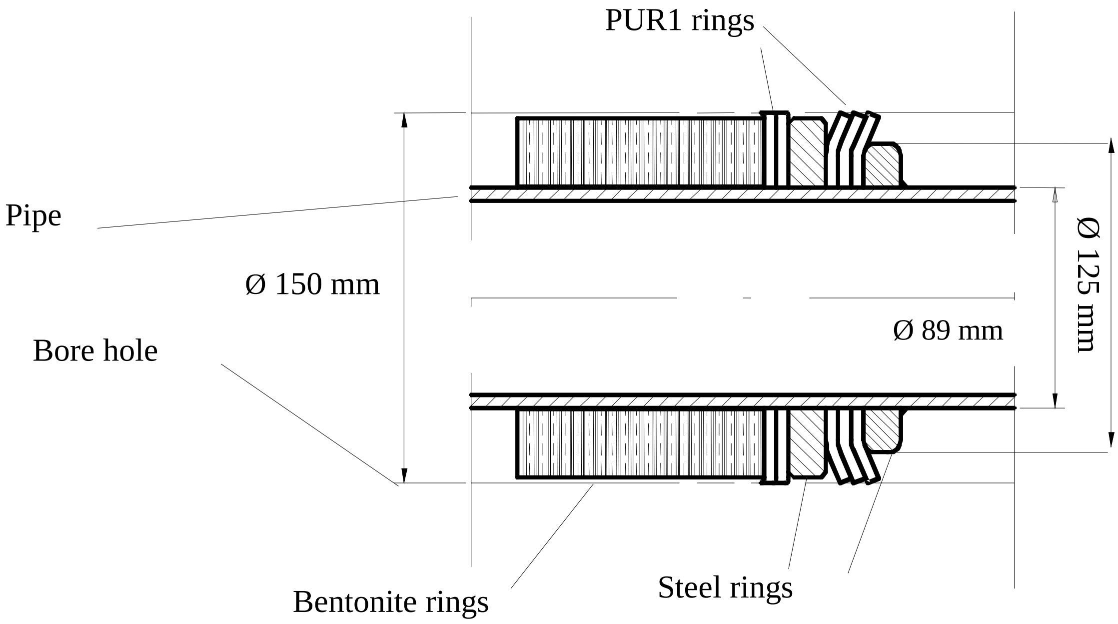


Figure 9-5*. Detail of the bentonite packer system*

9.6 Assembly of cable package

The cable packages were assembled above ground in the store and garage building of Äspö HRL. The average cable length was 90 m and the assembly required a lot of space. This meant that the cable packages were somewhat difficult to handle.

A special rack on wheels had been constructed for holding the steel plate during the preparation of the system of cables, tubes and flanges (see Figure 9-6), for the transport down the tunnel and installation.

The assembling of the cable package was done according to the instrumentation plan, and each tube was checked and marked. To simplify the handling and identification of the tubes they were given different colours depending on their role. Blue tubes were used for measuring water pressure in the backfill, white tubes for Glötzl total pressure cells, and silver-coloured tubes for Glötzl pore water pressure cells. Black tubes were used for the packers in the rock. Beige tubes were used for monitoring water saturation. All tubes containing cables were standard transparent white. When the tubes were in place, they were marked with identity tags at each end. At this stage the inner ferrule connections were tightened and the package strapped together, using approximately one strap every 0.5 m. A lot of effort was put into making easily handled packages. Once a cable package was strapped it was shaped as a bundle and placed on the rack with a steel plate (see Figure 9-7). The rack was placed on a wooden pallet that was transported down to the test tunnel.

Figure 9-6. *The rack with an assembly of cables and a flange.*

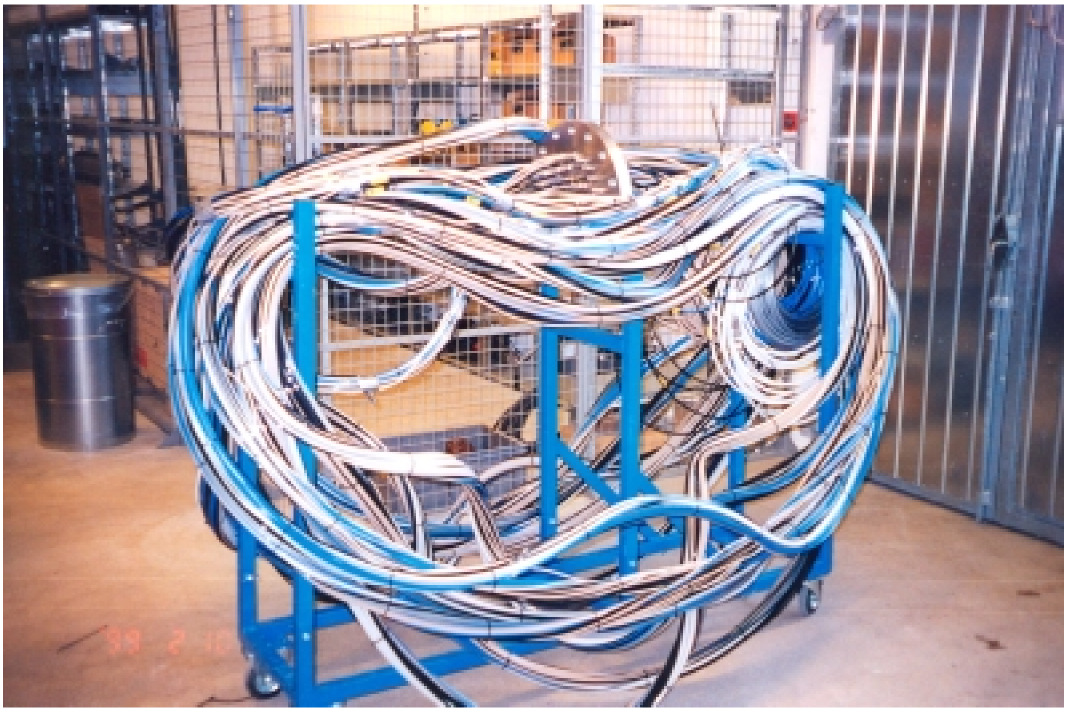


Figure 9-7. *A cable bundle on the rack ready for installation.*

9.7 Installation of cable package

Before a cable package was transported to the test tunnel a wire had been pulled through the installed pipe and cone. The plane surface of the cone, to which the flange was attached to the cone, was cleaned. The rack was moved into the Backfill tunnel and the cable package was put on the ground. The seal between the steel cone and the flange was put in place. The end of the package was connected to the end of a wire in the backfill tunnel. The package was pulled through (Figure 9-8) and temporarily hung on the construction frame in the Demonstration tunnel. When almost the entire package was drawn through the pipe the steel plate was lifted from the rack and brought towards the steel cone. The plate and cone were finally brought in contact and screwed together (Figure 9-9).

Figure 9-8. *Guidance of a cable package through a cone and tube.*

Figure 9-9. *Fastening the plate on the flange of the cone.*

9.8 Leakage testing

The tightness of all through connections was tested in three different leakage test series.

In the first test series the volume between the casing and the borehole was filled with water and the water level at the upper open end of the holes monitored. Under this low pressure gradient all holes except HZ0053A01 were tight. HZ0053A01 turned out to be connected to KXZRD6H, a 3.1 m long hole that was sealed. On inspection with borehole camera it was noted that water from HZ0053A01 came from a hole two meters into KXZRD6H. This part of the borehole was packed off during the grouting of the casing in HZ0053A01.

In the second test series the casing was pressurised from the inside. Hydraulic packers were placed at both ends of the controlled casing. The closed volume was filled with water from the end of the casing at the test tunnel. When the entire volume of the pipe had been filled, the packer in the Demonstration tunnel was closed. Pressure vessels and nitrogen was used for increasing the pressure to about 1 MPa. To detect significant leakage the level in the pressure vessel was monitored. The pressure in the casing was monitored for about two hours.

It was concluded that all the casings except one (in HZ0043A01) were tight. The leakage of this casing was located about 3 m from the test tunnel. When this hole was grouted a packer was placed in the leaking section, which prevented grout from entering the casing.

The third test series was performed in the following steps:

1. The ferrule connections on the low pressure side of the plate were tightened when the tubes were drawn through the plate.

2. After the cable package was installed in the bore hole an air pressure of about 2 bars was applied inside the pipe and cone. Leakage testing fluid was used for detecting leakage. A special temporary seal was used for sealing the other end of the pipe (see Fig 9-10). Leakage was detected in two of the plates.

3. The ferule connections on the high pressure side of the steel plate were tightened.

4. The test was repeated for the two leaking steel plates. No leakage could be detected at these tests.

Figure 9-10. *Temporary seal for the third leakage test series*

10 Surveying of geometry

10.1 General

A laser beam was placed in the centre axis of the test tunnel to serve as a reference line when placing instruments on the compacted layers. During the backfilling the position of the following parts were measured with laser technique:

- The surface of the inclined concrete wall
- The tunnel profile (about 10 points per layer at its contact with the rock)
- Each compacted layer (6 points not counting the points in the profile)
- The permeable mats (about 16 points per section)
- Each instrument installed in the backfill
- The prefabricated concrete beams separating the backfill from the plug

10.2 Equipment

The laser instrument that was used for providing the central line of the tunnel was a Latronix model LD3-635P-230. The following two types of surveying stations were used:

1. Geodimeter 468DR with field computer Goepad. This device has a built in laser pointer and does not require a reflector.
2. Leica TC1610 and field computer Psion Geodos.

The code SBG Geo was used for calculating the co-ordinates.

10.3 Denomination

The surveyed objects have an identity that consists of 9 letters or numbers. The first position describes the type of object. In The Backfill and Plug Test the following 5 types of objects dominated: Area (A), Point (P), Tunnel (T), core drilled hole (K) and percussion borehole (H). The second and third positions state the actual experiment, which are XB for the Backfill and Plug Test. The remaining six positions are used for identities specific for the Backfill and Plug Test. These are identical to the instrument identifications presented in Chapter 11.

11 Instrumentation and other installations

11.1 General

This chapter describes the instrumentation for measuring hydraulic and mechanical processes in the Backfill and Plug Test. The water pressure in the rock is measured in boreholes in 75 sections separated by packers. In the backfill, 34 pore water pressure sensors have been installed while there are 57 sensors for monitoring the water saturation and 21 cells for recording total pressure. The water pressure in the drainage layers of filter mats is measured in all 12 layers. Four pressure cylinders have been installed for measuring the mechanical properties of the backfill. In section A4, 13 local permeability probes were installed. The system for measuring this property was developed and installed by AITEMIN and is described in Appendix 8. Micro-organisms have been put in both in the 30/70 backfill (section A5) and in the 0/100 backfill (section B2). These installations are described in Appendix 12.

The positions of the measuring points in the backfill are numbered and described in relation to the backfill section, the number of the compacted layer, the tunnel axis, and the rock surface. Those in the rock are numbered and described in relation to the borehole number and the measuring section in the borehole.

11.2 Location of backfill instruments

11.2.1 Strategy for describing the position of each measuring point

Each instrument in the backfill was given a short unique name consisting of 1-2 letters describing the type of measurement, and of 1-3 figures numbering the device. In addition to the name a short description of the position was added.

The sections, separated by permeable mat layers, were shown in Figure 7-1. Sections A1, A3, A4, A5, A6 and B1-B4 consist of 6 layers with the intended thickness 0.21 m according to Figure 7-2. Section A2 consists of 7 layers and A5 of 5 layers. This was done to keep the pre-determined positions of the sections. Section B5 consists of 12 backfill layers. In practice the layer thickness varied somewhat within and between the layers. Each layer corresponds to one compaction sequence. As described in chapter 2, Section B1 was exchanged for section A6.

The instruments were placed in the backfill after compaction and the designations are related to the actual layers. Each measuring point is also defined by the co-ordinates in the layer. The *x*-co-ordinate is the horizontal distance from the centre of the tunnel and the *y*-co-ordinate is the distance perpendicular to the *x*-axis along the layer. Some instruments have been placed at a special distance from the rock. For these cases the co-ordinate begins with the letter R and is then given the co-ordinates for the intersection with the rock surface. Minus sign refers to the distance from roof and + the distance from floor. An instrument in the backfill is thus termed in the following way:

1. Type of measurement (1 letter)
2. Serial number (1-2 figures)
3. Section (1 letter, 1 figure)
4. Layer (1 figure)
5. *x*-coordinate
6. *y*-coordinate

Items 1 and 2 identify the device and items 3-6 describe the location. A pore water pressure transducer (number 8) located in section A2, layer 3, 0.5 m to the left of the centre line and 0.3 m below the roof in the y-direction was thus termed:

W8 (A2/3/-0.5/R-0.3)

Section B5 contains more than 6 layers, the numbering being shown in Figure 7-3. Section B6 consists of layers of highly compacted blocks of backfill with 5 cm thickness and the layer number refers to the block layer number (Figure 11-1). The y-coordinate R-C-C means that the transducer is placed in the centre of the layer.

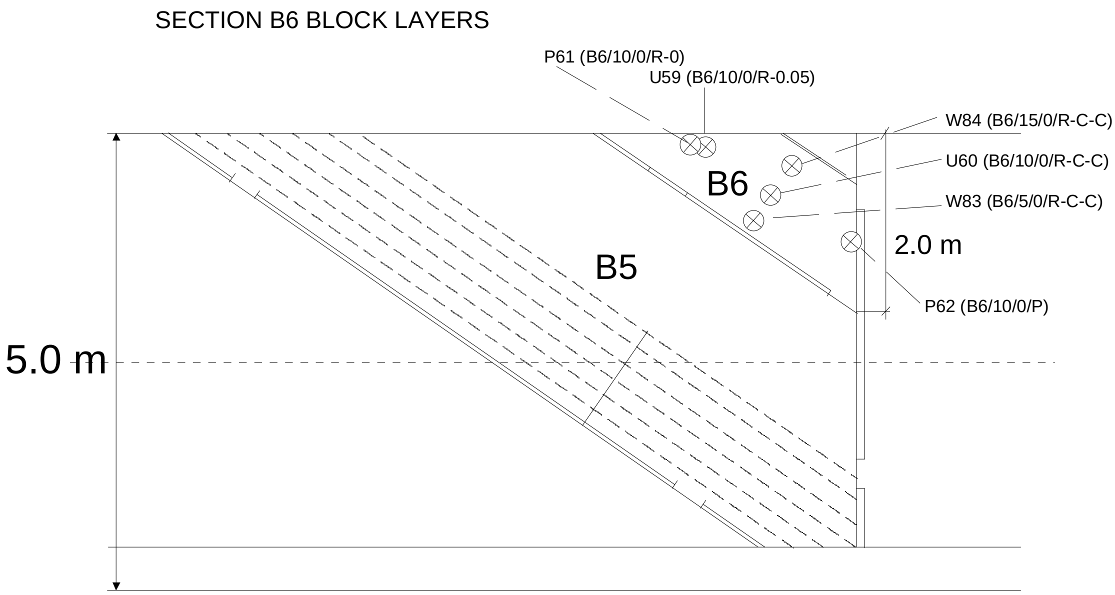


Figure 11-1. *Instruments in backfill section B6.*

11.2.2 Position of each instrument in the backfill

The aim was to place all instruments in layers 1-4 in order to keep the two outer layers free from transducers and cables. A further reason was that the end plate, where the tubes were attached to the through-connections, was placed in layers 1-4, which meant that the two final layers could be compacted without problems.

The positions of the instruments, except for ENRESA:s hydraulic conductivity devices which are described in Appendix 8, are described in Tables 11-1 to 11-4.

Tabell 11-1. Numbering and position of instruments for measuring total pressure (P) (parallel means that the cells are placed parallel to the surface of the backfill layer)

Type and number	Section	Layer	X (m)	Y (m)	Fabricate	Remarks
P1	A3	1	0	R+0.2	Glötzl A	Horizontal
P2	A1	5	0	R-1.1	Glötzl A	Parallel
P3	A2	3	0	0,6	Rocktest	Parallel
P4	A2	3	0	R+0.65	Rocktest	Parallel
P5	A2	1	0	R-0.2	Rocktest	Parallel
P6	A2	6	0	-3,15	Glötzl A	Horizontal
P7	A4	3	0	R-0	Glötzl B	At rock
P8	A4	3	0	R+0	Glötzl B	At rock
P9	A5	3	0	R-0.2	Glötzl A	At rock
P51	A6	3	0	R-0.3	Rocktest	Under blocks
P52	B2	3	0	R-0	Glötzl A	Under blocks
P53	B2	3	0	0,2	Glötzl B	Horizontal
P54	B2	6	0	-2,78	Rocktest	Horizontal
P55	B3	3	0	0,3	Glötzl A	Parallel
P56	B3	3	0	R+0.65	Glötzl A	Parallel
P57	B2	7	0	R+1.1	Rocktest	Parallel
P58	B4	3	0	R-0	Glötzl A	Under blocks
P59	B3	5	0	R-1.1	Glötzl A	Parallel
P60	B4	1	0	R-0.2	Rocktest	Parallel
P61	B6	10	0	R-0	Rocktest	Between blocks
P62	B6	10	0	P	Glötzl B	At wall

Tabell 11-2. Numbering and position of instruments for measuring pore water pressure (U)

Type and number	Section	Layer	X (m)	Y(m)	Fabricate	Remarks
U1	A1	3	0	0,3	Glötzl	
U2	A1	3	0	3,1	Glötzl	
U3	A1	3	0	-2,6	Glötzl	
U4	A1	3	2	0	CT Tube + Druck	Twin tubes
U5	A1	3	-2	0	CT Tube + Druck	Twin tubes
U6	A2	1	0	0,3	Glötzl	
U7	A2	6	0,2	3,15	Glötzl	
U8	A3	1	0,25	-2,8	Glötzl	
U9	A1	5	-0,2	R-1.1	Glötzl	
U10	A2	3	0	0,3	Glötzl	
U11	A2	3	-0,2	R+0.65	Glötzl	
U12	A2	3	1,3	0	CT Tube + Druck	
U13	A2	3	-1,3	0	CT Tube + Druck	
U14	A2	6	-0,15	-0,1	Glötzl	
U15	A2	1	-0,2	R-0.2	Glötzl	
U16	A4	3	0	0,3	Glötzl	
U17	A4	3	0	R-0	Glötzl	
U18	A4	3	0	R+0	Glötzl	
U19	A4	3	R-0	0	Glötzl	
U20	A4	3	R+0	0	Glötzl	
U21	A5	3	0	0,3	Glötzl	
U22	A5	3	1,3	0	CT Tube + Druck	Twin tubes
U23	A5	3	-1,3	0	CT Tube + Druck	Twin Tubes
U24	A5	3	-0,2	R-0.2	Glötzl	
U51	A6	3	-0,2	R-0.3	CT Tube + Druck	Under the Blocks
U52	A6	3	-0,2	-2	CT Tube + Druck	Twin Tubes
U53	B3	1-2	0	R+0.05	CT Tube + Druck	
U54	B2	5	-0,2	R+0.2	CT Tube + Druck	marked as w66
U55	B2	6	-0,2	R+0.65	CT Tube + Druck	marked as w63
U56	B4	3	0	R-0	CT Tube + Druck	
U57	B3	5	-0,2	R-1.1	CT Tube + Druck	
U58	B4	1	-0,2	R-0.2	CT Tube + Druck	
U59	B6	10	0	R-0.05	CT Tube + Druck	
U60	B6	10	0	R-C-C	CT Tube + Druck	Twin Tubes

Tabell 11-3. Numbering and position of instruments for measuring water content (W)

Type and number	Section	Layer	X (m)	Y (m)	Fabricate	Remarks
W1	A1	1	0	0	Wescor Psychrometer	
W2	A1	3	0	0	Wescor Psychrometer	
W3	A1	5	0	0	Wescor Psychrometer	
W4	A2	1	0	0	Wescor Psychrometer	
W5	A2	3	0	0	Wescor Psychrometer	
W6	A2	4	0	0	Wescor Psychrometer	
W7	A3	1	0	0	Wescor Psychrometer	
W8	A3	3	0	0	Wescor Psychrometer	
W9	A3	3	0	2,5	CT Tube	
W10	A3	3	0	R-0.5	Wescor Psychrometer	
W11	A3	3	0	-2	CT Tube	
W12	A3	3	0	R+0.5	Wescor Psychrometer	
W13	A3	3	1,2	0	CT Tube	
W14	A3	3	R-0.3	0	Wescor Psychrometer	
W15	A3	3	-1,2	0	CT Tube	
W16	A3	3	R+0.3	0	Wescor Psychrometer	
W17	A3	4	0	0	Wescor Psychrometer	
W18	A4	1	0	0	Wescor Psychrometer	
W19	A4	3	0	0	Wescor Psychrometer	
W20	A4	4	0	0	Wescor Psychrometer	
W21	A5	1	0	0	Wescor Psychrometer	
W22	A5	3	0	0	Wescor Psychrometer	
W23	A5	3	0	2.5	Wescor Psychrometer	
W24	A5	3	0	-2	Wescor Psychrometer	
W25	A5	4	0	0	Wescor Psychrometer	
W51	A6	1	0	0	Wescor Psychrometer	
W52	A6	3	0	0	CT Res. Probe	
W53	A6	3	0	R-0.4	Ct Tube	
W54	A6	3	0	-2	Ct Tube	
W55	A6	3	-1,3	0	Ct Tube	
W56	A6	3	1,3	0	Ct Tube	
W57	A6	4	0	0	CT Res. Probe	
W58	B2	1	0	0	CT Res. Probe	
W59	B2	3	0	0	CT Res. Probe	
W60	B2	3	0	2,5	Ct Tube	
W61	B2	3	0	R-0.3	Ct Tube	Under the Blocks
W62	B2	3	0	-2	Ct Tube	
W64	B2	3	-1,3	0	Ct Tube	
W65	B2	3	1,3	0	Ct Tube	
W67	B3	1	0	0	Wescor Psychrometer	
W68	B3	3	0	0	CT Res. Probe	

Type and number	Section	Layer	X (m)	Y (m)	Fabricate	Remarks
W69	B3	3	0	R-0.3	CT Tube	Under the Blocks
W70	B3	3	1,3	0	CT Tube	
W71	B3	3	-1,3	0	CT Tube	
W72	B3	4	0	0	CT Res. Probe	
W73	B4	1	0	0	CT Res. Probe	
W74	B4	3	0	0	CT Res. Probe	
W75	B4	3	1,3	0	CT Tube	
W76	B4	3	-1,3	0	CT Tube	
W77	B5	2	0	0	Wescor Psychrometer	
W78	B5	5	0	0	CT Res. Probe	
W79	B5	8	0	2	CT Res. Probe	
W80	B5	8	2	2	Ct Tube	
W81	B5	8	-2	2	Ct Tube	
W82	B5	11	0	2	Ct Tube	
W83	B6	5	0	R-C-C	Wescor Psychrometer	
W84	B6	15	0	R-C-C	Wescor Psychrometer	

Tabell 11-4. Numbering and position of pressure cylinders (C)

Type and number	Section	Layer	X	Y	Fabricate	Remarks
C1	A2	4-5	0	R+0	CT Pressure Cylinder	
C2	A2	2	0	R-0	CT Pressure Cylinder	
C51	B2	4-5	0	R+0	CT Pressure Cylinder	
C52	B4	2	0	R-0	CT Pressure Cylinder	

11.3 Location of measuring sections in the rock

The measuring sections are identified by two letters and 2-3 figures. The letters are U (for pore water pressure) and R (for rock). The numbers are given in the following way:

Short holes in roof: 1-12

Long holes in the roof: 101-107

Short holes in the right wall (seen from the entrance of the drift): 21-32

Long holes in the right wall: 121-129

Short holes in floor: 41-52

Long holes in the floor: 141-147

Short holes in left wall: 61-72

Long holes in the roof: 161-167

Long hole in the end of the drift: 121

Table 11-5 shows the location of the measuring section for each instrument and the corresponding bore hole number. The backfill section in which the borehole starts is also given.

Figures 11-2 and 11-3 show the location of the measuring sections in vertical and horizontal cross sections.

Tabell 11-5. Numbering and positions of instruments for measuring pore water pressure in the rock

Type and number	Location	Measuring section (m)	Borehole number	Sec -tion	Fabricate	Diameter (mm)
UR1	Roof	0.5-1.0	KZ0065I01	A1	Druck	56
UR2	Roof	0.5-1.0	KZ0063I01	A2	Druck	56
UR3	Roof	0.5-1.0	KZ0061I01	A3	Druck	56
UR4	Floor	0.5-1.0	KZ0052G01	A4	Druck	56
UR5	Roof	0.5-1.0	KZ0057I01	A5	Druck	56
UR6	Roof	0.5-1.0	KZ0054I01	B1	Druck	56
UR7	Roof	0.5-1.0	KZ0052I01	B2	Druck	56
UR8	Roof	0.5-1.0	KZ0050I01	B3	Druck	56
UR9	Roof	0.5-1.0	KZ0048I01	B4	Druck	56
UR10	Roof	0.5-1.0	KZ0046I01	B5	Druck	56
UR11	Roof	0.5-1.0	KZ0043I01	B5	Druck	56
UR12	Roof	0.5-1.0	KZ0041I01	B5	Druck	56
UR21	Right wall	0.5-1.0	KZ0066B01	O	Druck	56
UR22	Right wall	0.5-1.0	KZ0064B01	O	Druck	56
UR23	Right wall	0.5-1.0	KZ0061B01	A1	Druck	56
UR24	Floor	0.5-1.0	KZ0057B01	A2	Druck	56
UR25	Right wall	0.5-1.0	KZ0057B01	A3	Druck	56
UR26	Right wall	0.5-1.0	KZ0055B01	A4	Druck	56
UR27	Right wall	0.5-1.0	KZ0053B01	A5	Druck	56
UR28	Right wall	0.5-1.0	KZ0050B01	B1	Druck	56
UR29	Right wall	0.5-1.0	KZ0048B01	B2	Druck	56
UR30	Right wall	0.5-1.0	KZ0046B01	B3	Druck	56
UR31	Right wall	0.5-1.0	KZ0044B01	B4	Druck	56
UR32	Right wall	0.5-1.0	KZ0042B01	B5	Druck	56
UR41	Floor	0.5-1.0	KZ0065G01	O	Druck	56
UR42	Floor	0.5-1.0	KZ0063G01	O	Druck	56
UR43	Floor	0.5-1.0	KZ0061G01	O	Druck	56
UR44	Floor	0.5-1.0	KZ0059G01	A1	Druck	56
UR45	Floor	0.5-1.0	KZ0057G01	A2	Druck	56
UR46	Floor	0.5-1.0	KZ0054G01	A3	Druck	56

Type and number	Location	Measuring section (m)	Borehole number	Sec -tion	Fabricate	Diameter (mm)
UR47	Floor	0.5-1.0	KZ0052G01	A4	Druck	56
UR48	Floor	0.5-1.0	KZ0050G01	A5	Druck	56
UR49	Floor	0.5-1.0	KZ0048G01	B1	Druck	56
UR50	Floor	0.5-1.0	KZ0046G01	B2	Druck	56
UR51	Floor	0.5-1.0	KZ0043G01	B3	Druck	56
UR52	Floor	0.5-1.0	KZ0041G01	B4	Druck	56
UR61	Left wall	0.5-1.0	KZ0066A01	O	Druck	56
UR62	Left wall	0.5-1.0	KZ0064A01	O	Druck	56
UR63	Left wall	0.5-1.0	KZ0061A01	A1	Druck	56
UR64	Left wall	0.5-1.0	KZ0059A01	A2	Druck	56
UR65	Left wall	0.5-1.0	KZ0057A01	A3	Druck	56
UR66	Left wall	0.5-1.0	KZ0055A01	A4	Druck	56
UR67	Left wall	0.5-1.0	KZ0053A01	A5	Druck	56
UR68	Left wall	0.5-1.0	KZ0050A01	B1	Druck	56
UR69	Left wall	0.5-1.0	KZ0048A01	B2	Druck	56
UR70	Left wall	0.5-1.0	KZ0046A01	B3	Druck	56
UR71	Left wall	0.5-1.0	KZ0044A01	B4	Druck	56
UR72	Left wall	0.5-1.0	KZ0042A01	B5	Druck	56
UR101	Roof	1.5-2.5	KZ0065I02	A1	Druck	56
UR102	Roof	4.0-5.0	KZ0065I02	A1	Druck	56
UR103	Roof	1.5-2.5	KZ0055I01	A3	Druck	56
UR104	Roof	4.0-25	KZ0055I01	A3	Druck	56
UR106	Roof	1.5-2.5	KZ0041I02	B5	Druck	56
UR107	Roof	4.0-5.0	KZ0041I02	B5	Druck	56
UR122	Right wall	1.5-2.5	KZ0065B02	O	Druck	56
UR123	Right wall	4.0-5.0	KZ0065B02	O	Druck	56
UR124	Right wall	4.0-5.0	KXZSD8HR	A2	Druck	86
UR125	Right wall	8.4-	KXZSD8HR	A2	Druck	86
UR126	Right wall	1.5-2.0	KXZRD7HR	A3	Druck	86
UR127	Right wall	4.0-8.1	KXZRD7HR	A3	Druck	86
UR128	Right wall	1.5-2.5	KZ0041B02	B5	Druck	56
UR129	Right wall	4.0-5.0	KZ0041B02	B5	Druck	56
UR141	Floor	1.5-2.5	KZ0065G02	O	Druck	56
UR142	Floor	4.0-5.0	KZ0065G02	O	Druck	56
UR143	Floor	1.5-2.5	KXZB3	A3	Druck	56
UR144	Floor	5.0-10.0	KXZB3	A3	Druck	56
UR146	Floor	1.5-2.5	KZ0041G02	B4	Druck	56
UR147	Floor	4.0-5.0	KZ0041G02	B4	Druck	56
UR161	Left wall	1.5-2.5	KZ0065A02	O	Druck	56
UR163	Left wall	4.0-25	KXZSD8HL	A2	Druck	86
UR162	Left wall	4.0-5.0	KZ0065A02	O	Druck	56
UR165	Left wall	1.5-2.0	KXZRD7H	A3	Druck	86
UR166	Left wall	2.5-3.0	KXZRD7H	A3	Druck	86
UR167	Left wall	1.5-2.5	KZ0041A02	B5	Druck	56
UR168	Left wall	4.0-5.0	KZ0041A02	B5	Druck	56

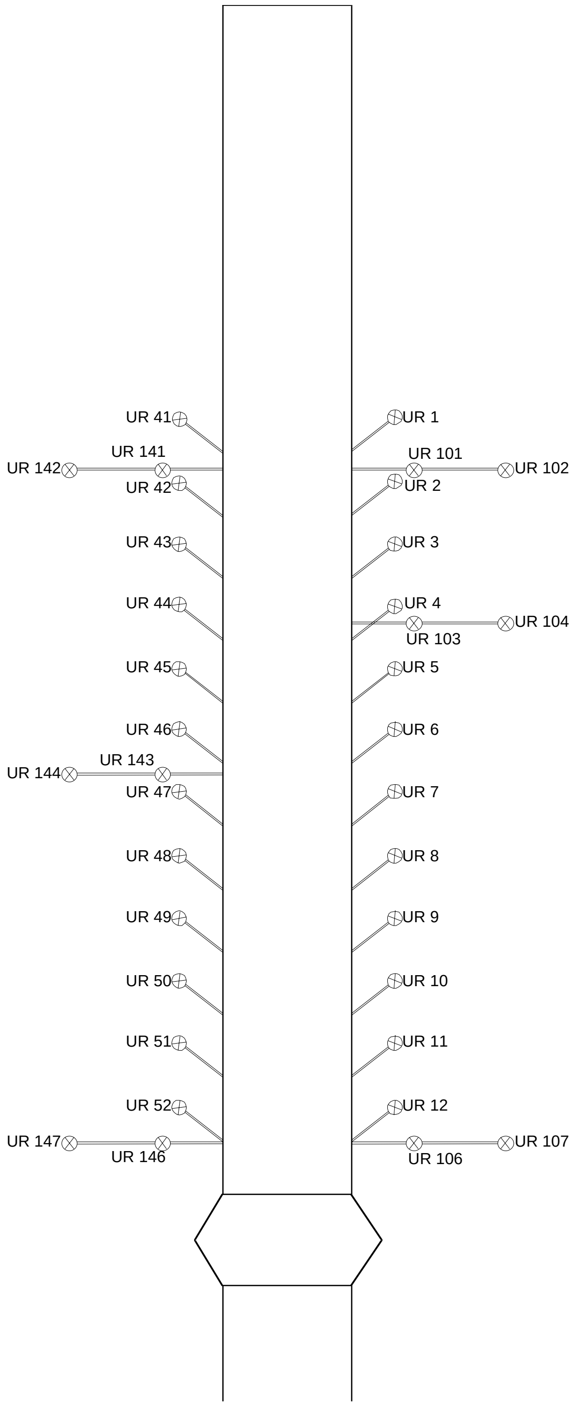


Figure 11-2. *Vertical section describing the position of measuring points in the boreholes of the rock in the floor (left side of figure) and the roof. The long boreholes are not to scale.*

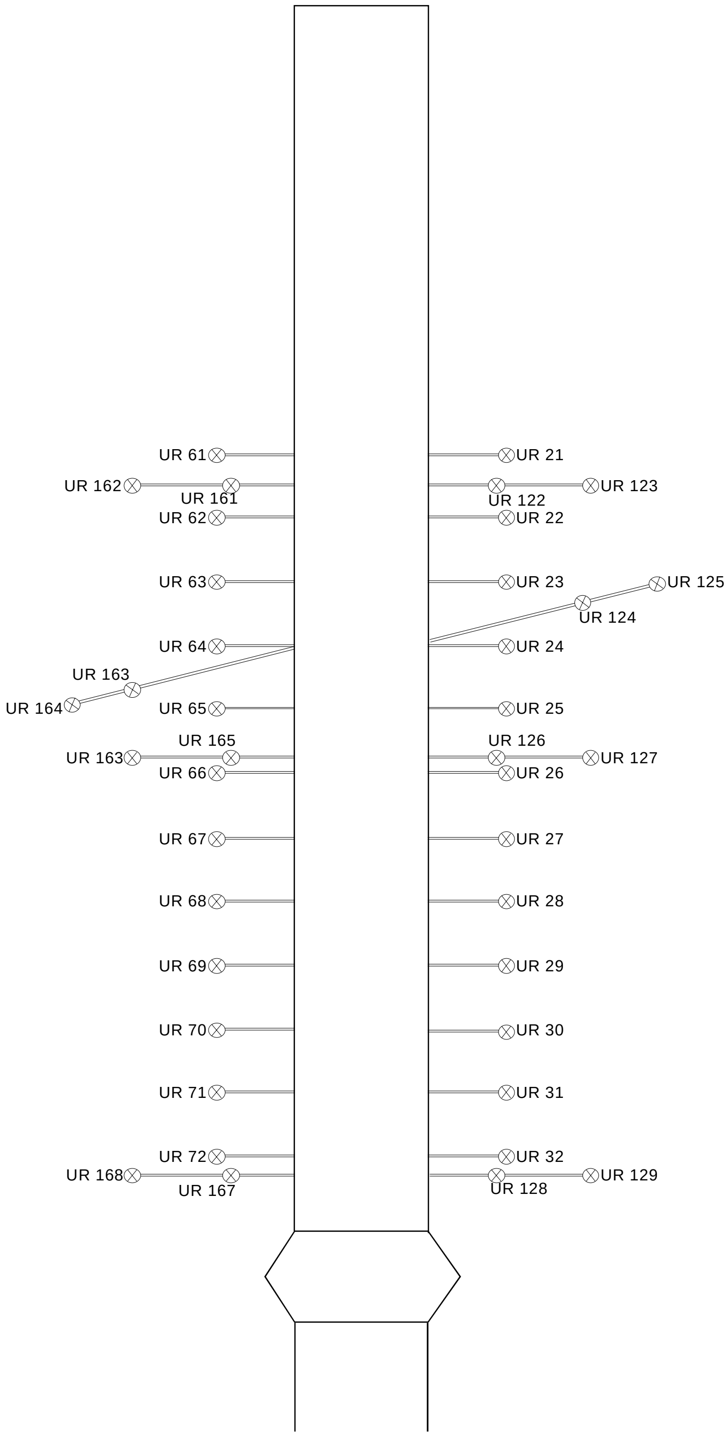


Figure 11-3. *Horizontal section describing the position of measuring points in the boreholes of the rock in the walls. The long boreholes are not to scale.*

11.4 Techniques and instruments for measuring water pressure in the rock

11.4.1 General

The water pressure in rock is measured in core-drilled boreholes sealed with bentonite packers in the following way:

Two Tecalan tubes leading from the measuring sections are connected to Druck pore water pressure cells (model PTX 1400). The porewater pressure cells are placed in the measuring house. Altogether, water pressure is measured in 75 borehole sections (measuring range 0-4 MPa).

Measurements are made in 1-2 sections in each borehole. Most of the holes are only 1 m long with a packer installed so that a section 0.5-0.15 m from the outer end is sealed. Two tubes lead into each measuring section for de-airing purposes. The measuring sections are sealed with packers with bentonite rings kept in place by rubber packers that are bolted to the rock.

A number of different packer principles have been considered. Since it was necessary to prevent leakage for a long period of time, and since maintenance is difficult, the most common types of packers, such as mechanical or hydraulic ones, were excluded.

A mechanical packer combined with bentonite was used in the Stripa tests and the experience from these tests has been taken into account. The advantage of such a packer is the longevity provided by the bentonite. The bentonite has to be mechanically supported in the axial direction.

11.4.2 Short packers

The short packer includes two steel tubes, one for de-airing and one for filling water. The length of the longer of the two steel tubes was adjusted to reach the bottom of each individual borehole. Figure 5-3 is a schematic picture of the short packer.

With an estimated saturated density of about 2000 kg/m^3, the bentonite has a swelling pressure of about 5 MPa and a hydraulic conductivity of about 10^{-13} m/s. For bentonite cylinders compacted at 100 MPa these values are reached after swelling of about 20%. A larger swelling combined with the salinity of the water will decrease the swelling pressure and increase the permeability. When the swelling is larger than 35%, corresponding to a saturated density of 1900kg/m^3, the swelling pressure becomes low enough to jeopardize the function of the packer

It is important that as much as possible of the packer is located in the hole after installation in order to facilitate the subsequent backfilling. A special tool was constructed for holding the packer in the right position during the expansion of the rubber.

The packers are made of acid proof stainless steel to minimise corrosion.

The bentonite rings had to be manufactured with great precision to yield a saturated bentonite with 2000 kg/m^3 density after swelling. This limits the allowable swelling to

20%. The outer diameter of the rings was 54 mm (for 56 mm diameter boreholes) and the inner diameter 28 mm. The height of each ring was 50 mm.

A compaction form in which two pistons could be moved for getting a high homogeneity of the bentonite rings was constructed; a drawing is shown in Appendix 9. MX-80 bentonite with a water ratio of about 10% was compacted under 100 MPa pressure, which gave bentonite rings with a bulk density of about 2080 kg/m³. Subsequent tests showed that the quality, strength and geometry of the bentonite rings were sufficient.

11.4.3 Long packers

The long packers are based on the same principle as the short ones. The Principal design of the packer is shown in Figure 11-4.

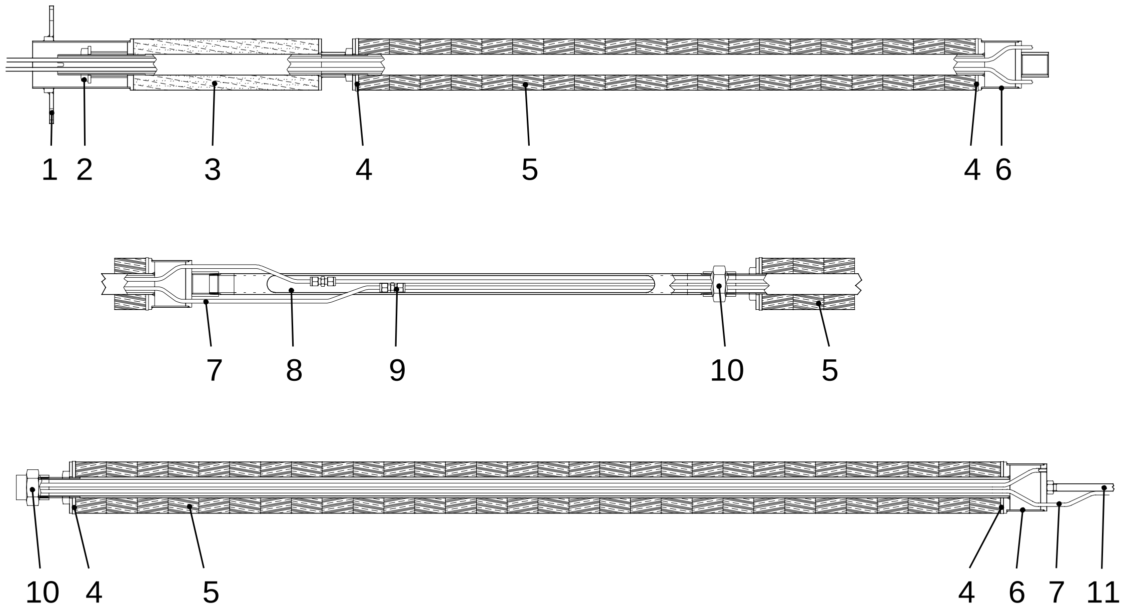


Figure 11-4. Principle design of the long packers

The numbers in the subsequent description refer to Figure 11-4.

The sections to be measured are sealed off with bentonite packer elements (5). From each test section two stainless steel tubes (7) are led to the tunnel; one from the top and one from the bottom of the section. These tubes were initially used for the de-aeration of the sections and water saturation of the bentonite. During the test period they will be used for pressure logging.

The tubes to the inner sections are led through an opening in the central steel pipe (8), into and through the watertight ”distributing chamber” (6). The tube in the innermost section is led to the bottom of the hole in a stainless steel rod (11).

During installation the packers were pushed into the borehole, the parts joined by the connector (10) and the tubes connected by Swagelok couplings (9). When the packers were in the correct position and secured to the rock wall by expansions bolts (1), the polyurethane packer (3) was expanded by tightening the expansion nut (2). This was done in order to maintain the water inside the borehole during the saturation of the bentonite.

In order to minimise water flow along the packer, in the saturation period, soft polyurethane gaskets (4) were inserted at the ends of each packer. When the bentonite expands longitudinally and exerts pressure on these gaskets they seal against the borehole walls.

This type of packer was installed in Ø 86 mm and Ø 56 mm bore holes with lengths from 5 to 25 m.

11.5 Total pressure in the backfill

11.5.1 General

"Total pressure" is the sum of the swelling pressure (or effective stress) and the pore water pressure. It is measured using the following two instrument types:

- Glötzl total pressure cells of the hydraulic type. Two models have been used: E 10/20 KF 50 VA24 model A (Glötzl A) and model F (Glötzl B). The measuring range is 0-5 MPa. Type A is used for measurement in the soil while type B has been fixed to the rock surface with concrete for measuring the pressure between the backfill and the rock. 9 cells of type A and 4 cells of type B have been installed.
- Roctest total pressure cell with vibrating wire transducer model TPC-0 (0-4 MPa). 8 cells of this type have been installed in the backfill.

11.5.2 Glötzl

The principle of a hydraulic total pressure cell is shown in Figure 11-5 and a picture of an installed cell in Figure 11-6. In order to determine the total pressure, the hydraulic pressure in one of the tubes is increased until it equals the pressure in the pressure cell. At that moment the valve in the pressure cell opens resulting in a flow through the cell and in a pressure drop. The applied pressure is monitored continuously and the pressure recorded.

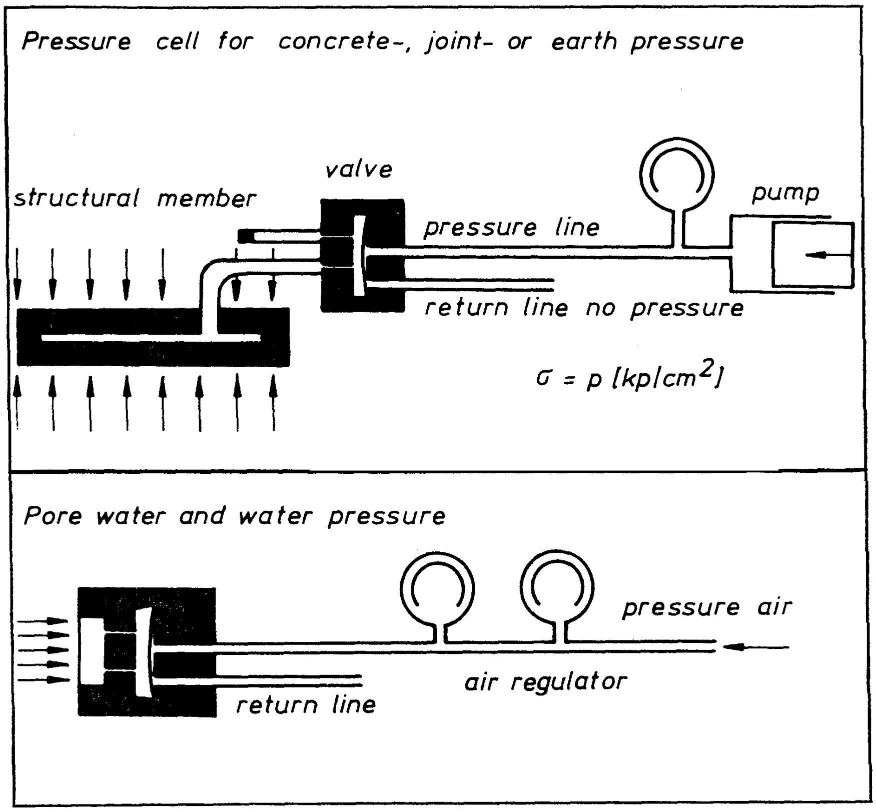


Figure 11-5. *Principle design of GLÖTZL total and pore water pressure cells. (From GLÖTZL's brochure "Earth pressure cells").*

The advantage of this system is that the electrical transducer is placed outside the test volume and may easily be calibrated or exchanged during the test. A piezoelectric or strain gauge type of transducer with a 0-1 V output signal is normally used.

The data acquisition system consists of three parts:

- Three multiplexers type U10H10B18D to which the tubes from the pressure cells are connected. This unit permits a number of pressure cells to be connected to one transducer.
- A transducer, which transforms the pressure to an electrical signal.
- A control unit type MFA6E of the data acquisition system that initialises the measurement, records the pressure during the measurement and stores the measured total pressure.

***Figure 11-6**. Installation of a Glötzl total pressure cell in the backfill.*

11.5.3 Roctest

The vibrating wire pressure cell has a membrane that is deformed when subjected to pressure and a wire connected to it and to the socket of the cell (Figure 11-7). The resonant frequency of the wire is a function of the length, tension and mass per unit length of the wire. To get a reading the wire is excited (forced to vibrate) and the resonant frequency recorded and used for determining the deformation, which gives the pressure on the membrane. There are two main types of transducers:

1. A coil is used for exciting (plucking) the wire. A current is run through the coil making it act as a magnet and thus pulling the wire sideways. All of the frequencies but the resonant frequency of the wire die out in a very short time. The same coil is then used for reading the resonant frequency. This is called the pluck and read method.

2. The wire is excited continuously by a coil and a feed back loop is used for synchronising the current through the coil with the resonant frequency of the vibrating wire. This or a second coil is used for reading the frequency. This method is referred to as the auto resonant method. The Roctest pressure cell uses this method.

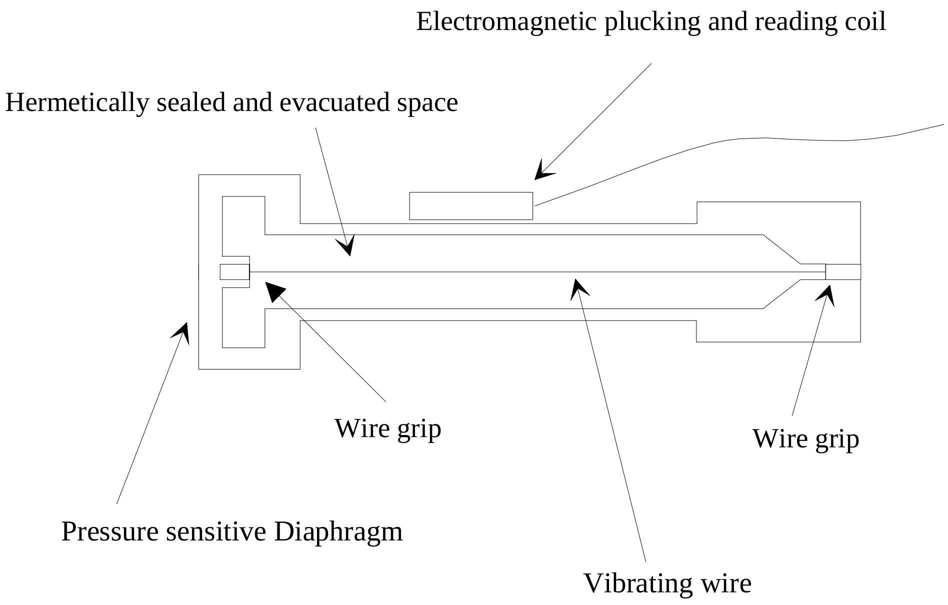


Figure 11-7. *Principle design of a vibrating wire transducer*

11.6 Porewater pressure measurements in the backfill

11.6.1 General

Two measuring principles are used for measuring pore water pressure in the backfill. One of them is similar to the method for measuring water pressure in the rock. The other is a Glötzl hydraulic transducer.

11.6.2 Glötzl

The measuring principle of the Glötzl pore pressure transducers is the same as of the total pressure cells described in chapter 11.5. The only difference is that there is a filter outside the membrane in the pore pressure cell (cf. Figure 11-8).

Figure 11-8. *Glötzl pore water pressure cell.*

11.6.3 Filter tips

The measuring principle for the filter tips is the same as for the water pressure measurements in the rock. A filter tip is connected to a Tecalan tube and the measurement of water pressure is made outside the backfill with the same type of pressure transducer as used for monitoring water pressure in the rock. In a few measuring points, the filter tip is connected to two tubes (see Figure 11-9) so that the filter can be flushed with water and de-aired. These are marked with twin tubes in the tables in chapter 11.2. Such a filter can also be used for tracer testing.

The measuring principle with filter, ferrule connection and pressure transducer was tested in the laboratory. A sample of crushed rock with the largest fraction removed was compacted in a steel cylinder. The density at saturation was 2390 kg/m³ corresponding to 95% Proctor density. The setup is shown in Figure 11-10.

The sample was surrounded by a filter. At first, the sample was saturated from the filter and a constant water pressure was then applied to the filter. Pressure and temperature were recorded, using the data collection system intended for the Backfill and Plug Test (Orchestrator and Data scan devises).

Figure 11-9*. Twin tube filter tip.*

The main purpose of the test was to verify that the measuring principles work and that there should be no problem with the filter, such as clogging. The latter was tested by letting water flow from the filter through the backfill and into the filter tip. The tightness of the couplings and the through-connections was tested by applying a high water pressure in the cell.

A similar test was made with a mixture of bentonite and crushed rock.

The results of the tests showed that the filters worked as expected and no clogging or other problems were encountered.

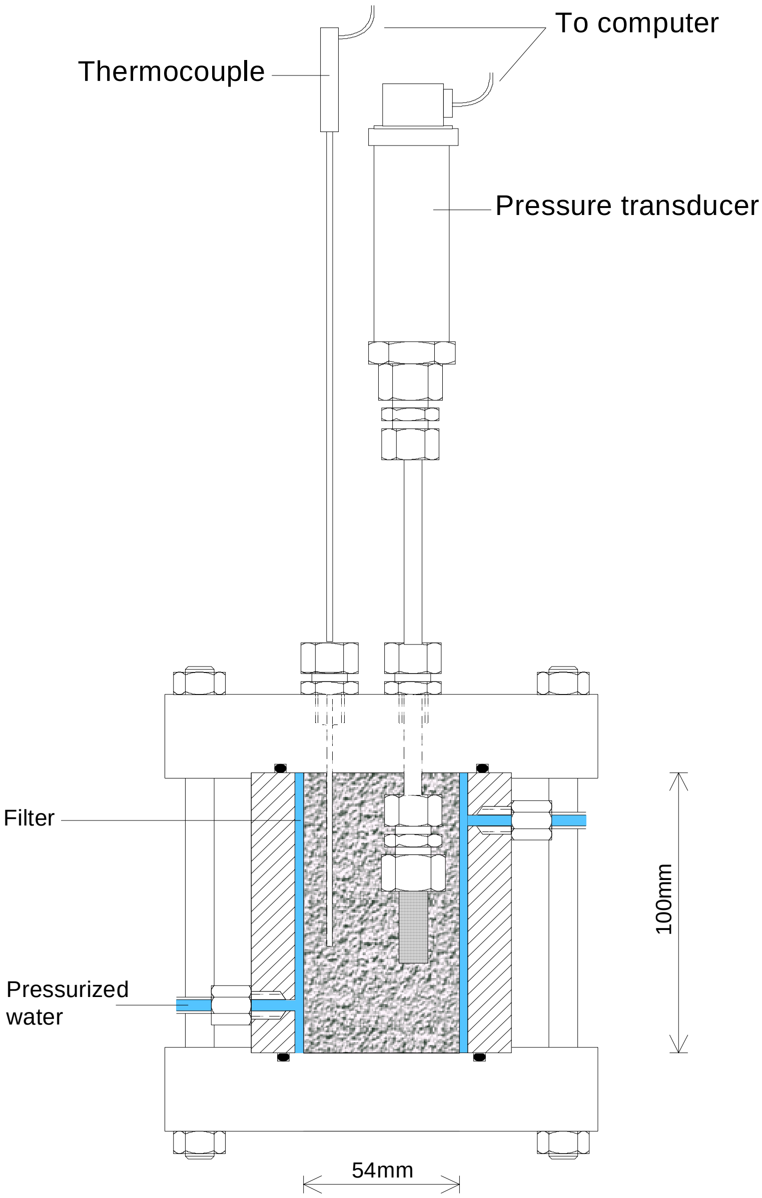


***Figure 11-10**. Schematic drawing of the laboratory test set up*

11.7 Monitoring of the water saturation process

11.7.1 General

The backfill was placed and compacted at a water content that was lower than the one corresponding to complete water saturation. Since flow testing between the permeable mats cannot be made until the backfill is completely water saturated, it was important to monitor the saturation.

A large number of possible methods have been investigated. Some of them are well suited for buffer materials but not for backfill materials. Three devices were chosen for the Backfill and Plug Test:

- Electrical resistivity probes installed in the bentonite-free backfill
- Soil psychrometers installed in the bentonite mixed backfill
- A filter tip mounted on a tube embedded in backfill material installed in both backfill types

The three methods are briefly described.

11.7.2 Electrical resistivity method

The method utilizes the difference in electrical resistance of different soil types (Figure 11-11), porosity and temperature (Figures 11-12 and 11-13) and degree of saturation. The flow of current through soil is mainly due to electrolytic action and therefore depends on the concentration of dissolved salts in the pore water. The mineral particles of a soil are poor electrical conductors. The resistivity of a soil therefore decreases both with increasing water content and with increasing salt concentration of the pore water. The purpose of using the probe in the backfill test was to measure the change in degree of saturation during the water uptake.

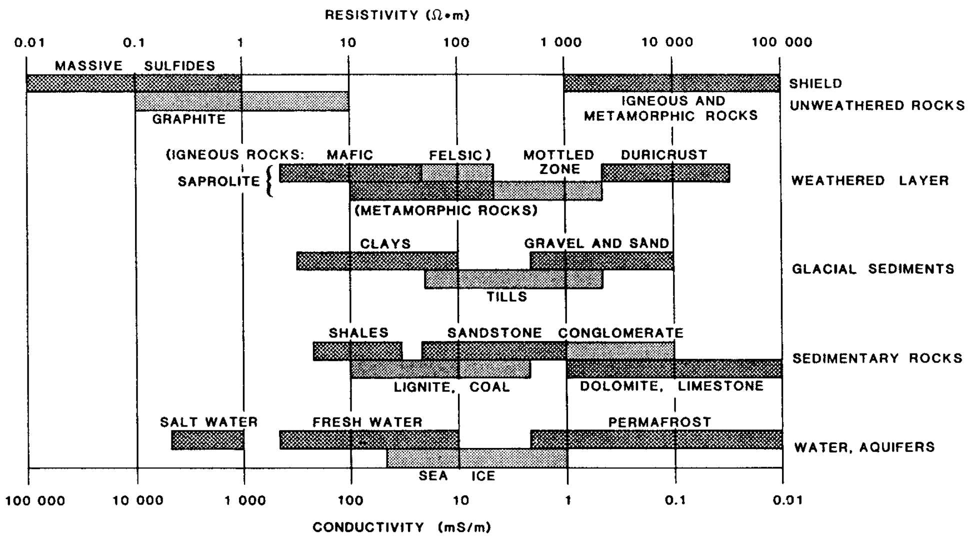


Figure 11-11. *Typical range of electric resistivity of geologic materials /11-1/.*

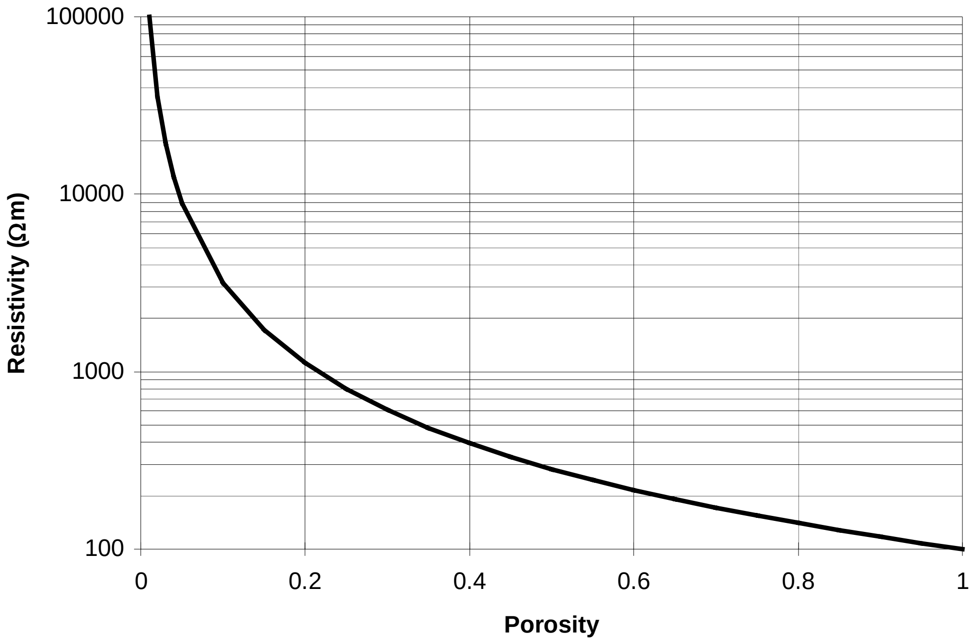


Figure 11-12. *Resistivity as a function of porosity for a clay /11-1/*

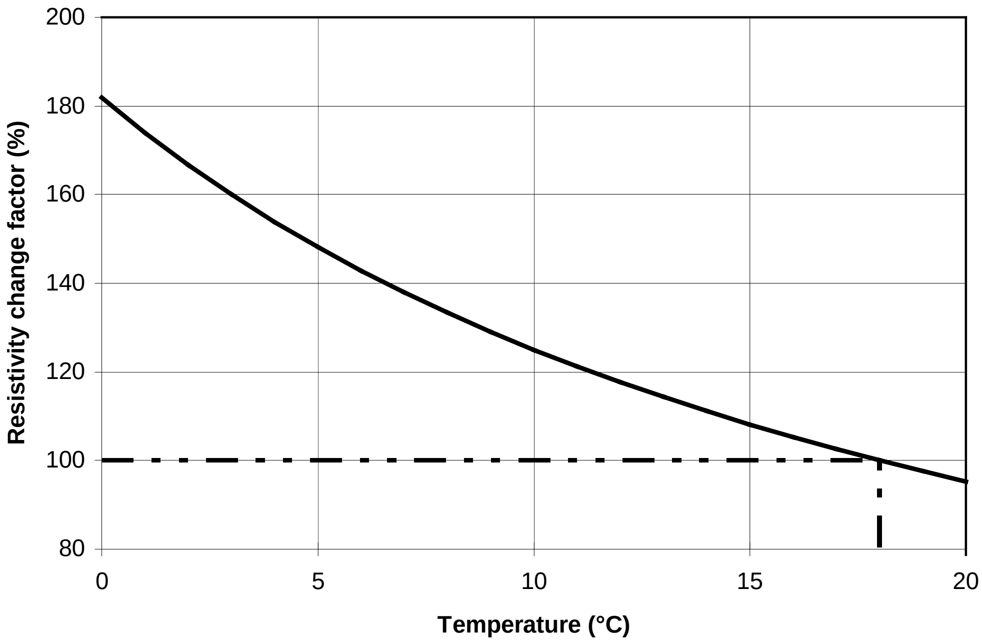


Figure 11-13. *Resistivity divided to resistivity at 18°C as function of temperature for a clay /11-1/.*

Probe construction

A drawing of the probe is shown in Figure11-15. It consists of four electrodes separated from each other by polyurethane. The cable from the probe is inserted in a tecalan tube in order to prevent leakage of water along the cable.

A known current (I) between the two outer electrodes, produce an electrical field in the soil. The potential drop (E) between the two inner electrodes is measured and the current, potential drop and distance between the electrodes are used for calculating the resistivity of the soil.

A prototype of the probe was manufactured and tested in Clay Technology's laboratory. The probe was placed in a box (0.5x0.5x1.0 m^3) filled with crushed rock with different densities and water ratios, and the electric resistivity measured. Soil with the dry densities 1850 kg/m^3 and 2080 kg/m^3, corresponding to 80 and 90% proctor, were used. The water ratio was varied between 5 and 18 %. Ten tests were performed. The purpose of the tests was to find the intensity of the current required for this type of material. Furthermore, preliminary calibration indicating the resistance as function of the water ratio of the backfill was done (see Figure 11-14). The salt content of the water added was 1.2 %.

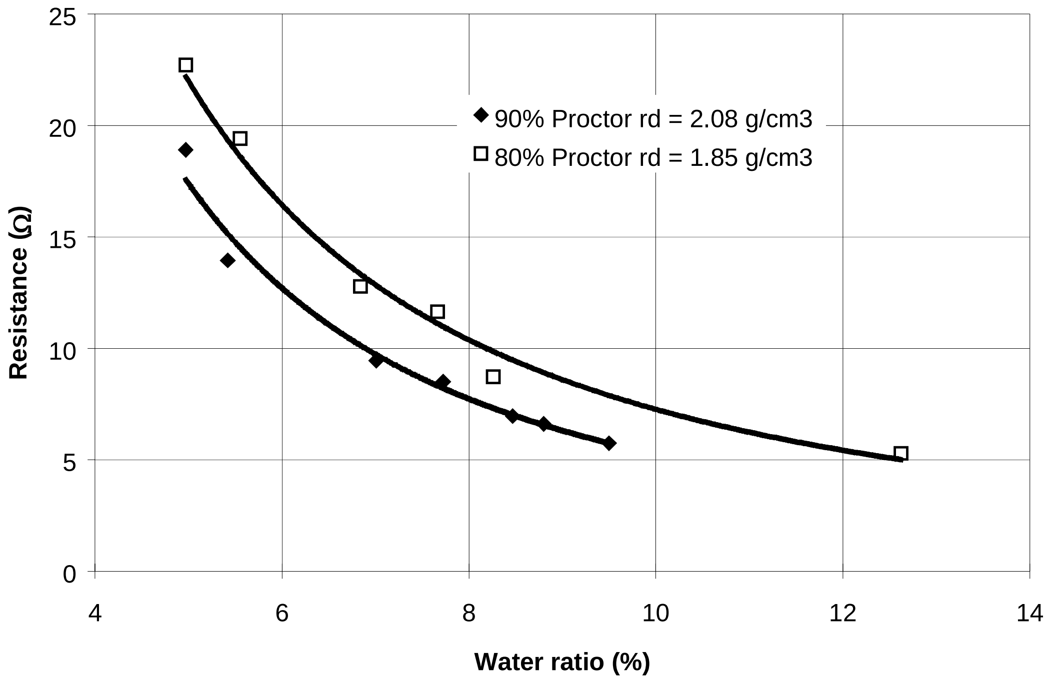


Figure 11-14. *Measured resistance in 30/70 as function of water ratio.*

12 probes were manufactured 10 of them were installed in the Backfill and Plug Test.

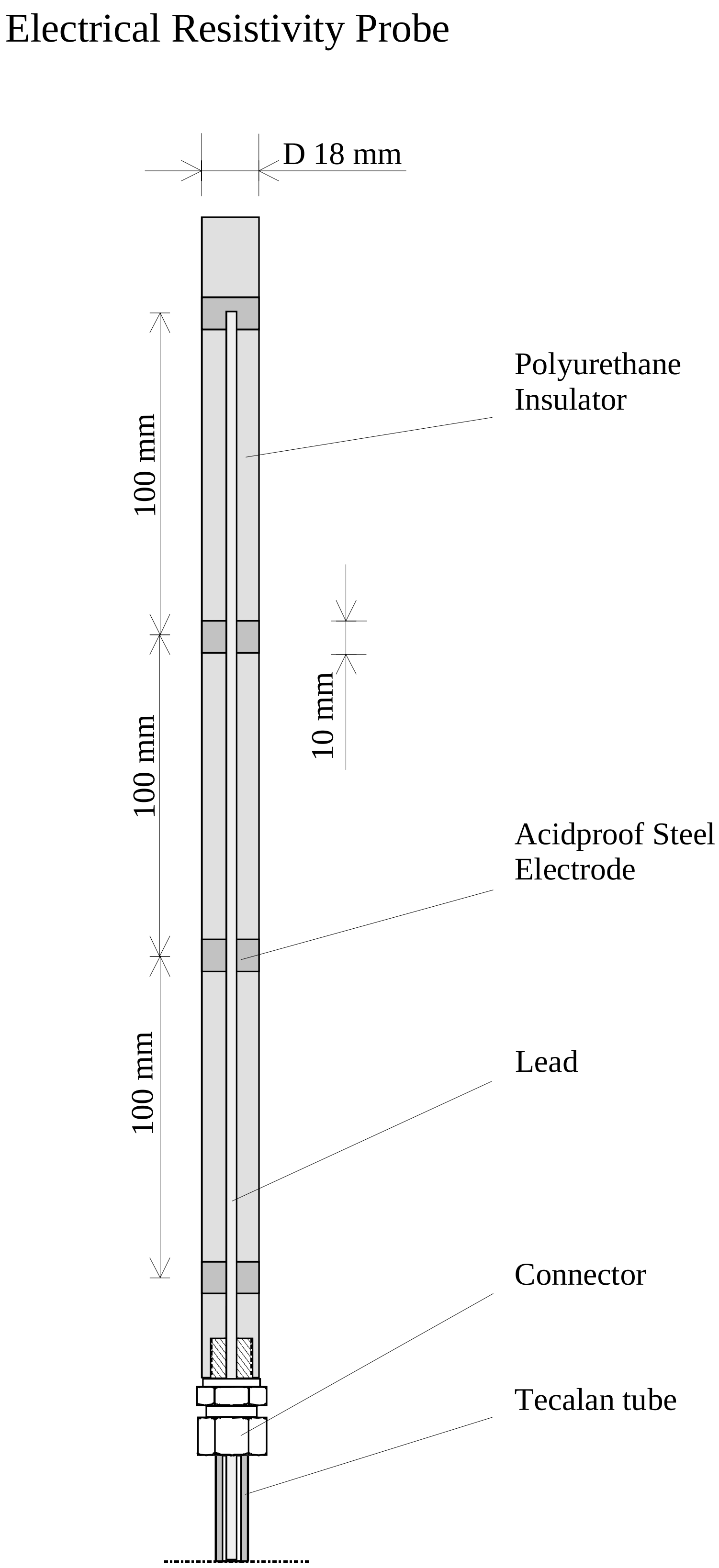


Figure 11-15. *Electrical resistivity probe (design Torleif Dahlin, Lund Institute of Technology.)*

11.7.3 Psychrometer method

A soil psychrometer (Figure 11-16) measures the relative humidity in the pore system. Using thermodynamics the relative humidity can be expressed as suction (or water potential) of a soil according to Eqn 11-1 (Kelvin equation).

$$\psi = \frac{RT}{V_w} ln(\frac{u_v}{u_{v0}}) \tag{11-1}$$

where

$\psi =$ suction of the soil (Pa)

$R =$ universal gas constant (8.3143 J mol^{-1} K^{-1})

$\frac{u_v}{u_{vo}} =$ relative humidity of the pore air

$T =$ absolute Temperature (K)

$V_w =$ molar volume of water (1.8E-5 m^3 mol^{-1})

The wet bulb method has been chosen for the field test at which a thermocouple was cooled below the dew point by means of the Peltier effect, causing droplets of condensed water on the junction surface of the psychrometer. The output temperature from the thermocouple during the cooling is shown in Figure 11-17. Cooling is applied (period *a* to *b*) and then interrupted. The subsequent temperature increase causes water to evaporate, which causes further temperature decrease due to the evaporative cooling (period *b* to *c*). The wet bulb temperature decrease persists until complete evaporation takes place (period *c* to *d*). Finally, the thermocouple signal that ambient temperature prevails (period *d* to *e*).

The measured curve shown in Figure 11-17 thus corresponds to the temperature and period *c* to *d* (the wet bulb) is used for calculating the relative humidity, which in turn is used to calculate the suction according to Eqn 11-1.

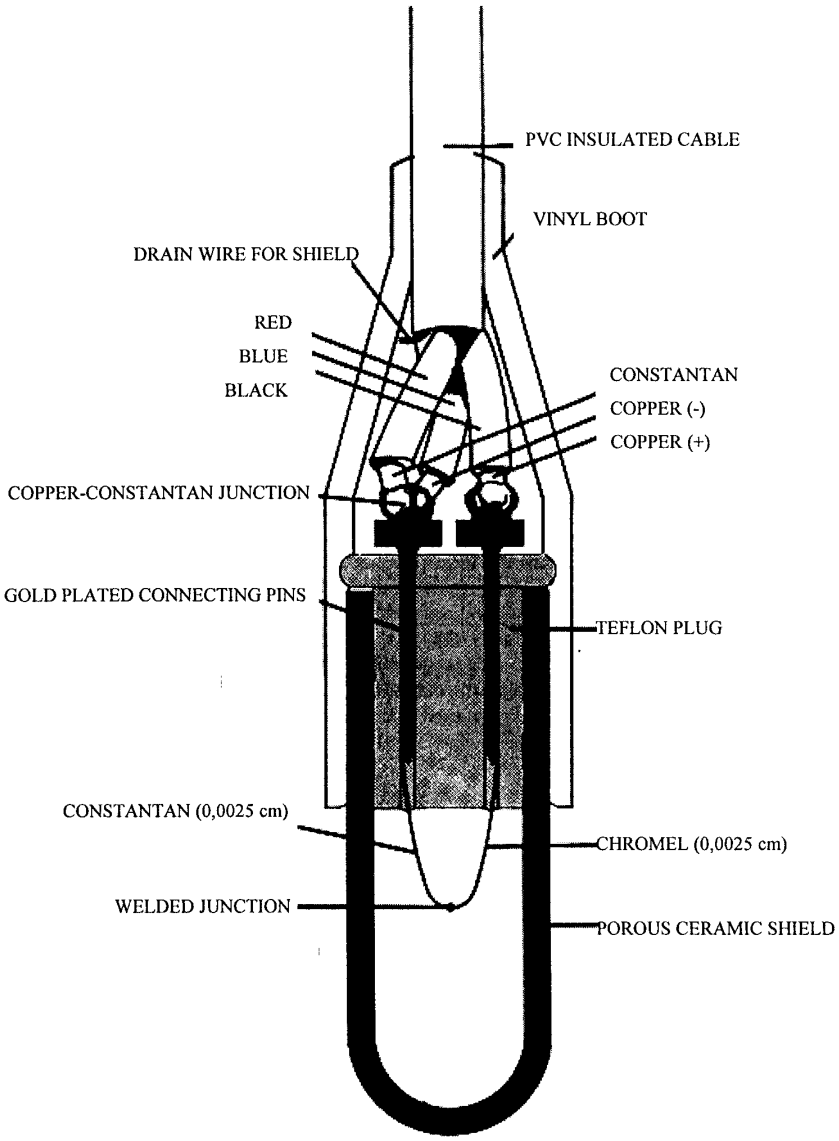


Figure 11-16. *Soil psychrometer (taken from /11-2/)*

<u>Laboratory tests</u>

The following laboratory tests of psychrometers were made:

- Calibration of psychrometers in moisture chambers
- Establishment of the relation between suction potential and water ratio and density of the backfill material in moisture chambers
- Test of psychrometers in different backfill materials

Samples of mixtures of crushed rock and bentonite were placed in chambers with constant relative humidity and constant temperature. Samples with compacted material (high density) as well as non-compacted material (low density) were prepared and allowed to take up water until they reached equilibrium with the surrounding relative humidity. The water ratio was then measured and the clay water ratio calculated.

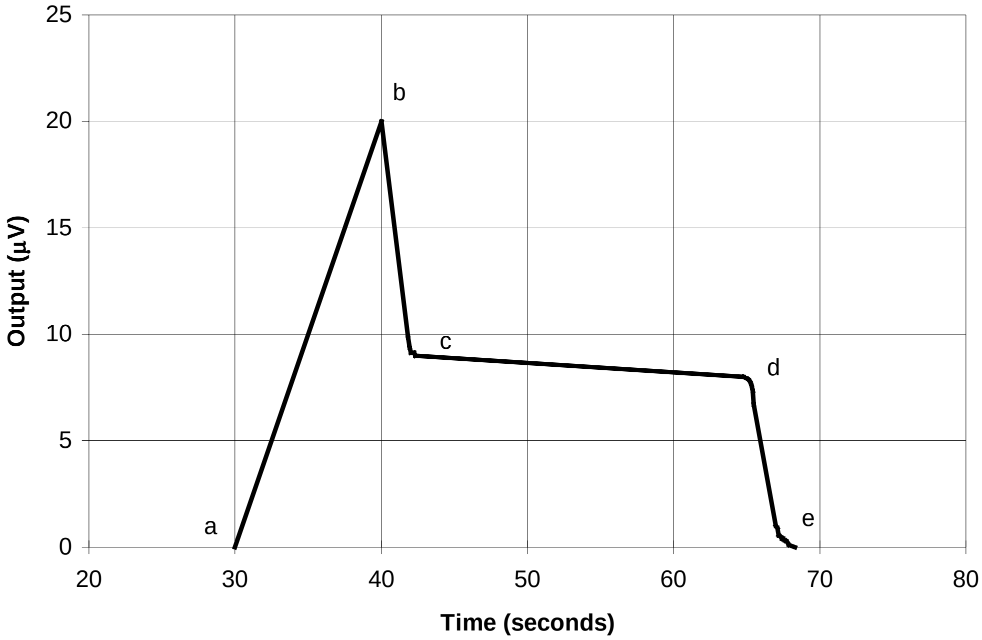


Figure 11-17. *Output from a psychrometer measurement (wet bulb method)*

The clay water ratio (w_c) can be calculated by Eqn. 11-2

$$w_C = \frac{[w-(1-k)\times w_b]}{k} \tag{11-2}$$

where

w	= water ratio of the mixture
w_b	= water ratio of the ballast material
m_{sc}	= dry mass of clay
m_{st}	= total dry mass of clay

k is defined as

$$k = \frac{m_{sc}}{m_{st}} \tag{11-3}$$

Results from such tests are shown in Figure 11-18. The figure shows that the suction is a function of clay water ratio of the mixtures. It is independent of the density of the samples and of the clay content. Measuring the relative humidity in a backfill is thus a way to determine the water ratio.

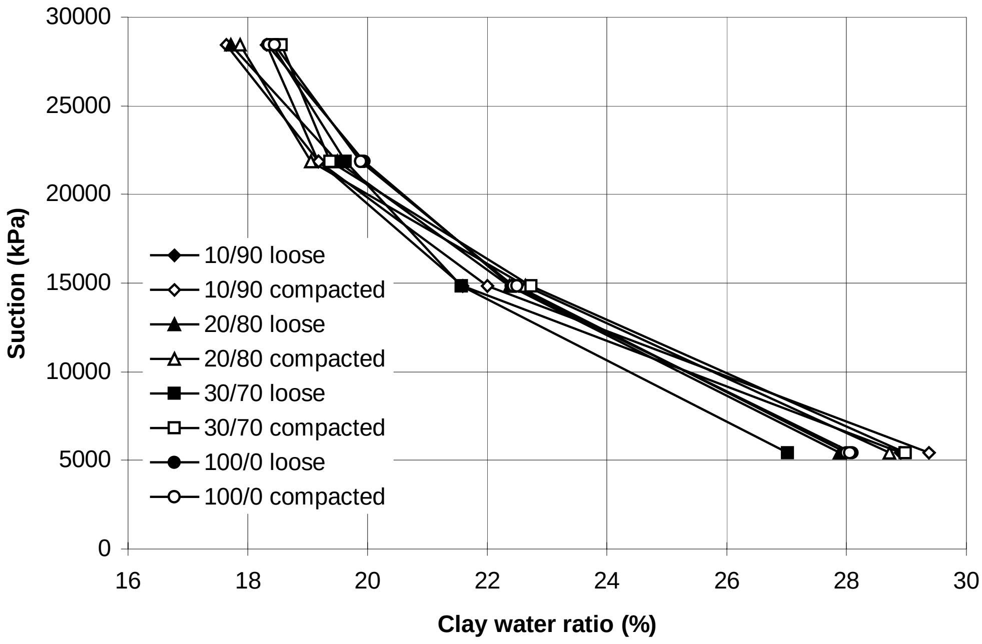


Figure 11-18. *Measured suction for different mixtures of crushed rock and bentonite as functions of clay water ratio.*

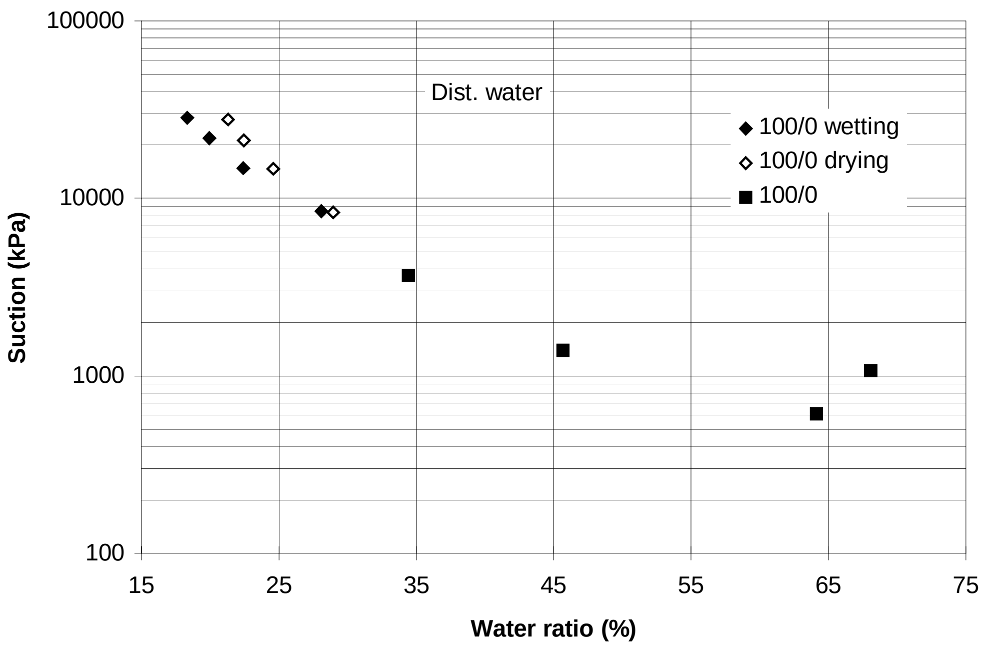


Figure 11-19. *Suction of pure MX-80 as function of water ratio. The tests were made with Äspö water and distilled water.*

Figure 11-19 shows results from tests performed with salt water and distilled water on pure bentonite. The suction at high water ratio (> 33%) was measured using a soil psychrometer, while the other results were derived from moisture chamber tests. The figure indicates that the psychrometer can be used for clay water ratio measurements between 35 and 70%. A mixture of 30% bentonite and 70% crushed rock with a clay water ratio between 35-70% corresponds to a 11-21% water ratio of the mixture. The psychrometer is thus useful for measuring water ratios between 11% and 21% in this material, which corresponds to the expected water ratios in the field test.
Figure 11-20 shows some results from direct measurements of suction with a psychrometer in mixtures of 30/70 when either distilled water or water with a salt content of 1.2 % ("Äspö water") has been added. The influence of the salt content of the Äspö water is obvious. The difference is caused by the additional suction potential of the salt water. Measurements in other mixtures have also been made and are compiled as suction versus clay water ratio in Figure 11-21.

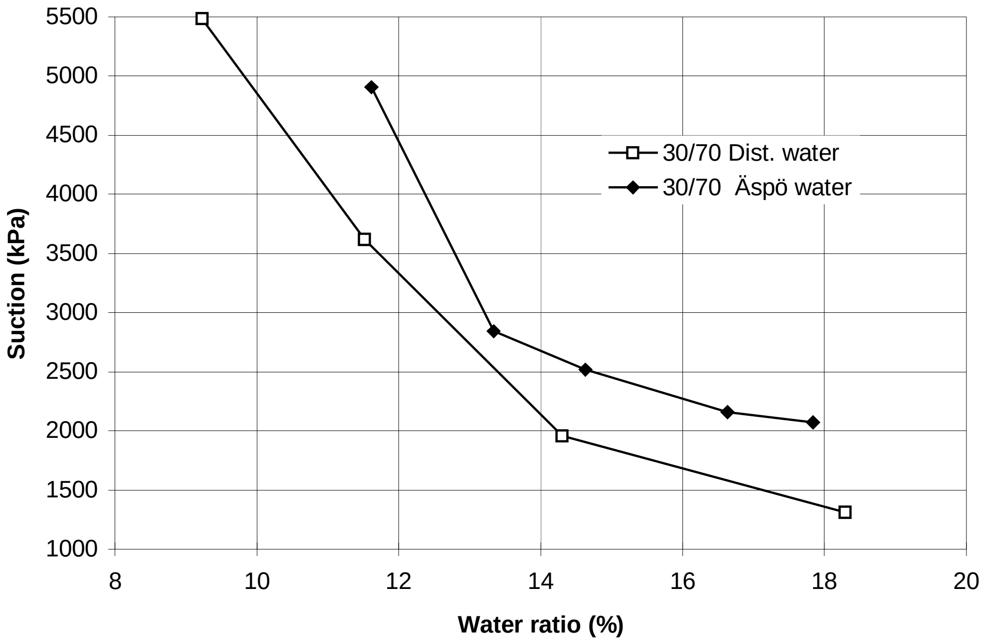
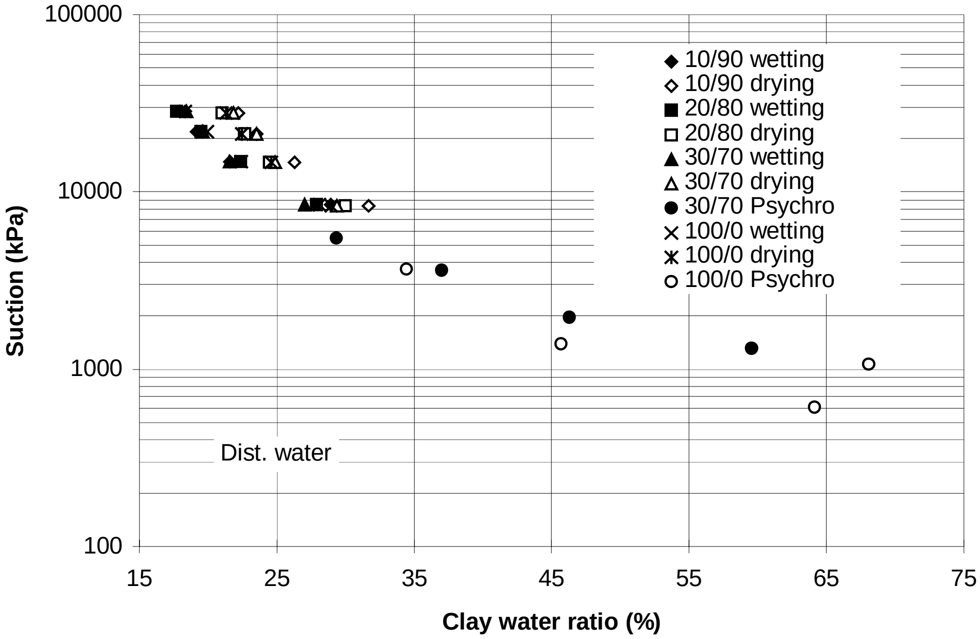


Figure 11-20. *Direct measurements of suction with psychrometers in mixtures of 30/70.*



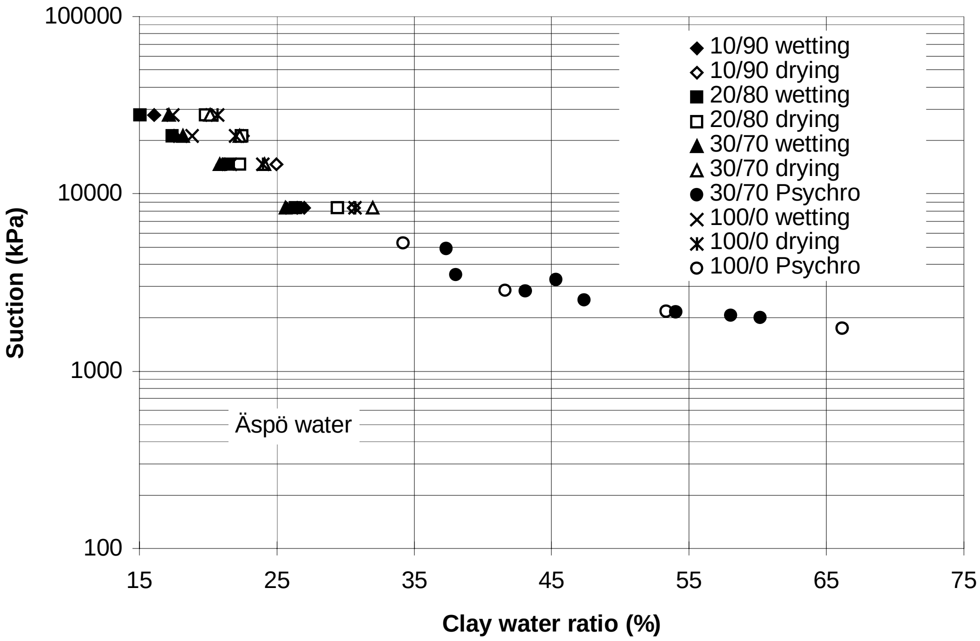


***Figure 11-21**. Compilation of suction measurements versus clay water ratio with either distilled water added (upper) or Äspö water.*

Psychrometer measurements were also made in backfill material without bentonite. The results (Figure 11-22) showed that the measured suction decreased with increased water ratio when distilled water was used, while for Äspö water it was the opposite. It seems that the suction of the mixture with Äspö water approaches the value 1200 kPa, which corresponds to the potential of the water itself. It was concluded that the psychrometer is not useful for the bentonite-free backfill.

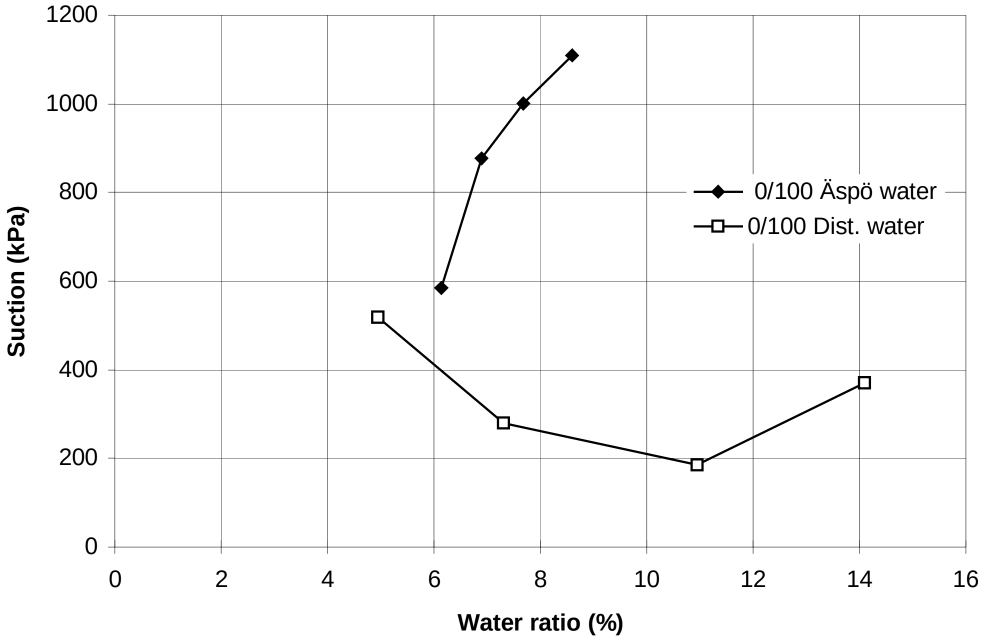


Figure 11-22. *Direct measurements of suction with psychrometers in pure crushed rock*

<u>Probe design</u>

The psychrometer had to be protected from mechanical damage during the compaction of the backfill. It was also important to prevent water leakage through the psychrometer cables. The psychrometers were therefore enclosed in probes consisting of a chamber, a tecalan tube with a cable, and a watertight coupling (Figure 11-23). A steel filter at the tip of the probe also protected the psychrometer from direct contact with the backfill. A photo of an installed Psychrometer is shown in Figure 11-24.

Probes for 30 psychrometers were manufactured. The probes, couplings and filters were made of acid-proof stainless steel.

All psychrometers were calibrated before installation in the backfill. The calibration was made in chambers with constant temperature at four different levels of relative humidity established with different salt solutions as described earlier in this chapter.

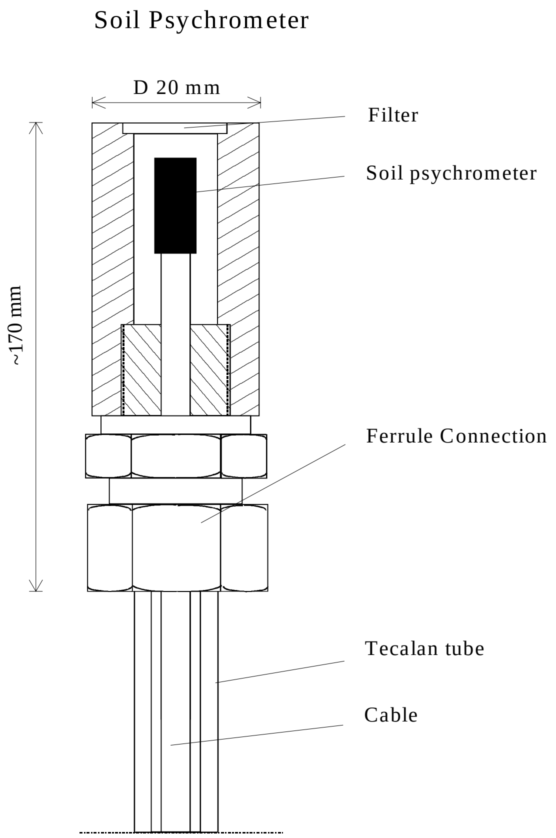


Figure 11-23. *A soil psychrometer built in into a probe.*

Figure 11-24. *Photo of a Psychrometer installed in the backfill*

11.8 Measurement of temperature

Since no heat is generated in the experiment, temperature sensors have only been installed in two points for general information on their performance. Thermocouples of type K from Heraeus Electro-Nite AB were chosen. Temperature can also be measured by the psychrometers and by the devices for measuring hydraulic conductivity installed by ENRESA.

11.9 In situ measurement of mechanical properties

11.9.1 General

An important role of the backfill is to minimise upward swelling of the buffer in the deposition holes and to support the roof. A way of checking the mechanical function of the backfill is to determine the strain when applying a load in situ. The controlling parameters are the swelling pressure and the bulk modulus of the backfill, but the hydraulic conductivity and the swelling pressure must also be considered at evaluation of such tests. These relationships have been investigated in laboratory tests /1-1/ and the results from these tests have been used for designing the field experiment. Pressure cylinders with 420 mm diameter were installed both in the tunnel floor and roof. Once the backfill is saturated and all other tests have been completed, the pressure cylinders will be activated. Pressure will be applied stepwise on the backfill and the displacement of the cylinders measured.

The tests with the pressure cylinder in the floor will simulate how the swelling pressure from the buffer mass in a canister hole affects the backfill. The swelling pressure from the buffer mass is expected to be about 5 MPa.

The results from laboratory tests have been used for calculating the maximum expansion of the pressure cylinders. A cylinder with a 500 mm diameter, a maximum effective pressure of 5 MPa and a tunnel height of 5.5 m yield a maximum expected settlement of the backfill of about 3 cm for the crushed rock and about 15 cm for the 30/70 mixture. The calculations are based on that the compression modulus of 30/70 is 20 MPa and that the compression modulus of 0/100 is 100 MPa /1-1/.

The time for complete development of the settlement for each load step can be calculated using Terzaghi's theory of consolidation. Since permeable and permeable mat sections are placed in the backfill every second meter, the perpendicular distance between two draining layers above the pressure cylinders is 1.5 m. The calculated time for developing 95% of the settlement is 340 days for the 30/70 mixture close to the floor if the hydraulic conductivity evaluated from oedometer tests is used and less than a day for the crushed rock. The estimation of the consolidation rates are based on that the hydraulic conductivity of 30/70 material is 10^{-11} m/s and that the hydraulic conductivity of crushed rock is 10^{-7} m/s. The consolidation will be faster close to the roof.

The demands on the pressure cylinder can be summarized as follows:

- The steel in the cylinder should withstand salt water for at least 5 years
- The maximum net pressure on the cylinder will be 4 MPa (water pressure)

- The maximum internal pressure will be 10 MPa (water pressure plus applied effective pressure)
- The cylinder should withstand an excentric load (50 kNm)
- The maximum displacement of the cylinder will be 10 - 15 cm

For each load step a constant pressure will be applied for a long time. During that period the pressure and the displacement will be measured continuously.

11.9.2 Design

The device consists of two concentrically placed cylinders with 420 mm diameters and 400 mm height (Figure 11-25). The chamber between the cylinders is filled with hydraulic oil, which can be pressurized stepwise trough a pipe with a pump placed outside the test volume. The pressure in the liquid will be measured with a pressure transducer (0 - 10 MPa) and the displacement of the cell can be measured with a potentiometer placed in the center of the cylinders. The device was made of steel. It was coated with zinc and also painted in order to prevent corrosion.

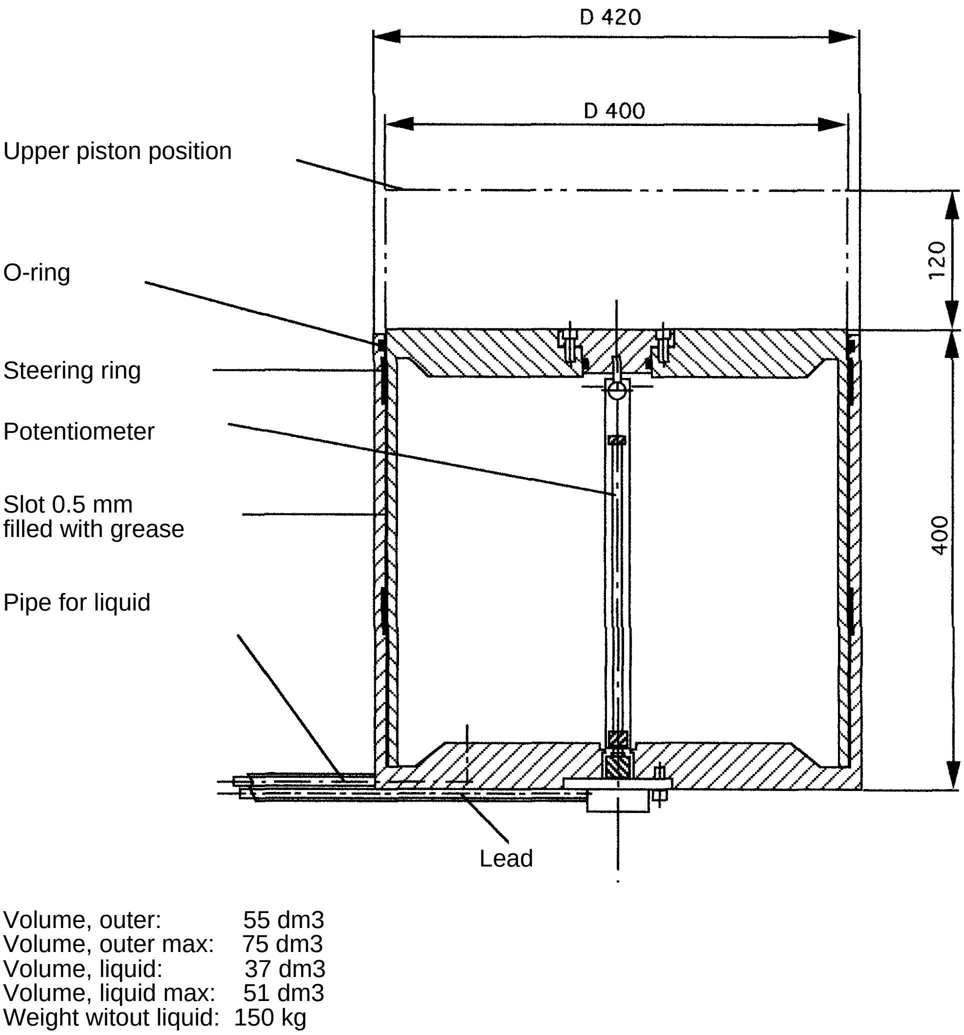


Figure 11-25. *Pressure cylinder*

The cylinder was tested in Clay Technology's laboratory. The tests included calibration of the gauges (deformation and pressure) and test of the cylinder with an eccentric load.

The installation is described in chapter 4.7.

11.10 Local permeability measuring system

A description of the Local Permeability Measuring System and the installation is presented in Appendix 8.

11.11 Choice of material for transducers and couplings

11.11.1 Environment

The environment in the backfill where the instruments were placed is highly corrosive which is due to the high concentration of chloride ions. The instruments need to give reliable readings for 5 years, which is the longest duration foreseen for the test. The temperature is expected to be about 10°C and not exceed 20°C at any time.

11.11.2 Materials

According to the Swedish Institute of Corrosion (KI), steel with a high percentage of molybdenum (Mo) is very corrosion-resistant in an environment with a high concentration of chlorides. KI proposed to use *Avesta 254 smo* steel with 6% Mo. Unfortunately this material is difficult to acquire and to process, since it is very hard.

Different types of acid proof stainless steel (ASTM 316, SS 2343, SS 2353 or SS 2367) are acceptable materials with respect to corrosion. Experience from the Äspö HRL and laboratory tests shows that acid proof stainless steel has endured the environment without any major corrosion. Swagelok, a manufacturer of couplings, claim that their couplings made of acid proof steel are resistant to corrosion in seawater at temperatures up to 70°C.

Other possible materials are different kinds of plastics and ceramics. These materials are usually not strong enough to fulfil the required mechanical properties. Other metals that would suffice are titanium and different alloy types such as Inconell and Hastaloy, but they are more expensive and are probably not required at low temperatures.

According to KI the use of titanium in combination with acid proof stainless steel would increase the rate of corrosion of the acid proof steel. The conclusion was that acid proof stainless steel with a minimum Mo content of 2% could be used for all transducers in the Backfill and Plug Test.

11.12 Data acquisition

11.12.1 General

All the cables and tubes from the test area were led to the data collection house (see the drawing in Figure 11-26 and the photo in Figure 11-27). The cables and enter the measuring house on the backside. The Orchestrator, which is the general data acquisition system for the Äspö HRL, has been used for collecting most data.

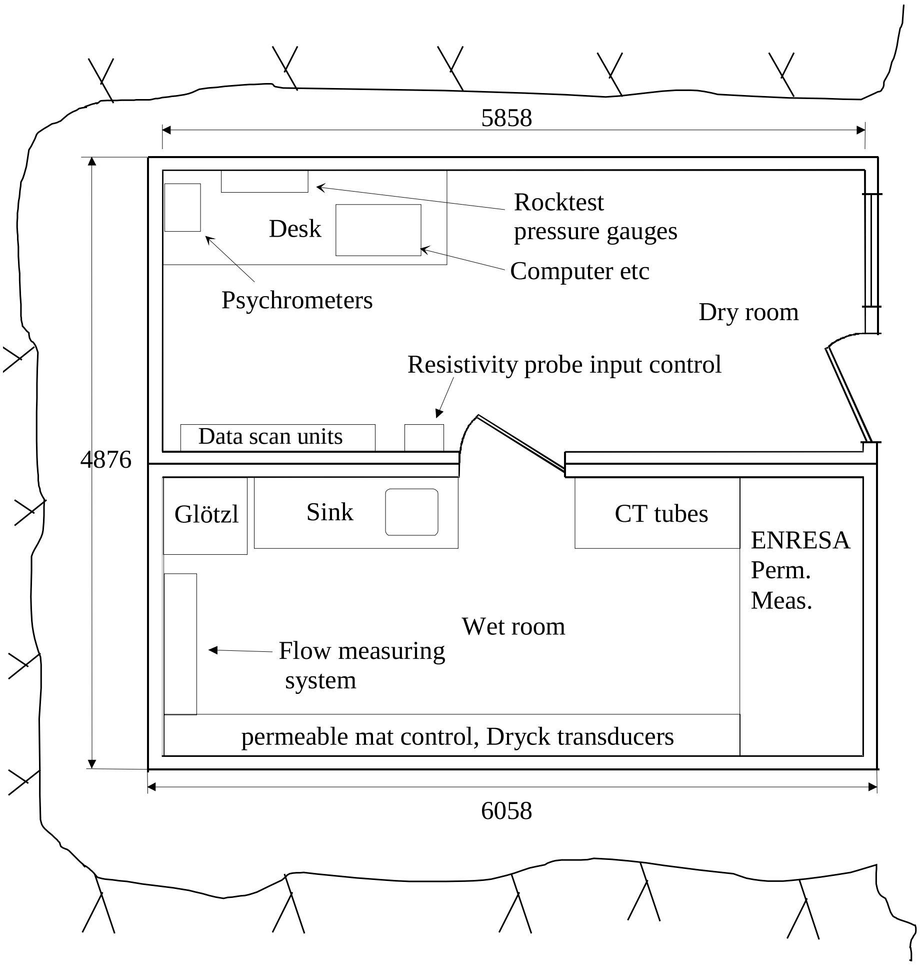


Figure 11-26. Layout of the data acquisition house.

Figure 11-27. *Photo of the data acquisition house*

11.12.2 Druck transducers

The Druck transducers are used for measuring water pressure in the bore holes and in the backfill. The Druck transducers have 4-20 mA output and were connected to Orchestrator via datascan units. The layout of the mounting of the Druck transducers and the layout of the permeable mat control is shown in Appendix 11.

11.12.3 Glötzl pressure cells

The Glötzl pressure cells measure total pressure and pore water pressure in the backfill. The Glötzl Data Acquisition System (MFA 6 E) is operated from the Data collection computer. A program written in Visual Basic sends commands to the MFA 6 E initiating measurements and extracting results. The data are saved in the excel format and transferred to an Access database.

11.12.4 Roctest pressure cells

The Roctest pressure cell measures total pressure in the backfill. The Roctest cells are controlled by a Campbell CR10X logger. This normally controls the entire measuring sequence, from the sending of different frequencies to the pressure cells to the analyses of the returning signal and storing of results. In this case an Orchestrator module which allows Orchestrator to control the CR10 X has been developed. Hence, the data from the Roctest pressure cells are collected directly in the Orchestrator system.

11.12.5 Psychrometers

The measuring principle chosen, the wet bulb method, for the psychrometers was described in chapter 11.7.4. The logger used, a Campbell CR7, was programmed to carry out the entire measuring sequence and to store all the output data. Based on experience from the FEBEX test no automatic evaluation of the output has been made. Instead all output data have been stored and evaluated manually. The data stored in the CR7 has been transferred to the main computer once a week.

11.12.6 Resistivity probes

The data acquisition system for the resistivity probes consists of two independent parts. One provides the current to the outer electrodes that creates the difference in electric potential over the length of the probe. The other is the Orchestrator system, which measures the difference in electric potential over the inner electrodes. The Orchestrator system is set to log the event only. Orchestrator scans the difference in potential continuously but does not store any data until there is a certain pre-determined difference between two consecutive values. Also these data are evaluated manually.

11.12.7 Temperature probes

The thermocouples were connected to Orchestrator via data scan units.

11.12.8 Local permeability probes

AITEMIN is responsible for data collection and evaluation. The system is described in Appendix 8.

11.13 System for filling the permeable mats with water and controlling the water pressure

11.13.1 General

The system was designed to fill the permeable mats with water, saturate the backfill and later perform flow tests. Therefore it has to be flexible and allow considerable variation in application of pressures and monitoring of flow. The flow to the mats is measured during the saturation and the flow in and out of the mats during the flow testing.

11.13.2 Overview

According to Figure 11-28 the system consists of four parts:

1) Three storage tanks where water is collected from the inner part of the backfill drift.

2) Equipment for pressurising the water. Three pressure vessels for applying pressures up to 600 kPa and two pumps for applying high pressures and high flow.

3) Four units for measuring and controlling flow and pressure

4) Control panel for the permeable mats

The water from the storage tanks is led to the pressure vessels and the pumps where the water is pressurised. Water is led from the pressure vessels and pumps to one of the four flow meters and further (the set-up of the flow meters is shown in Appendix 10) to the control panel for the permeable mats. Via the control panel it is possible to select desired flow and pressure to the permeable mats.

During the flow testing the set up will be changed so that is possible also to measure the flow from mats and sections, as the pressure is kept constant.

11.13.3 Storage tanks

Three 1500 l storage water tanks have been installed (Figure 11-30). A centrifugal pump CPE3-060 is used for transporting water between the tanks and to the pressure vessels.

The locations of the storage tanks and the pressure vessels are shown in Figure 11-29.

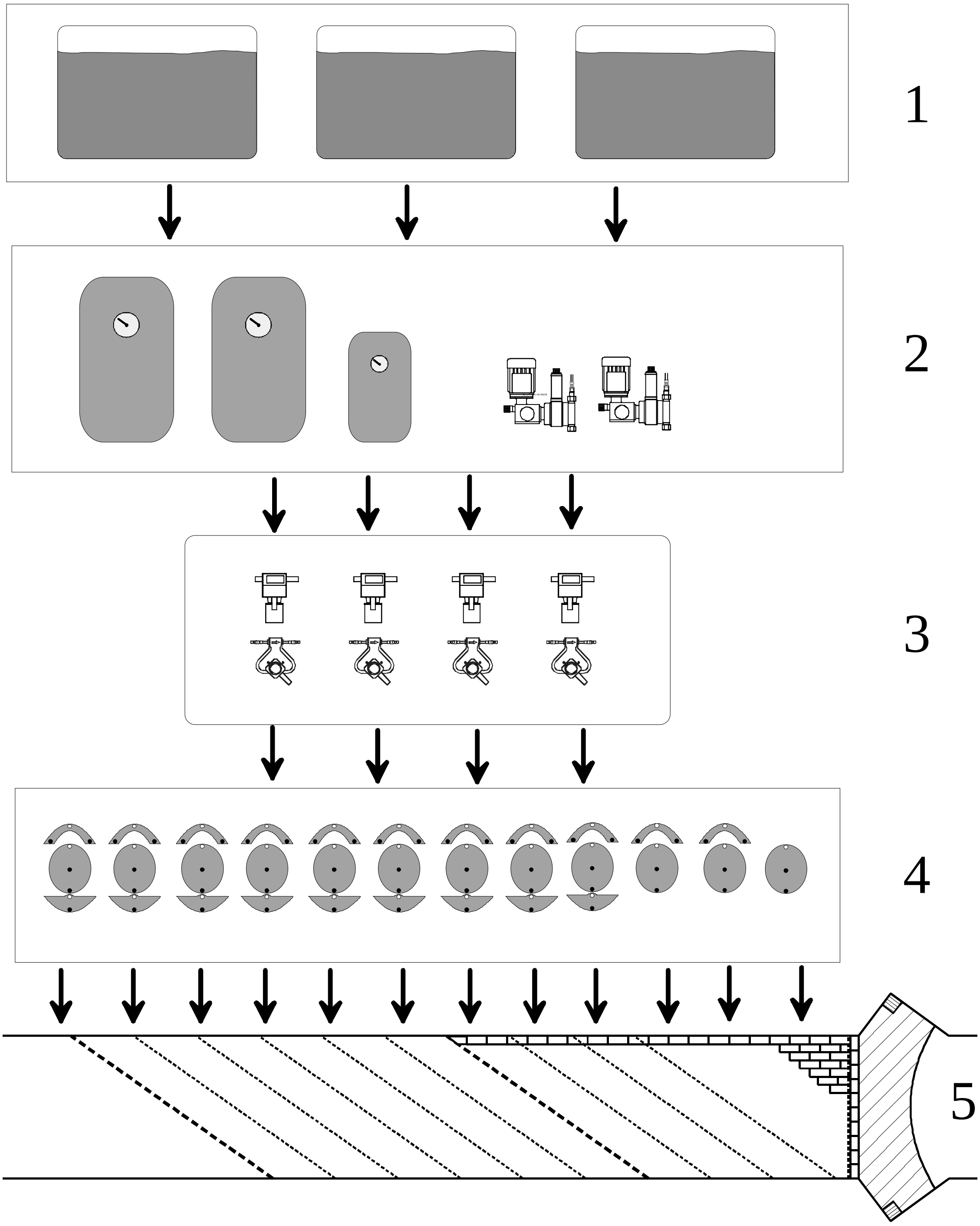


***Figure 11-28**. Schematic overview of the flow and pressure control system. The components are described in the text.*

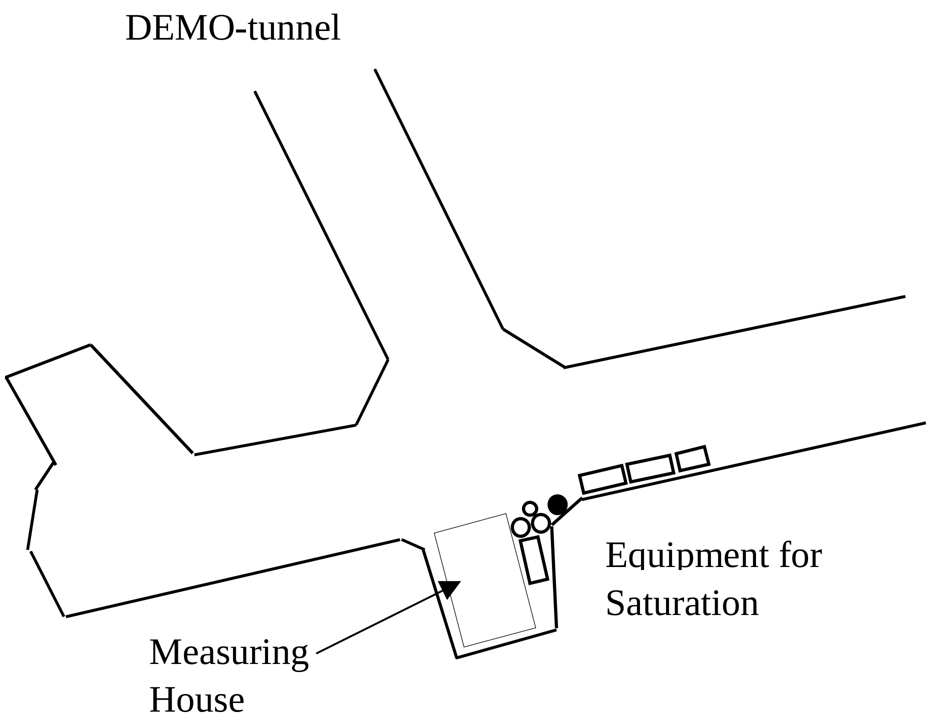


Figure 11-29. *Location of pressure vessels and storage tanks*

Figure 11-30. *Photo of pressure vessels and storage tanks.*

11.13.4 Pressure vessels and pumps

Two 500 l stainless pressure vessels (PUMPPULUHJA, see Figure 11-31) and one 150 l stainless pressure vessel (PUMPPULUHJA) have been installed. The latter is mainly used as backup when the 500 litres vessels are filled. The pressure vessels were equipped with Rima 600 kPa manometers.

Nitrogen gas is used for supplying the pressure. Two gas valves for nitrogen gas (AGA RB 200/2 0.1-1 bar SG 633 015 and SG 633 014 (see Figure 11-32) will be used for setting the pressure.

The pumps for applying pressure and flow to the test volume are plunger diaphragm pumps (SERA R 409.1-45/KM/28 and R 409.1-17 KM/20).

Figure 11-31. *The pressure vessels with the 600 kPa manometers.*

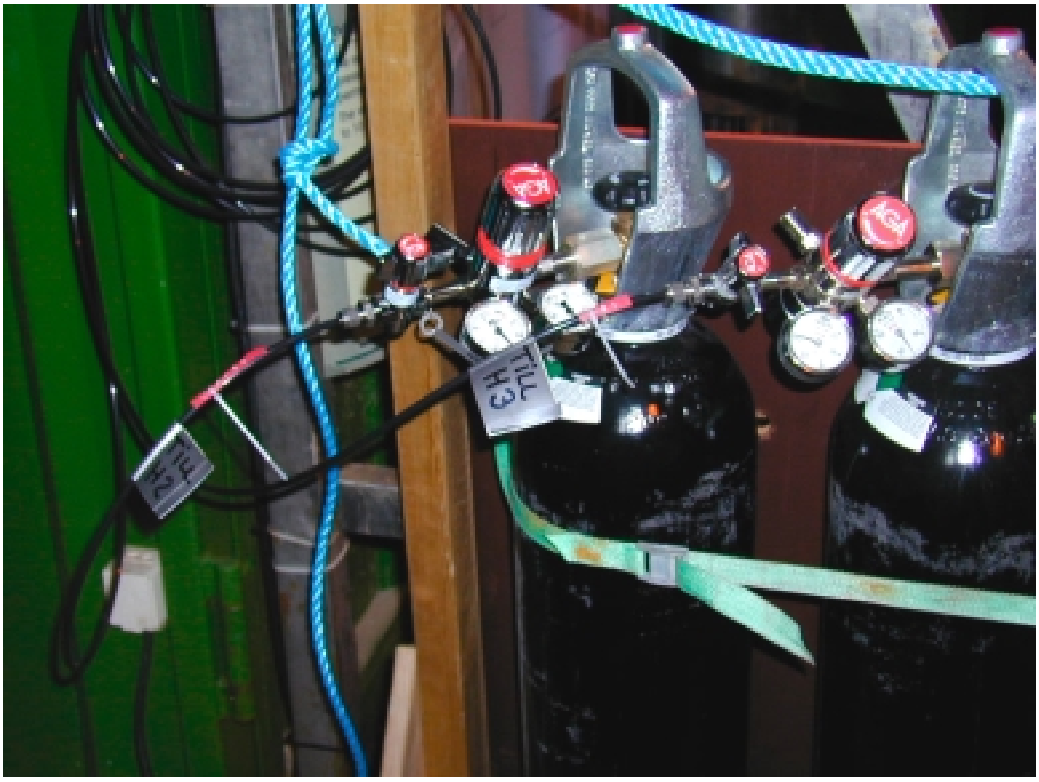


Figure 11-32. *The AGA gas valves*

11.13.5 System for measuring and regulating flow and pressure

The axial conductivity of the backfill and the near field rock will, after water saturation, be tested by applying a water pressure gradient along the tunnel between the permeable layers and measuring the water flow. The tests will be made with a system for measuring and regulating flow and pressure.

The purpose of the system is to measure the flow from and control the pressure in the permeable layers. With this system the flow between the permeable mats in each permeable layer can also be measured.

The system consists of the following units:

1. four flow sensors CMF 010 M with amplifiers
2. four types of pressure/flow control valves (Brooks control valves No. 1,2,6 and 7)

A picture of the system is shown in Figure11-33 and a drawing of the system is presented in Appendix 10.

Figure 11-33. System for measuring and regulating flow and pressure

11.13.6 Control panel for the permeable mats

The tubes from the permeable mats were arranged so that the respective permeable mat could be identified (Figure 11-34). The control panel makes it possible to connect a desired pressure source (pressure vessel or pump) through a flow-meter to a desired permeable layer or layers. A drawing of the control panel is presented in Appendix 11.

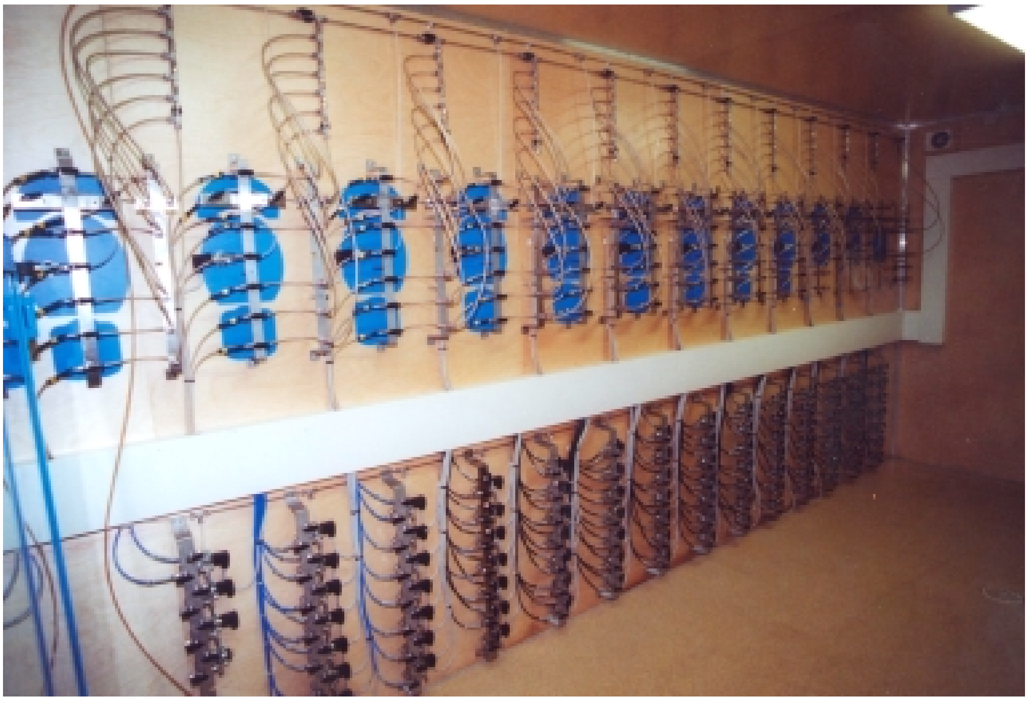


Figure 11-34. *Photo of control panel for the permeable mats (upper system) and the system for connecting the Druck transducers*

11.14 Installation of micro-organisms

The installation of micro-organisms is presented in Appendix 12.

12 PLUG

12.1 Functional requirements

The plug in the ZEDEX drift is required to:

1. Provide mechanical support to the compacted backfill material

2. Sustain a high hydraulic pressure

3. Provide a tight seal between the backfilled part and the open part of the tunnel

The mechanical support of the backfill is the least demanding requirement. The soil pressure is a couple of hundred kPa.

The support provided by the backfill must be established very soon, i.e. not later than at the end of the backfilling operation.

The hydraulic pressure that will emerge over time is on the order of a couple of MPa and requires a rigid construction that is safely anchored to the rock.

The sealing function must be provided soon after the tunnel backfilled has become saturated. The sealing effect requires that 1) the plug itself is low-permeable, 2) a seal is arranged between the plug and the rock and 3) possible flowpaths past the plug in the EDZ are cut off.

12.2 Plug design

12.2.1 Main features

The following general design principle was decided:

- The plug shall be constructed from water-tight, reinforced concrete.

- The plug needs to be keyed into the rock walls for mechanical support and for cutting off EDZ flowpaths. A slot has to be cut in the rock walls.

- An O-ring sealing of highly compacted bentonite shall be inserted between the plug and the rock.

12.2.2 Calculations

Scoping FEM flow calculations and FEM mechanical analyses were performed in order to determine load conditions and plug dimensions. All calculations were performed assuming the tunnel to be of circular cross section and the plug to be symmetric about the tunnel axis.

Flow calculations

The flow calculations showed that the hydrostatic pressure acting on the plug may amount to 1.8 MPa to 2.5 MPa /1-10/. The results vary depending on the conductivity assumed for the rock and for the plug materials (concrete and bentonite) and on the properties of the EDZ and the plug/rock interface. An example is shown in Fig 12-1.

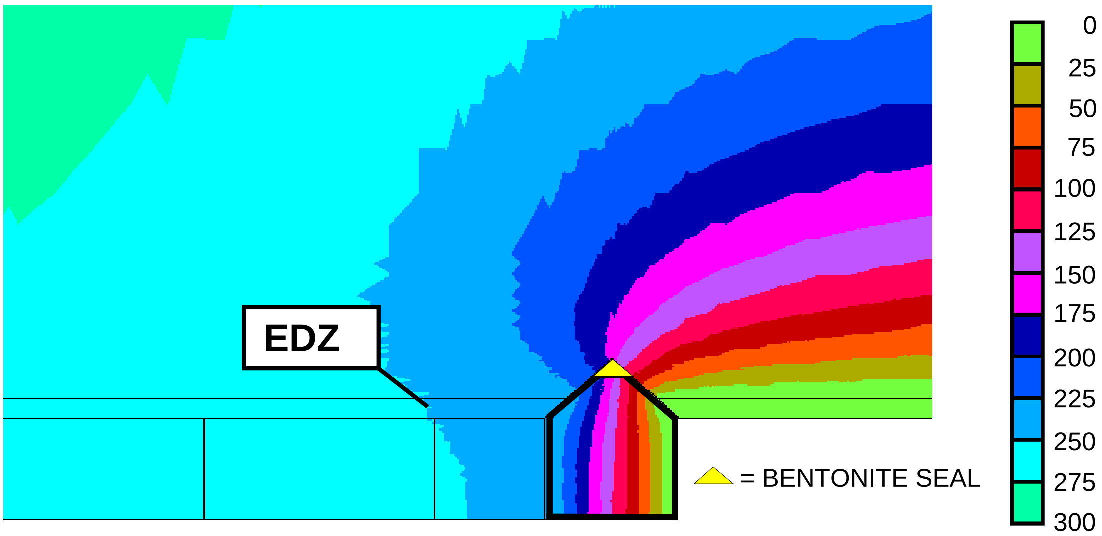


Figure 12-1. *Hydraulic head contours [m] around axisymmetric plug, keyed into the tunnel periphery. The pressure on the plug is 2.44 MPa.*

Mechanical calculations

A 3.2 MPa hydrostatic load, uniformly distributed over the 20m^2 plug area, was assumed in the FEM stress analysis /1-10/. The calculations showed that the plug should have a parabolic shape on the side facing the open tunnel in order to minimise the plug volume and the amount of reinforcement. The thinnest, central part of the plug needs to be 1.2 m thick. The load-carrying, or bearing, area, i.e. the slot surface on the open side, should have a 39° inclination relative to the tunnel axis.

12.2.3 Design and construction sequence

General

Figure 12-2 shows the design schematically.

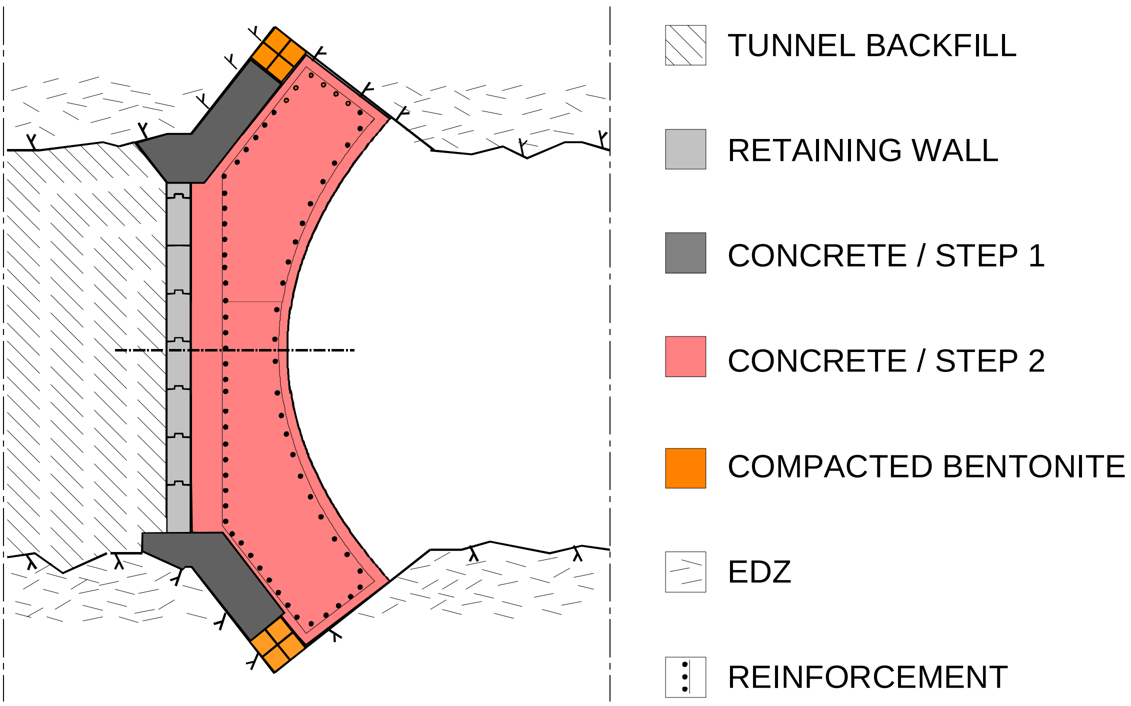


Figure 12-2. *Longitudinal cross section of rotationally symmetric plug*

Slot shape and depth

The slot has a triangular cross section. To get a 90° angle at its inner end, the inclination of the backfill side of the slot surface was set at 51°(since the inclination at the open side should be 39° according to the stress analyses). The edge is circular with a diameter of 8 m, which gives an average slot depth of about 1.5 m.

Retaining wall and abutment

The 0.3 m thick retaining wall is made of prefabricated 0.3 m x 0.6 m reinforced concrete beams. The abutment is cast and reinforced together with a 0.5 m thick rim that, together with the slot surfaces, defines the space reserved for the bentonite O-ring. The abutment and rim are referred to as “concrete step 1” in Fig 12-2. The stress analyses have shown that stresses in the “concrete step 1” volume are low. This means that “concrete step 1” is hardly involved in the transfer of forces from the pressurized backfill to the rock and that it can be cast separately from the main plug body.

O-ring sealing

The O-ring space, which has a 0.5 m x 0.5 m square cross section, is filled with bentonite blocks. The target swelling pressure is 5 MPa, which means that an average dry density of 1600 kg/m^3 is required.

Plug body

The main plug body transfers the hydrostatic pressure to the slot surface bearing area. The stresses that are generated in it ("concrete step 2" in Fig12-2) are almost entirely compressive according to the stress analyses. Only surface reinforcement is required /1-10/.

Rock/concrete interface

Due to shrinkage of the concrete, gaps may form at the interface between the rock and the plug body. This may allow for deformations that are not accounted for in the stress analyses when the hydrostatic load is applied to the plug. To minimise such deformations it is necessary to fill up the gaps to establish a firm mechanical interaction between plug and rock. Therefore, tubes for contact grouting are fixed to the slot surface bearing area. The grouting will not to be required until the hydrostatic load is going to be applied. Contact grouting may also improve the hydraulic sealing between plug and rock, if the bentonite O-ring should not perform as expected.

Sequence of operations

The sequence of operations was as follows:

- Slot excavation.
- Casting of concrete step 1 (floor and walls)
- Installation of retaining wall in conjunction with the backfilling between the outermost part
- Casting of step 1 concrete (roof region)
- Application of SyncoFlex flexible sealing lists for sealing of the concrete components "step1 and step 2" .
- Installation of system for contact grouting
- Construction of main body mould
- Preparations for application of steel reinforcement
- O-ring emplacement
- Application of reinforcement

- Installation of cooling system
- Sealing of gaps between bentonite blocks
- Casting of step 2 concrete

12.3 Plug position

The central part of the plug, i.e. the slot edge, is located at about 42 m distance from the tunnel stuff (Fig 12-3). See also drawing ZEDEX-111B. The position was selected to achieve a sufficient distance from the ZEDEX/transport tunnel intersection and from major fractures, and yet reserve a length of tunnel sufficient to accommodate the backfill experiment. In the co-ordinate system specifically used for the ZEDEX tunnel, the centre of the plug is in (Lm, Sm, Gdz) =(38.00, -0.06, 1,85). In the general Äspö system this corresponds to (X, Y, Z) = (2277.52, 7281,49, -416,08). Fig 12-3 shows the slot in relation to a theoretical 5 m diameter circular tunnel, while Fig 12-4 shows the slot intersection with actual tunnel profiles in sections Lm 39, Lm 38 and Lm 37.

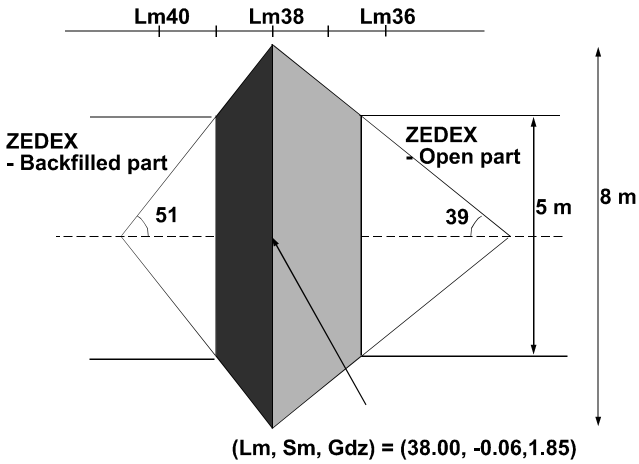


Figure 12-3. *Position of the plug*

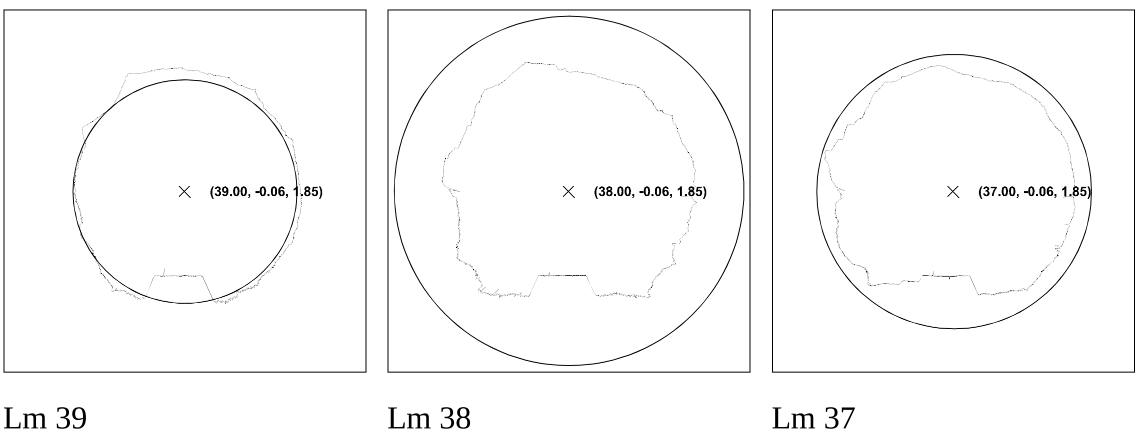


Figure 12-4. *Slot/tunnel intersections. Tunnel profiles are viewed from inside of ZEDEX tunnel. Tunnel profiles were obtained by laser scanning. The flat floor portion is an artefact, caused by the track used for moving the laser instrument along the tunnel.*

12.4 Slot

12.4.1 Excavation technique

No additional excavation damage in the rock surrounding the plug could be allowed, since this could jeopardise the sealing function. The slot was therefore excavated by use of core drilling without blasting operations close to the slot surfaces. The boreholes must be set close enough that a reasonably smooth slot surface was obtained. A rig that allows for accurate direction of the drilling aggregate and for drilling of many holes with the same inclination relative to the plug symmetry axis was designed. It consisted of 1) two circular frames that were bolted to the tunnel periphery and adjusted to coincide with the circular slot geometry, and 2) a system for rotating, directing and fixing the drilling aggregate.

12.4.2 Accuracy requirements

The outermost part of the slot will contain the bentonite O-ring seal. For making hte bentonite blocks fill the volume without requiring expansion beyond the volume that corresponds to the target swelling pressure (5 MPa), a number of requirements were defined. For the O-ring part of the surface, i.e. within 0.5 meters from the slot edge, the maximum allowed roughness was set to ±10 mm/dm^2. For other parts, i.e. the rock/concrete interfaces, 30 mm "ridges" between adjacent boreholes were allowed.

12.4.3 Excavation

The excavation of the recesses was made by drilling of a number of 0.8 m deep holes drilled radially in length section Lm38. They were intended to yield simple removal by use of careful blasting, once the slot surfaces had been drilled.

340 holes with 46 mm diameter were drilled from outside at 39° inclination relative to the plug symmetry axis. The drilling rig was then moved to the inner drilling position and another 340 holes were drilled at 51° inclination. In the slot/tunnel intersection, no rock remained between neighbouring holes. The maximal centre distance (in the inner end of the slot) was about 74 mm, which means that, on an average, 27 mm rock remained between adjacent holes. Therefore, all of the released rock did not fall out after completion of the drilling operation.

The 0.8 m radial holes were charged with Dynamex 25 mm and Gurit 11mm. The total charge amounted to 70 g/hole.

Larger blocks that remained after blasting were removed by use of a hydraulic expander.

12.4.4 Control measurements and reworking of the slot surfaces

Two series of measurements were conducted after the slot excavation for checking the geometry of the slot. Fig 12-5 shows the result of the first one. 120 points on the slot surfaces and on nearby parts of the tunnel periphery were measured.

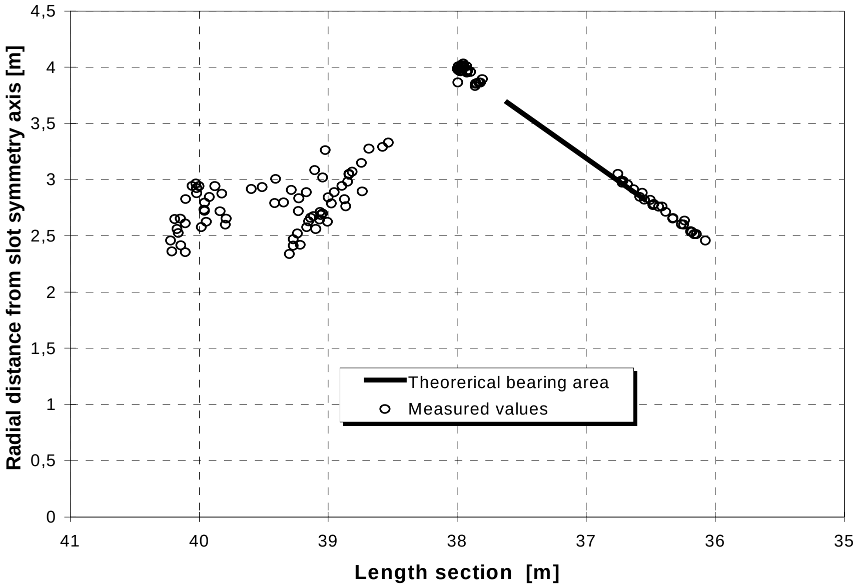


Figure 12-5. *Radial distance of points on the slot walls and on nearby points on the tunnel periphery*

Fig 12-6 shows the results of the second series of measurements, which was performed in four selected sections.

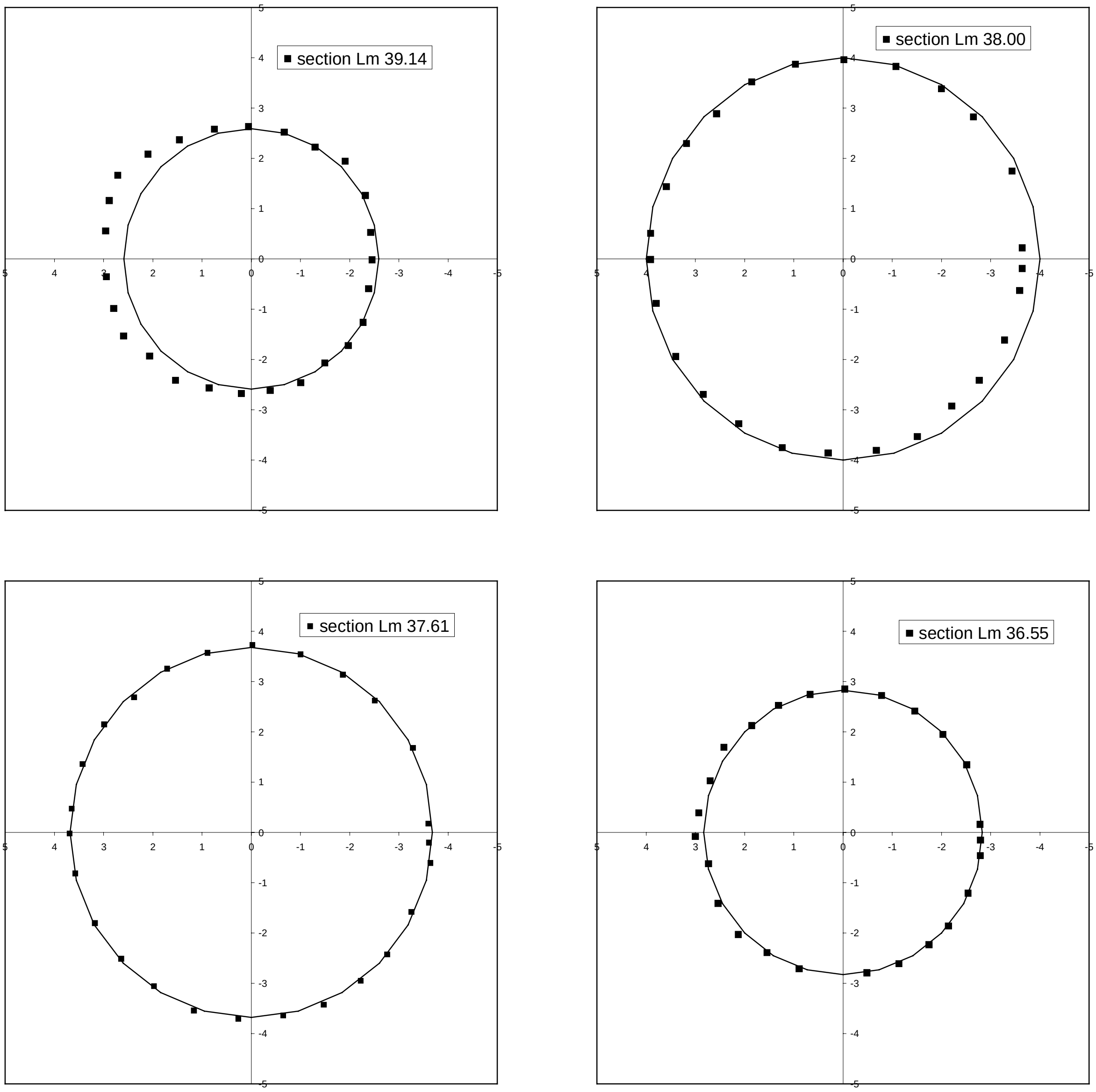


Figure 12-6. *Periphery points compared with theoretical sections. Views are from inside the tunnel.*

The control measurements show that the shape of the slot conforms well to the intended slot shape. In the slot edge, i.e. in section Lm38, more work was needed to remove remaining rock, in particular on the right hand side (see upper right part of Fig 12-6). The reason was that not all holes in this part had been drilled sufficiently deep to meet the holes drilled from the other side. Fig 12-7 shows the slot surface in the edge region after the final reworking had been completed.

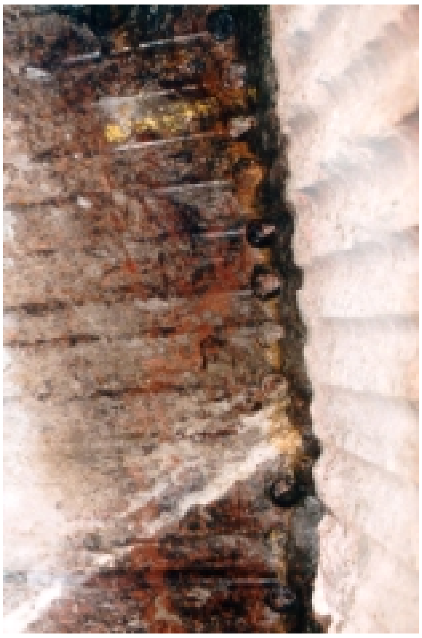


Figure 12-7. *Slot surfaces after reworking.*

12.5 Retaining wall and abutment

12.5.1 Construction

Construction of abutment and retaining wall proceeded as described below. Details are found in drawings ZEDEX-112C and ZEDEX-113B.

- The abutment and the conical rim, i.e. the *step 1 concrete (Fig 12-2)*, were bolted to the rock walls and cast up to a level just below the tunnel roof (see Fig12-8). In the upper part a clearance was left for the compaction truck. The inner periphery of the abutment was divided in straight-line segments, corresponding to the subdivision of the retaining wall into horizontally stacked concrete beams. Each segment was prepared with sockets for subsequent mounting of supporting angle irons.
- The seven 0.3 m x 0.6 m reinforced concrete beams were cast and stored on ground.
- Prior to starting the backfilling operation, the floor section of the slot was filled with gravel up to the level of the abutment threshold in order to allow for transport in and out of the tunnel.
- At the end of the backfilling operation, the concrete beams were mounted one by one and locked in position by use of supporting angle irons that were bolted to the abutment. When the backfill reached a level just below the top of the last mounted beam, the next beam was mounted, etc.
- After having backfilled the last part of the roof region, the uppermost part of the abutment and the retaining wall were cast in one, consecutive operation.

Figure 12-8. Abutment viewed from inside the ZEDEX tunnel prior to starting the backfilling operation

12.5.2 Sealing

Flexible swelling sealing lists were fixed to the step 1 concrete to minimise flow along paths in the interface between step 1 and step 2 concrete. Details are found on drawing ZEDEX-115E.

12.6 Rock/concrete interface

The bearing part of the slot surface was prepared for contact grouting. Three tubes were fixed to the wall at different distances from the slot edge, each of the tubes covering the entire circular slot periphery. Each of the three tubes was subdivided in three segments that could be pressurised with grouting cement individually. The details are shown in drawing ZEDEX-117B. A type of tube was used that allows for two grouting episodes, provided that the tubes are rinsed with pressurised water in between.

12.7 Bentonite O-ring

12.7.1 Bentonite blocks

Dimensions.

The longitudinal cross section of the space between rock walls and *concrete step 1* is approximately square-shaped and 0.5 m x 0.5 m in area (left part of Fig 12-9) excluding an average clearance of about 2 cm. The bentonite blocks were compacted to the dimensions shown in the central and right parts of Fig 12-9. The graded block thickness conforms to the radial slot geometry.

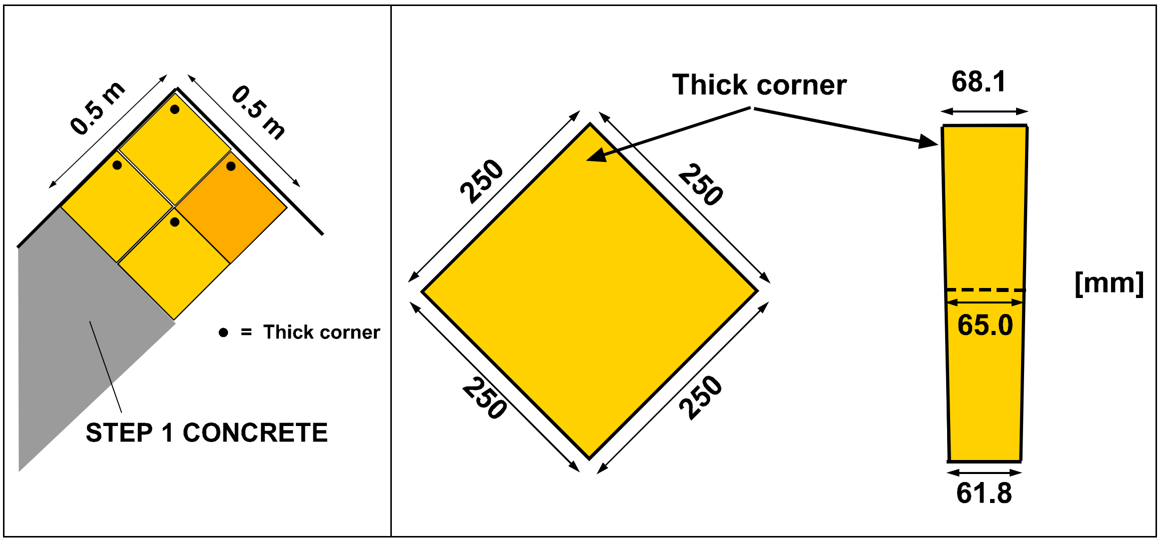


Figure 12-9. *Block geometry*

Material and manufacturing

To completely fill the O-ring space about 1500 blocks were needed. They were compacted from granulated MX-80 bentonite with 12% water ratio at Höganäs-Bjuf AB. To arrive at the target density after water saturation, the bulk block density needed to be about 2050 kg/m^3 corresponding to 8.33 kg/block. This required a compaction pressure of about 100 MPa. In addition to the 1500 blocks for the O-ring, another 300 blocks were manufactured for testing purposes.

12.7.2 Emplacement technique

Bentonite/water interaction.

The bentonite blocks had an initial water ratio of 12%, which gave very strong soil suction. The relative humidity in the ZEDEX tunnel is about 85%, which means that the blocks start taking up water and crack and eventually fall apart. In addition to the water taken up from the moist air, some water is supplied from the rock. Therefore the process of O-ring installation had to be fast, and the main body of the plug had to be cast very soon after completing the O-ring installation.

Block support

In the upper parts of the slot the blocks rest on the sloping concrete rim. In order to prevent slip, a system of metal strips was designed (see Fig 12-10).

Bentonite/concrete interface

The bentonite O-ring is surrounded by rock and concrete. At the time of bentonite emplacement, the step 1 concrete had hardened and matured. The plug main body, however, had to be cast after the bentonite O-ring was installed, which means that there would be an interaction between unsaturated bentonite blocks and fresh concrete. Laboratory tests have shown that some disturbance due to water transport from the concrete mass to the bentonite will appear. The interaction zone is, however, thin, and the disturbance will not have a significant effect on the sealing function /12-1/.

Sealing of gaps

Gaps between the bentonite and the rock wall, bentonite and concrete step 1, and between bentonite blocks had to be sealed in order to prevent concrete penetration when the plug main body was cast. The system used for gap sealing is shown in Fig 12-10. The sealing systems required that metal strips were installed around the entire periphery, not only in the upper part. Tangential gaps between blocks were sealed by use of mineral wool.

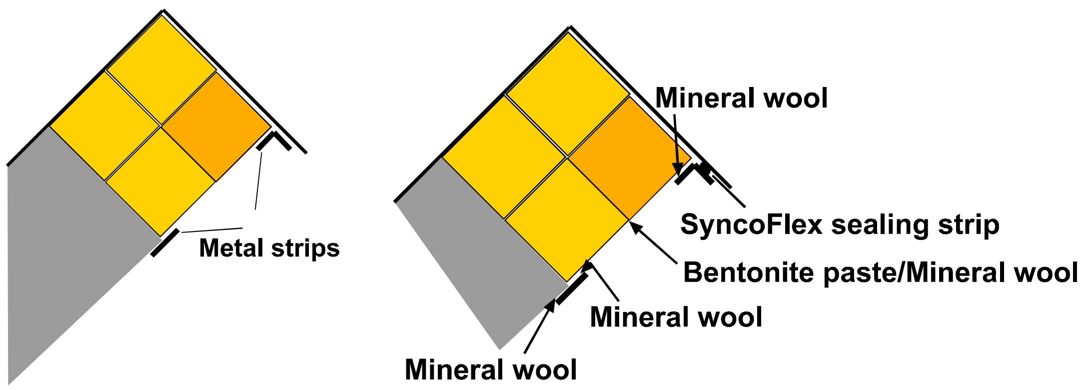


Figure 12-10. *Systems for supporting and confining blocks and for sealing of gaps. Radial gaps were sealed with bentonite paste only.*

12.7.3 Testing the emplacement technique

A number of bentonite blocks were installed in 5 different positions according to the left part of Fig 12-11 for testing the technique. The right part shows position V3. Each position except position B contained 5 layers of blocks, i.e. 20 blocks. Position B contained 40 blocks. The blocks were left for 10 days, and then recovered for laboratory determination of the water ratio. Only blocks in the central layers were sampled. The relative humidity in the vicinity of the slot was measured and controlled during the test period. Fig12-12 shows the relative humidity during the test period.

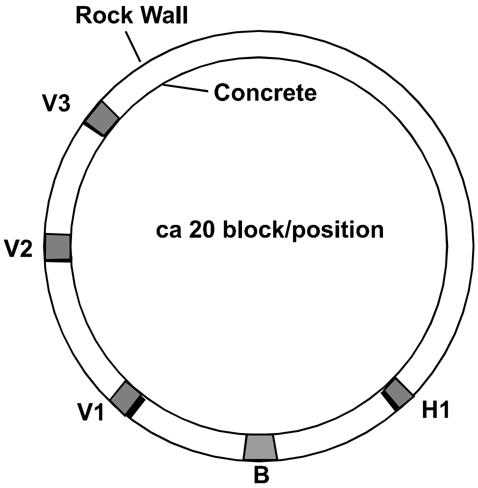


__Figure 12-11.__ Left: Positions for test emplacement. Right: Blocks in position V3. The reinforcement bars sticking out of the concrete rim were intended as support for the blocks, but were never used. Prior to the real emplacement they were removed.

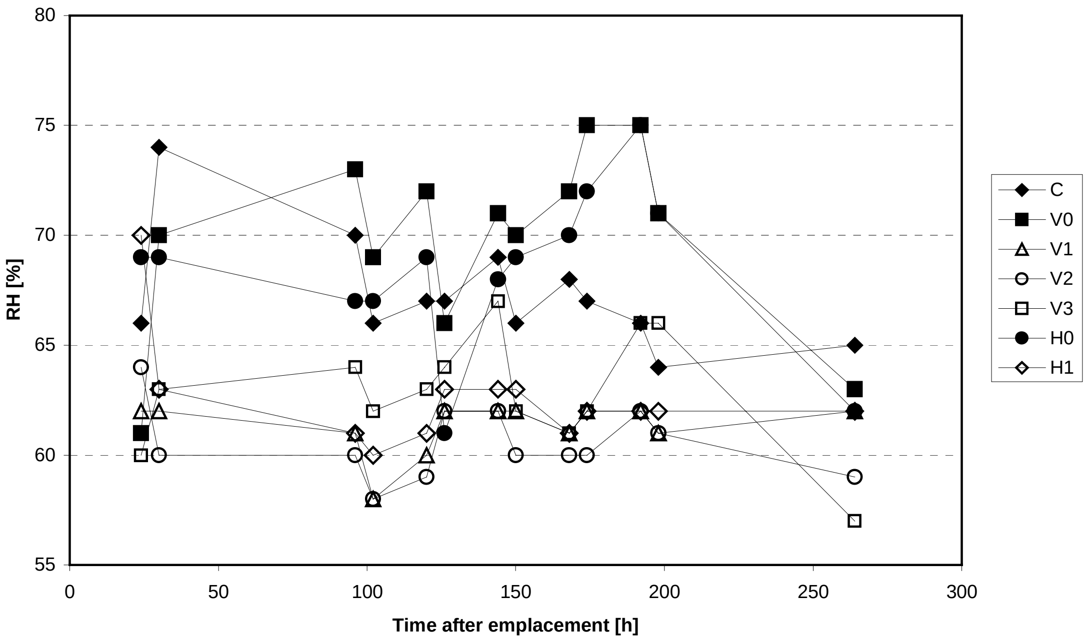


__Figure 12-12.__ Relative humidity in different parts of the test area. C is the central part of the tunnel. V0 and H0 are to the left and to the right, respectively, of position B.

Fig 12-13 shows examples of results from the water ratio determinations. The initial water ratio was 12%.

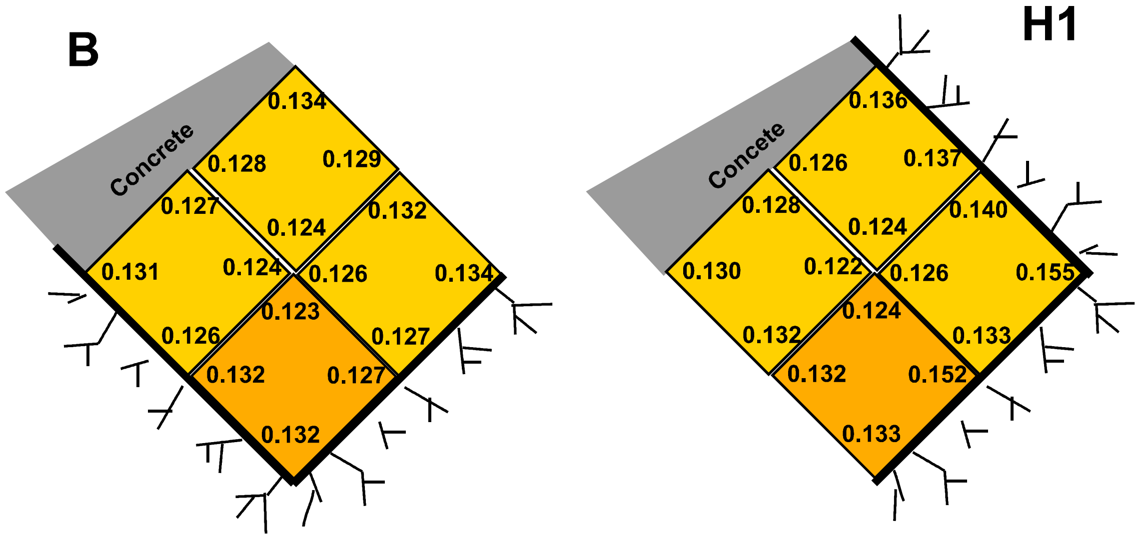


Figure 12-13. *Water ratios in positions B and H1 after 10 days*

During the test period water was pumped from a 1 m deep sump just outside the slot. The intention was to drain the EDZ locally and reduce the amounts of water that flowed down into the slot from the floor. Prior to emplacement, water moved down into the slot also from a small part of the slot wall at position H1. This flow came to an end as the blocks were positioned, which indicates that local swelling and sealing had occurred. The local increase in water ratio (right part of Fig 12-13) shows that water had been absorbed.

The general outcome of the test was that the bentonite blocks would endure a 10-day exposition to the moist tunnel air, and that the system with supporting metal strips was expedient and practical.

12.7.4 Emplacement

Prior to filling of the O-ring space, careful preparation was made to make it possible to cast the plug soon after the O-ring installation. The reason for doing as much of the work as possible in advance was to reduce the time between block emplacement and concrete casting, for reducing water uptake from the moist air. The preparatory work included:

- Construction of the concrete mould. Passages were left for transportation of bentonite blocks, reinforcement bars, cooling equipment and vibration equipment.
- Installation of system for contact grouting.
- Installation of system for evacuation of air that would be trapped in the slot top towards the end of the concrete casting.
- Installation of system for anchoring the reinforcement to the slot walls and to the “step 1” concrete.

The O-ring work thus had to be conducted in a closed space within the mould.

Temporary supports were bolted to the rock walls and the existing (step 1) concrete (Fig 12-14). Then, the O-ring space in the wall and roof regions were filled with blocks according to the system that had been tested previously, i.e. using metals strips to secure and confine the blocks. Gaps were sealed with SyncoFlex and mineral wool. Finally, the floor region was filled in the same way and the temporary supports were removed.

Theoretically, 1410 blocks were needed to fill the O-ring space but only 1300 were emplaced. The reason is probably that blocks were not always positioned as intended, i.e. with the thick corner pointing outwards (c.f. Fig 12-9), which created regions with gaps between blocks. In the upper right part, 13%-15% fewer blocks than intended were emplaced. As an average, 7% fewer blocks than intended were emplaced. The density after saturation in the upper right region was estimated to be 1907 kg/m^3, including the impact of gaps between blocks and rock and between blocks and concrete. This density does just suffice to yield the minimum required swelling pressure, 2MPa.

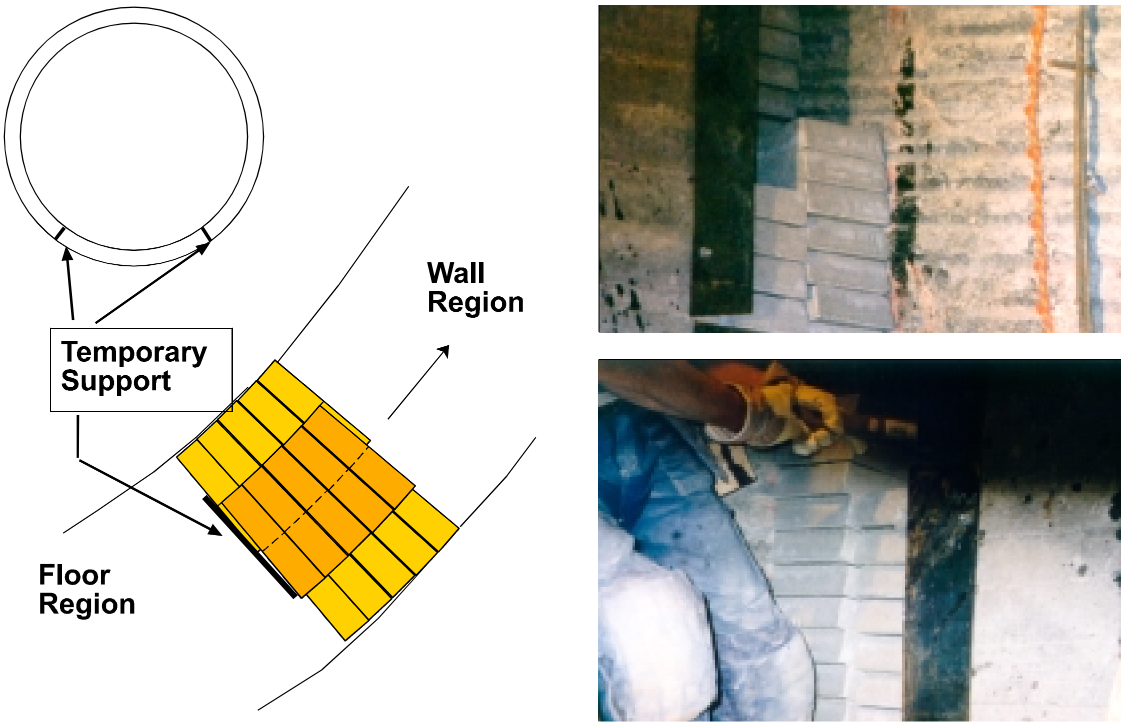


__Figure 12-14.__ Left: The O-ring space in the wall and roof regions were filled first. Right: emplacement of blocks in the wall regions. In the upper picture, parts of the systems for contact grouting and reinforcement anchorage can be seen.

The emplacement work started on September 13 and was completed on September 16. The test emplacement had shown that the blocks could endure exposure to humid air for 10 days. This means that about 6 days were available for finishing the work within the mould. The moisture control, which had been applied for 24 hours a day during the test period, could, however, only operate at nights because of working condition considerations. A final inspection of the blocks on September 22 showed that all blocks were in acceptable condition.

Sealing of gaps between blocks was not performed until the reinforcement and the cooling system had been installed, i.e. just prior to casting the main body concrete.

12.8 Main body

12.8.1 Concrete mould

The concrete mould was built from a steel beam skeleton that was bolted to the rock, and a plywood/wood surface. The steel skeleton consisted of 14 radial beams for support of the surface and a system of beams for transfer of forces to the rock. The plywood/wooden surface was given a parabolic shape. Fig 11-15 shows the mould during construction. The details are found in drawings ZEDEX-121B, ZEDEX 122B and ZEDEX-123B.

Figure 11-15. *The mould*

Note that the mould was constructed prior to the bentonite O-ring. Parts of the mould were left open to allow for transport of bentonite blocks, reinforcement bars, cooling system and concrete vibration system.

12.8.2 Reinforcement

The reinforcement bars were mounted outside the mould, transported through temporary openings and assembled inside. Fig 11-16 shows the reinforcement facing the retaining wall and the parabolic mould surface. Details of the reinforcement are found in drawing ZEDEX-115E.

Figure 11-16. *Reinforcement at the retaining wall (left) and parabolic outer surface (right). Photographs were taken during installation of cooling pipes, i.e. after completion of the reinforcement work.*

12.8.3 Casting

An amount of 65 m^3 concrete BTG 1 K50 was used for the plug main body. The casting was performed on September 23 and completed within 12 hours. The concrete was pumped through holes that had been prepared at different levels of the mould. In the lower parts, corresponding to about 2/3 of the total plug height, the concrete was vibrated by use of vibro-tubes that were manually operated from inside the mould. Vibration of the upper part was performed by means of a vibrating machine that was positioned about 0.5 m below the slot roof-edge and fixed to the rock walls and to a cage of stiff rods. The cage was anchored to the rock walls but mechanically separated from the reinforcement. The rods reached all parts of the upper slot volume, thus ensuring efficient vibration also of that part of the plug. At the end of the casting, the rising concrete surface reached the vibrating machine which was eventually buried while still running. After having completed the casting, the power cords were cut.

The pumping continued until concrete poured out of de-airing tubes, which were mounted on the mould and reached the top of the free space just below the slot roof-edge.

12.8.4 Cooling

During hardening, concrete generates heat, which will cause volume expansion and thermal stresses that could create cracks. A cooling system was installed prior to casting the main body (Fig 11-15). It consisted of two circuits of steel pipes in the plug interior. The circuits were connected to an external cooling unit, which controlled water flow and temperature. The details of the circuits are shown in drawing ZEDEX-118C.

The design of the cooling system was based on a numerical analysis, performed by use of a code for calculation of risks for fracturing in hardening concrete. The analysis showed that a cooling power of 13 kW was needed.

After completion of the casting, cooling took place for about 50 hours. The temperature was monitored by use of sensors, located in selected positions within the hardening concrete, and compared with corresponding calculated temperatures. The temperatures appeared to be between 2 and 5 degrees higher that predicted, probably due to a higher initial temperature of the fresh concrete than assumed.

About 13 days after completed casting the free surfaces of the plug were inspected, without finding any indications of fracturing.

12.9 Monitoring

Prior to casting, two Gloetzl cells for measurement of the total pressure were installed on the surface of the bentonite O-ring. The intention was to monitor the development of the swelling pressure. The instruments were anchored to the reinforcement with the membranes fixed to the block surfaces. Sealing precautions, i.e. application of mineral wool, were taken to prevent intrusion of concrete between membrane and bentonite block surface. The two instruments were placed in the upper right part and the lower left part, respectively.

13 Data flow and documentation

The quality assurance entailed registration and follow up of all parts of the instrumentation and measurements during backfilling. The following protocols were used:

For each backfill section:

- An overview protocol of the sub-protocols used in one backfill section (S1).
- A check list for fastening all Tecalan tubes in the flange of the through connection cone (S2)
- A protocol for leakage test of the through connections (S3)
- A protocol for the filter mats and the connecting tubes (S4)

For each backfill layer:

- A drawing with a description of the instrument positions in the backfill
- A protocol with a table for comments on the instruments in the backfill and rock (L2)
- A protocol for measurement of density and water content (L3). The data were compiled on Excel data sheets and delivered to SICADA. The analyses of the data are presented in chapter 8. The files were given the names *sectionNN.xls* and *WNN.xls*, where NN stands for section name, i.e. one file for density and one for water ratio for every backfill section was created.
- A protocol for registration of the weight of backfill for each layer (L4). In practice, the scale of the carrier delivered a receipt that was attached to the protocol. The data were compiled in Excel data sheets and were delivered SICADA. The File was named weighing.xls. The analyses of the data are presented in chapter 8.

The surveying resulted in data files with co-ordinates for the instruments, the layers, the permeable mats and the rock/backfill contact.

The filled-in protocols are stored at Clay Technology in Lund. They were used when reporting the final instrument positions (see chapter 11). Data flow charts for the backfilling of one section and one layer are shown in Figure 13-1 and 13-2.

Apart from these special protocols, Äspö Daily Logs were completed and delivered to the co-ordinator.

Photographs were taken continuously during the backfilling and the main events were filmed.

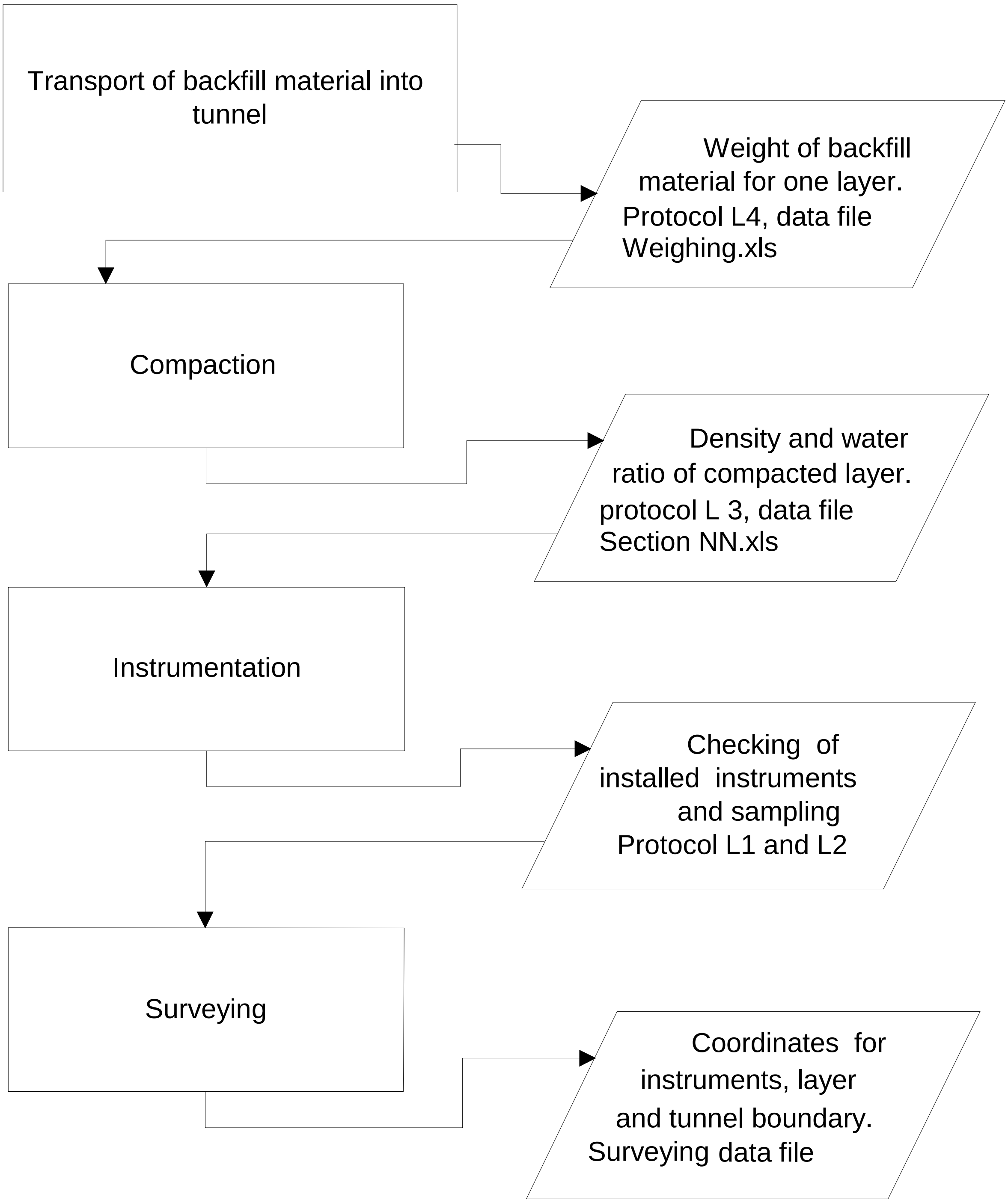


Figure 13-1. *Flow chart for one layer*

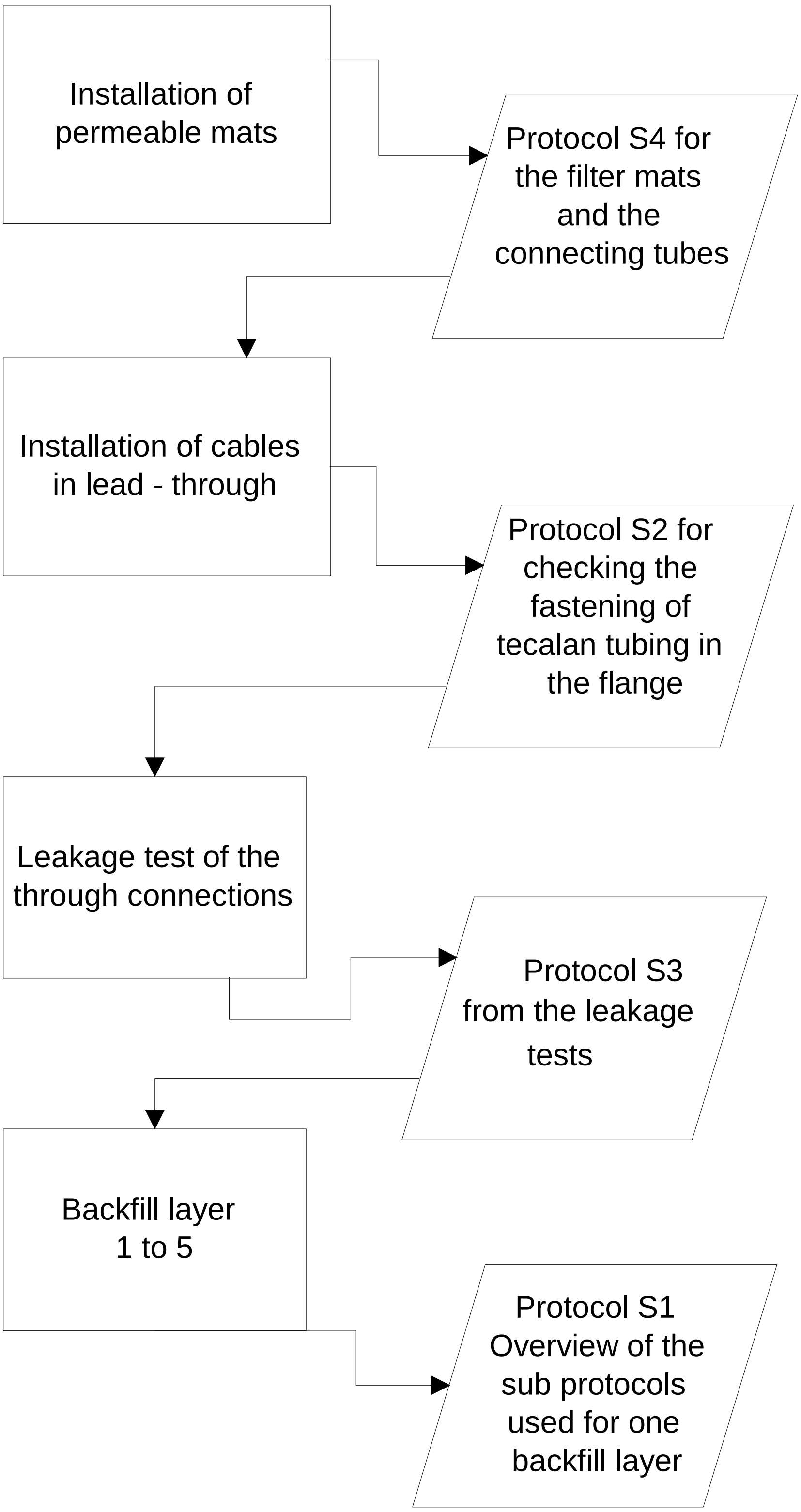


Figure 13-2. *Flow chart for one section*

14 Discussions and conclusions

14.1 General

Since this report only deals with the set up of the test, no conclusions can be made regarding function and test results, except for the densities reached during backfilling. In general the installation worked well and all problems that occurred were apparently solved in a satisfactory way. On the other hand the test set up was complicated, since many new techniques had to be developed and applied and many procedures took longer time than expected.

14.2 Mixing of backfill material

The mixing of backfill material for the Backfill and Plug Test was performed without any problems. Excluding the two weeks that included establishment and adjustment of the mixing station, the mean number of tons mixed per day was 55 for 0/100 and 17 for 30/70. It is difficult to increase the mixing rate with this type of technique.

14.3 Backfill equipment and technique

The backfilling was successful. The developed technique and equipment worked as expected.

The backfilling technique requires a lot of skill from the operator of the Backfilling equipment. The successively increased skill was evident when analysing the densities of the material close to the roof. It was found to be increased from the first couple of layers to the last three sections of 30/70 material. The instrumentation in the roof disturbed the compaction for almost all of the layers. Extreme irregularities in the roof of the tunnel also affected the density in this area in a negative way.

The backfilling equipment worked well in terms of achieving expected densities, but it needs to be improved concerning reliability, durability and safety.

The backfilling rate in the Backfill and Plug Test was not very high due to the extensive instrumentation. The backfilling rate of the existing backfilling system, excluding disturbances, is estimated to 1 m per shift. The backfilling speed can be further increased if the time for transporting material into the tunnel can be decreased. This may be achieved if the carrier is rebuilt with a conveyor belt or similar, making it possible to transport backfill material past the carrier to the backfilling surface. The material could then be supplied with a loader. The material could, for example, be dumped in a container mounted to the carrier.

14.4 Achieved densities

14.4.1 General

The problems encountered with respect to obtaining high densities close to the roof in earlier tests /1-11/, were overcome. The compaction work close to the walls, roof and floor of the tunnel worked well, at least in the outer sections, but it can probably be further improved in a bored tunnel with no instruments placed in the roof.

14.4.2 30/70

In the areas that could be accessed with the slope compactor a mean dry density of 1700 kg/m^3 was achieved. When backfilling proceeded without disturbance a median dry density of more than 1500 kg/m^3 at the roof was achieved. The skill of the operator of the carrier increased during the length of the backfilling. The density close to the roof in the two first sections was thus lower. Avoiding harming instruments in the roof resulted in local zones of lower density also in the later sections. Although the density close to the roof and walls is lower than 1700 kg/m^3 the influence on the average density is not very strong due to that the low-density zone is not reaching more than a few decimetres away from the rock. The bulk average dry density is therefore estimated to be between 1650 and 1700 kg/m^3.

14.4.3 0/100

The mean measured dry density of the 0/100 material was 2170 kg/m^3. The installation of bentonite blocks and blowing of pellets between the backfill and the roof worked well.

14.5 Plug

The decisions for the design of the plug were preceded by literature studies, case studies and numerical calculations. The experience from construction of plugs in underground defensive installations and in hydropower plant tunnels and other was considered and taken into account. The bentonite O-ring, which is motivated by the high demands on the sealing effect, is however a component that is unique to this application, and made both design and construction of the plug complicated issues.

The construction of the plug was made in five steps:

6. Slot excavation (drilling and reworking of slot surfaces)
7. Casting of abutment and conical rim (step 1 concrete)
8. Placement of prefabricated retaining wall beams
9. Placement of O-ring bentonite blocks
10. Casting of plug main body (step 2 concrete)

The technique used for slot excavation was found to be successful as far as accuracy in shape and position are concerned.

Casting of the conical rim required that the mould be accurately designed and constructed, since the space between rim and rock had to match the dimensions of the bentonite O-ring within few centimetres in order to keep the bentonite density within the intended range.

The successive positioning of the reinforced concrete beams turned out to be a straightforward and uncomplicated operation.

The O-ring installation was the most crucial step. The bentonite blocks had to be installed after part of the work for the main body (concrete step 2) had been completed (installation of reinforcement and cooling system, mould construction). The reason was that the time for bentonite exposure to the moist air had to be minimised. A major experience is that it would be necessary to supervise the emplacement operation more carefully to avoid blocks getting irregularly oriented. There are indications that such irregularities were caused in part of the O-ring, and that significant gaps were formed between blocks. However, it was estimated that the achieved O-ring density was sufficient to give the required 2 MPa minimum swelling pressure.

The casting of the plug main body was preceded by a careful inspection of the O-ring. One main concern was that fresh concrete might penetrate the gaps between blocks and give cause to flow paths within the O-ring. Just prior to casting, these gaps were sealed with bentonite paste and/or mineral wool. Mineral wool was used with caution: radial gaps were sealed only with bentonite paste, since continuous, or interconnected, mineral wool filled gaps connecting the bentonite/concrete interface with the bentonite/rock interface could short circuit the O-ring. The effect of the sealing can not be evaluated until after the plug has been dismantled. Altogether, the casting seems to have been successful. The temperatures measured during cooling agreed reasonably well with the predicted temperatures and no visible cracks were found on the exposed surfaces. Inspection of videotapes, recorded by use of logging equipment inserted in holes that were specifically drilled through the hardened concrete for this purpose, indicated that the concrete pumping pressure had been sufficient to fill also the uppermost parts.

14.6 Installation

A number of practical problems specific for the Backfill and Plug test were encountered and solved during the installation, for example:

- Lead-throughs

In order to reach a high tightness of the lead-throughs the design became rather complicated. The installation was obstructed by problems with hole drilling since the holes were not straight and the widenings for the bentonite rings were eccentric. In spite of these difficulties the installation was apparently successful.

- Sealing off and draining the inner part of the tunnel

The inner part of the tunnel, which is not used for the experiments, had to be parted from the test sections with an inclined concrete wall, and drained with two pumps installed in a pump well.

- Installation of cables and instruments in the backfill.

The cables and instruments had to be installed on the surface of the backfill layers and lead in cable trenches to the lead-through cones. The cables needed to be separated in order not to cause flow paths and also had to be protected from being damaged by the vibrator. The latter was especially problematic at the entrance to the lead through cones where the space between the cables was filled with bentonite pellets and blocks around the cone instead of using compacted backfill.

- Removing extensiometers

Several earlier applied extensiometers that were bolted and integrally cast with the rock inside the holes had to be removed and sealed.

- Plugging of bore holes

55 boreholes with lengths varying between 3 and 40 m had to be plugged and sealed with concrete and bentonite. This was especially tricky for the holes directed upwards.

- Careful compaction close to instrumentation in the rock.
- Protection of rock instrumentation in the floor

None of these problems affected the test set-up and in general the installation worked very well.

Reference list

/1-1/ **Börgesson L., Johannesson L.-E. and Sandén T.; 1996.**
Backfill materials based on crushed rock. Geotechnical properties determined in the laboratory. SKB PR HRL 96-15.

/1-2/ **Johannesson L.-E., Börgesson L. and Sandén T.; 1998.**
Backfill materials based on crushed rock (part 2). Geotechnical properties determined in the laboratory. SKB-ÄHRL IPR 99-23.

/1-3/ **Börgesson L. and Hernelind J.; 1998.**
Preparatory modelling for the Backfill and Plug Test. Scoping calculations of H-M processes. SKB-ÄHRL IPR-99-11.

/1-4/ **SKB RD&D-Programme 95; 1995**
Treatment and final disposal of nuclear waste. Programme for encapsulation, deep geological disposal, and reserarch, development and demonstration.

/1-5/ **Pusch R., Nilsson J. & Ramqvist G.; 1985.**
Final Report of the Buffer Mass Test - Volume I: Scope, preparative field work, and test arrangements. Stripa Project TR 85-11.

/1-6/ **Pusch R., Börgesson L. & Ramqvist G.; 1985.**
Final Report of the Buffer Mass Test - Volume II: Test results. Stripa Project TR 85-12.

/1-7/ **Daoman J. and Ran C.; 1996.**
Bentonite as a Waste Isolation Pilot Plant Shaft Sealing Material. Contractor Report, Sand96-1968.

/1-8/ **- 1989.**
Design, construction, and evaluation of clay liners for waste management facilities - Office of solid waste and emergency response, U.S. environmental protection agency, Washington D.C. 20460

/1-9/ **Pusch R., Börgesson L. & Ramqvist G.; 1987.**
Final Report of the Borehole, Shaft, and Tunnel Sealing Test - Volume III: Tunnel Plugging. Stripa Project TR 87-03.

/1-10/ **Hökmark H.; 1997.**
Design av temporär plugg i ZEDEX-orten. SKB PR D-97-07.

/1-11/ **Gunnarsson D., Johannesson L.-E., Sandén T. & Börgesson L.; 1996.**
Field Test of Tunnel Backfilling. SKB PR HRL 96-28.

/3-1/ **Stenberg L. & Gunnarsson D.; 1998.**
Characterisation of the ZEDEX drift in advance of the Backfill and Plug Test, Progress Report HRL-98-10.

/3-2/ **Olsson O., Emsley S., Bauer C., Falls S. and Stenberg L.; 1996.**
Zedex – a study of the zone of excavation disturbance for blasted and bored tunnels. Volume 1-3. SKB International Cooperation Report 96-03. SKB, Stockholm, Sweden.

/3-3/ **Emsley S., Olsson O., Stenberg L., Alheid H.-J. and Falls S.; 1997.**
Zedex – a study of damage and disturbance from tunnel excavations by blasting and tunnel boring. SKB, Technical Report 97-30. SKB, Stockholm Sweden.

/3-5/ **Myrvang K.-A. and Wikman H.; 1998.**
Evaluation of in situ rock stress measurements at the ZEDEX test site area. SKB Progress Report HRL-97-22. SKB, Stockholm, Sweden.

/3-6/ **Hallbjörn L., Ingeval K., Martna J. and Strindell L.; 1990.**
New automatic probe for measuring triaxial stress in deep bore holes. Tunneling underground space technology, Vol. 5, pp141-145.

/3-7/ **Nilsson A.; 1997.**
Fracture mapping and cross section measurement wiht laser scanner in the TMB and ZEDEX tunnels. SKB Progress Report HRL-97-18 SKB, Stockholm, Sweden.

/4-1/ **Johannesson L.-E. & Börgesson L.; 1999.**
Compaction of bentonite blocks; Development of techniques for production of blocks with different shapes and sizes. SKB Report R-99-12.

/8-1/ **Börgesson L., Johannesson L.E. & Fredriksson A.; 1993.**
Laboratory investigations of highly compacted sand/bentonite blocks for backfilling. SKB djupförvar PR 44-93-010.

/11-1/ **Dahlin T.; 1993.**
On the automation for 2D resistivity surveying for engineering and environmental applications. Theses 1993. ISRN LUTVDG/TVGT – 1007-SE.

/11-2/ **Briscoe Ralph D.; 1984**
Thermocouple psychrometers for water potential measurements. From the proceedings of the NATA Advanced Study Insitute on “Advanced Agricultural Instrumentation”, Ciocco (Pisa), Italy, May 27-June, 1984

/12-1/ **Gunnarsson D. & Börgesson L., 1997.**
Egenskaper hos kontaktzon mellan betong och bentonit i pluggar. SKB IPR-02-21

Appendix list

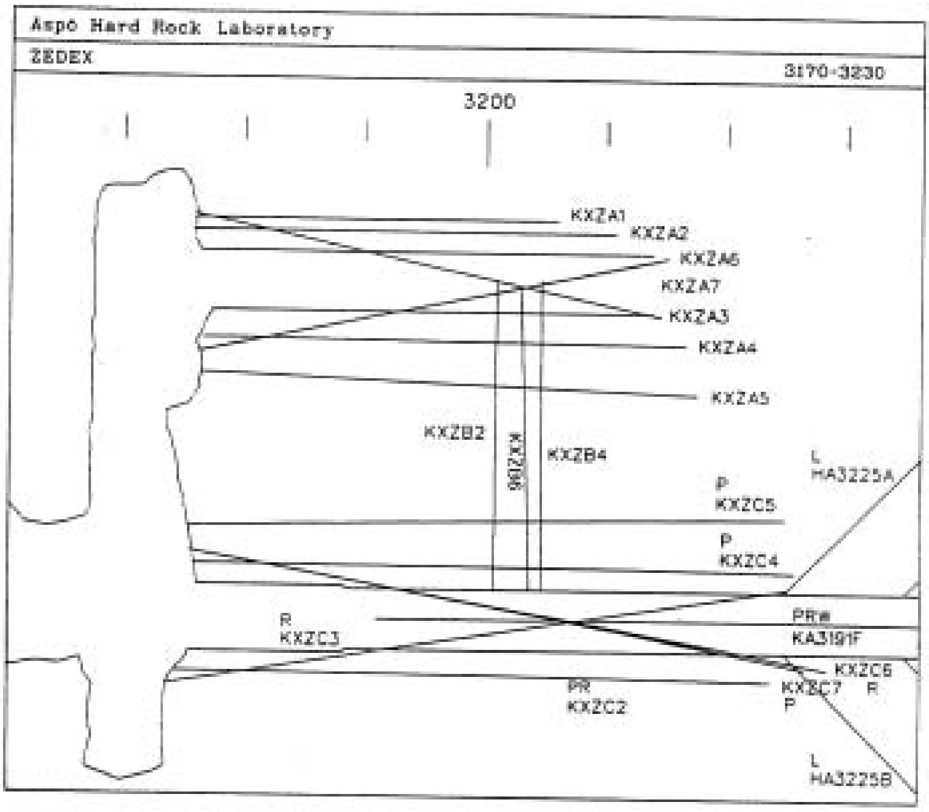
Appendix 1: List of boreholes that have been plugged.

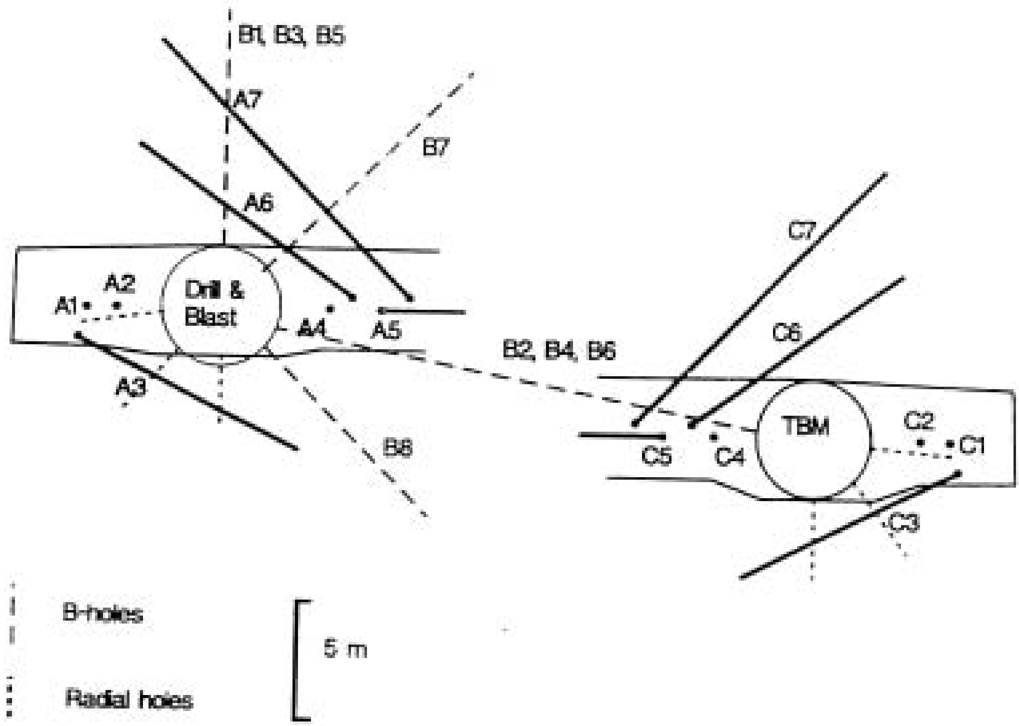
Borehole No.	Diameter	Length	Length of bentonite plug			
	(m)	(m)	(m)	56	76	86
KXZRD1H	0,086	3,15	3,15			3,15
KXZRD2I	0,086	3,14	3,14			3,14
RD2H	86		3,13			3,13
RD2V	86		3,15			3,15
KXZRD3V	0,086	3,05	3,05			3,05
KXZRD3I	0,086	3,1	3,1			3,1
KXZRD3H	0,086	3,2	3,2			3,2
KXZRD4H	0,086	3,15	3,15			3,15
KXZRD4VU	0,086	3,37	3,37			3,37
KXZRD4ILU	0,086	3,44	3,44			3,44
KXZRD4HL	0,086	3,33	3,33			3,33
KXZRD4ILD	0,086	3,39	3,39			3,39
KXZRD4VD	0,086	3,37	3,37			3,37
KXZRD4IRD	0,086	3,36	3,36			3,36
KXZRD4HR	0,086	3,35	3,35			3,35
KXZRD4IRU	0,086	3,33	3,33			3,33
KXZB1	0,056	15,15	4	4		
KXZB2	0,056	26,08	4	4		
KXZB4	0,056	26,15	4	4		
KXZB5	86	18,53	4			4
KXZB6	76	26	4		4	
KXZB7	0,076	16,1	4		4	
KXZB8	0,076	15,1	4		4	
KXZRD5H	0,086	3,1	3,1			3,1
KXZRD6H	0,086	3,1	3,1			3,1
KXZRD6V	0,086	3,19	3,19			3,19
KXZRD6I	0,086	3,15	3,15			3,15
KXZRD7I	0,086	3,05	3,05			3,05
KXZRD7A01	0,086	5,16	5,16			5,16
KXZRD7A02	0,086	5,09	5,09			5,09
KXZRD7A03	0,086	5,21	5,21			5,21
KXZRD7A04	0,086	2,42	2,42			2,42
KXZRD7A05	0,086	2,39	2,39			2,39
KXZRD7A06	0,086	2,4	2,4			2,4
KXZRD7A07	0,086	5,12	5,12			5,12
KXZRD7A08	0,086	5,08	5,08			5,08
KXZRD7A09	0,086	5	5			5
KXZRD7V	0,086	3,15	3,15			3,15
KXZRD7VD	0,086	8,08	8,08			8,08
KXZRD7ILU	0,086		8,06			8,06
KXZRD7ILD	0,086		8,02			8,02
KXZRD7IRU	0,086		8,1			8,1

Borehole No.	Diameter	Length	Length of bentonite plug			
	(m)	(m)	(m)	56	76	86
KXZRD7IRD	0,086		8,08			8,08
KXZRD7VU	0,086		8,24			8,24
KXZRD8H	0,086	3,25	3,25			3,25
KXZSD8HR	0,086	23,16	4			4
KXZRD9H	0,086	3,07	3,07			3,07
KXZSD81HR	0,086	3,91	3,91			3,91
KXZA1	0,086	30,24	30,24			30,24
KXZA2	0,056	35,14	35,14	35,14		
KXZA3	0,056	40,04	40,04	40,04		
KXZA4	0,056	40,07	40,07	40,07		
KXZA5	0,086	41,13	41,13	41,13		
KXZA6	0,056	40,08	40,08	40,08		
KXZA7	0,056	15,15	15,15	15,15		

Appendix 2: Positions of extensometer bore holes.

8



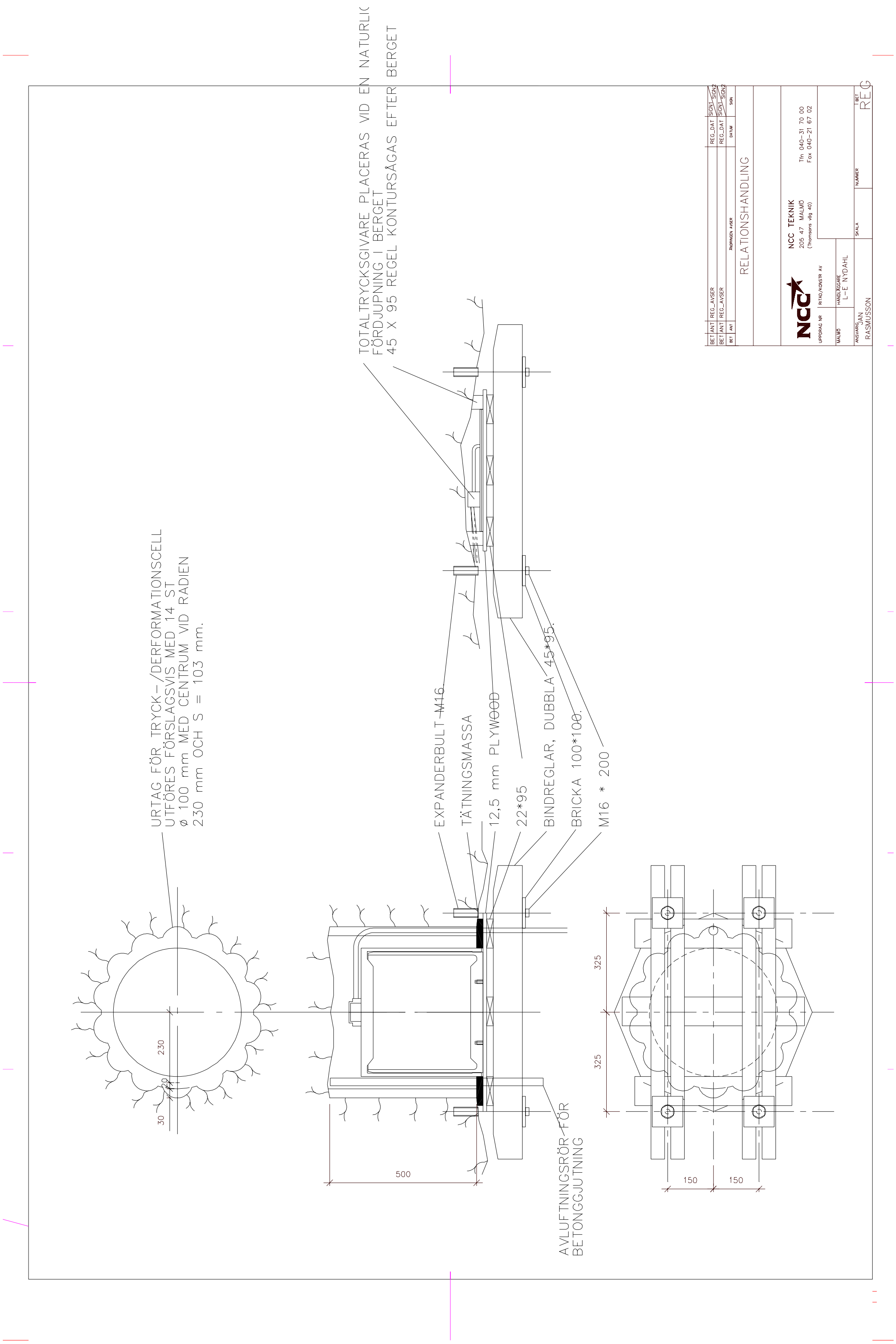


Appendix 3: bore holes for measuring water pressure

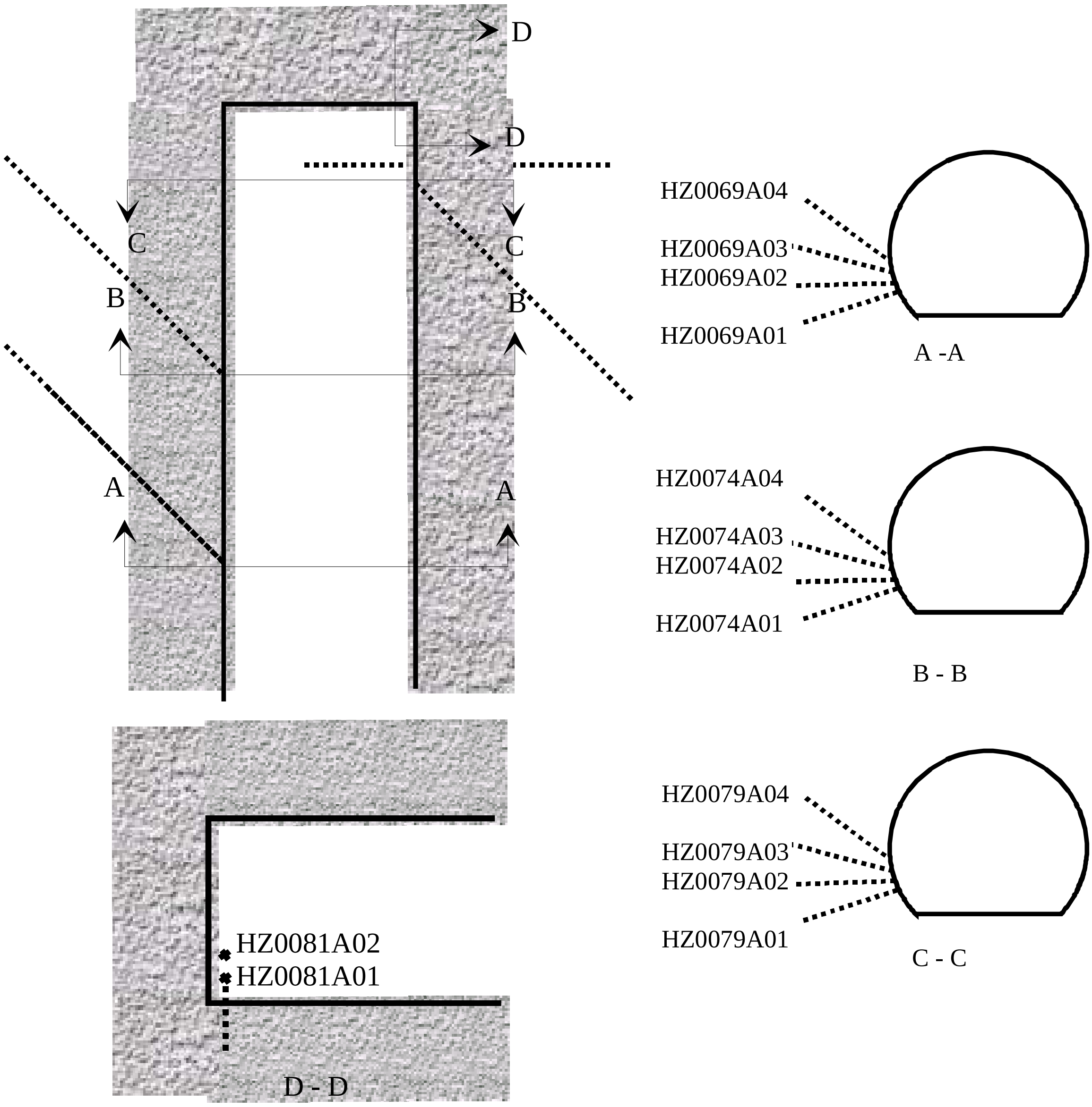
Bore holes that will be instrumented and tested in the Backfill and Plug Test:								
Name	**Relative the tunnel:**			**Diameter**	**Location:**		**Hydraulic**	**instrumen**
					distance from		**Testing**	
OK 980617	**inclinatior**	**bearing(°)**	**length(m)**	**(mm)**	**floor (m)**	**Section (m)**		
One meter bore holes:								
KZ0041I01	45	0	1	56	5	41,2	X	X
KZ0042B01	45	90	1	56	1,5	41,7	X	X
KZ0041G01	-45	0	1	56	0	41,2	X	X
KZ0042A01	45	-90	1	56	1,5	41,7	X	X
KZ0043I01	45	0	1	56	5	43,4		X
KZ0044B01	45	90	1	56	1,5	43,9		X
KZ0043G01	-45	0	1	56	0	43,4		X
KZ0044A01	45	-90	1	56	1,5	43,9		X
KZ0046I01	45	0	1	56	5	45,6	X	X
KZ0046B01	45	90	1	56	1,5	46,1	X	X
KZ0046G01	-45	0	1	56	0	45,6	X	X
KZ0046A01	45	-90	1	56	1,5	46,1	X	X
KZ0048I01	45	0	1	56	5	47,8		X
KZ0048B01	45	90	1	56	1,5	48,3		X
KZ0048G01	-45	0	1	56	0	47,8		X
KZ0048A01	45	-90	1	56	1,5	48,3		X
KZ0050I01	45	0	1	56	5	50	X	X
KZ0050B01	45	90	1	56	1,5	50,5	X	X
KZ0050G01	-45	0	1	56	0	50	X	X
KZ0050A01	45	-90	1	56	1,5	50,5	X	X
KZ0052I01	45	0	1	56	5	52,2		X
KZ0053B01	45	90	1	56	1,5	52,7		X
KZ0052G01	-45	0	1	56	0	52,2		X
KZ0053A01	45	-90	1	56	1,5	52,7		X
KZ0054I01	45	0	1	56	5	54,4	X	X
KZ0055B01	45	90	1	56	1,5	54,9	X	X
KZ0054G01	-45	0	1	56	0	54,4	X	X
KZ0055A01	45	-90	1	56	1,5	54,9	X	X
KZ0057I01	45	0	1	56	5	56,6		X
KZ0057B01	45	90	1	56	1,5	57,1		X
KZ0057G01	-45	0	1	56	0	56,6		X
KZ0057A01	45	-90	1	56	1,5	57,1		X
KZ0059I01	45	0	1	56	5	58,8	X	X
KZ0059B01	45	90	1	56	1,5	59,3	X	X
KZ0059G01	-45	0	1	56	0	58,8	X	X
KZ0059A01	45	-90	1	56	1,5	59,3	X	X
KZ0061I01	45	0	1	56	5	61		X
KZ0061B01	45	90	1	56	1,5	61,5		X
KZ0061G01	-45	0	1	56	0	61		X
KZ0061A01	45	-90	1	56	1,5	61,5		X

KZ0063I01	45	0	1	56	5	63,2	X	X
KZ0064B01	45	90	1	56	1,5	63,7	X	X
KZ0063G01	-45	0	1	56	0	63,2	X	X
KZ0064A01	45	-90	1	56	1,5	63,7	X	X
KZ0065I01	45	0	1	56	5	65,4		X
KZ0066B01	45	90	1	56	1,5	65,9		X
KZ0065G01	-45	0	1	56	0	65,4		X
KZ0066A01	45	-90	1	56	1,5	65,9		X
Five meter bore holes:								
KZ0041I02	90	-	5	56	5	40,8		X
KZ0041B02	0	90	5	56	1,5	40,8	X	X
KZ0041G02	-90	-	5	56	0	40,8	X	X
KZ0041A02	5	-90	5	56	1,5	40,8	X	X
KZ0065I02	90	-	5	56	5	64,8		X
KZ0065B02	5	90	5	56	1,5	64,8	X	X
KZ0065G02	-90	-	5	56	0	64,8	X	X
KZ0065A02	5	-90	5	56	1,5	64,8	X	X
KXZRD7H	*0*	90	3,2	86			X	X
KXZRD7HR	*0*	-90	8,09	86			X	X
25 meter bore holes								
KZ0055I01	90	-	25	56		55		X
KZ0082F01	0	0	25	56	1,5	81,8		X
KXZSD8HL	*5*	-95	25,98	86			X	X
KXZSD8HR	*-5*	85	23,16	86			X	X
KXZB3	-90	0	15,2	56			X	X
Bore holes along the tunnel axel								
KXZA1	-1		30,24	86		X		
KXZA2	-1,2		35,14	56		X		
KXZA6	-8,9		ca. 40	56		X		

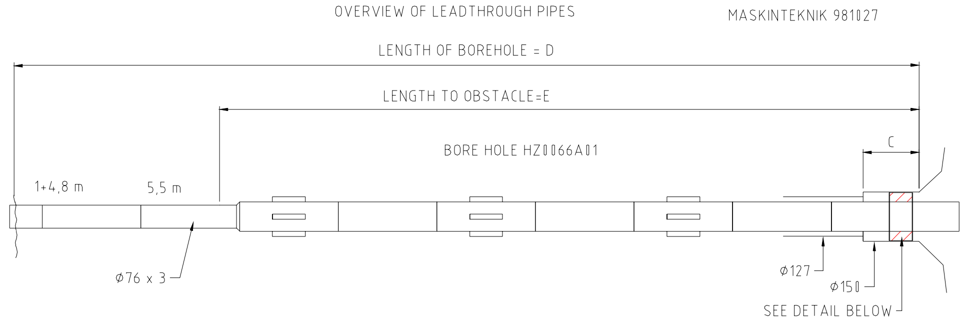
Appendix 4: Drawing of arrangements for the mounting of the pressure cylinders

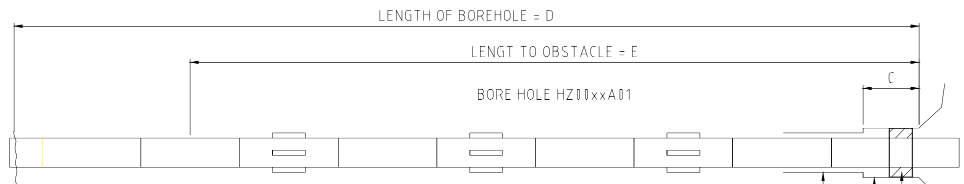

TOTALTRYCKSGIVARE PLACERAS VID EN NATURLI
FÖRDJUPNING I BERGET
45 X 95 REGEL KONTURSÅGAS EFTER BERGET
URTAG FÖR TRYCK-/DERFORMATIONSCELL
UTFÖRES FÖRSLAGSVIS MED 14 ST
ø 100 mm MED CENTRUM VID RADIEN
230 mm OCH S = 103 mm.
EXPANDERBULT M16
TÄTNINGSMASSA
12,5 mm PLYWOOD
22*95
BINDREGLAR, DUBBLA 45*95.
BRICKA 100*100.
M16 * 200
AVLUFTNINGSRÖR FÖR
BETONGGJUTNING
230
30
20
500
325
325
150
150
RELATIONSHANDLING
NCC TEKNIK
205 47 MALMÖ
(Thomsons väg 40)
Tfn 040-31 70 00
Fax 040-21 67 02
MALMÖ
L-E NYDAHL
JAN
RASMUSSON

Appendix 5 Positions of bore holes used for drainage

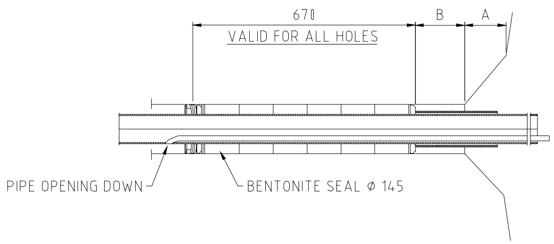


Appendix 6: Specifications of lead through pipes.

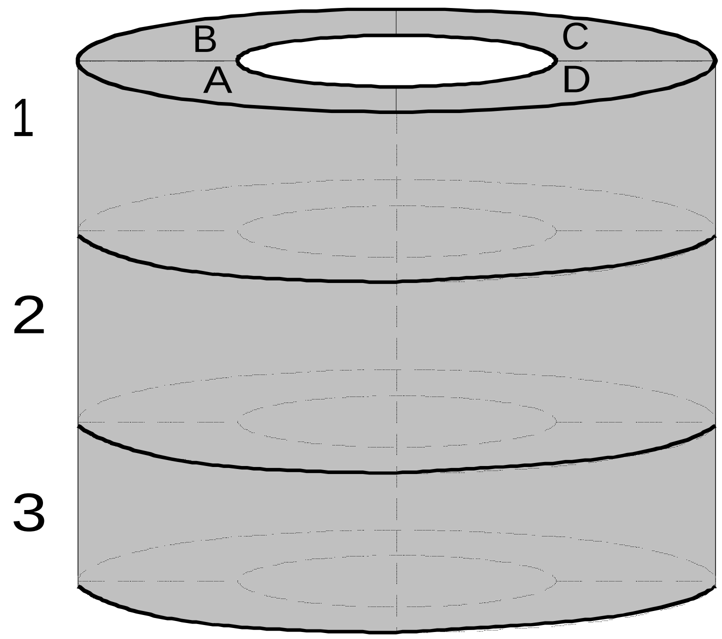




	A [m]	B [m]	C [m]	D [m]	E [m] STOP WHEN CHECKING HOLES RING SHOULDER PIPE ø120	ø120	ø99	
HZ004A01	-	0,25	1,30	22,10				
HZ0043A01	0,60	0,14	2,75	23,90				
HZ0045A01	0,53	0,30	2,40	27,40				
HZ0047A01	0,70	0,57	2,87	28,05	20,00	20,00		
HZ0049A01	0,50	0,45	2,85	29,85				
HZ0051A01	0,57	0,40	2,90	30,15	17,70	17,95		
HZ0053A01	0,50	0,17	2,55	30,35				
HZ0054A01	0,55	0,35	2,60	30,35				
HZ0056A01	0,60	0,55	2,70	31,05	17,60			
HZ0058A01	0,58	0,40	2,60	31,90	12,35			
HZ0060A01	0,60	0,20	2,62	32,75	27,00			
HZ0066A01	0,60	0,25	2,70	33,50	9,00	22,75	24,00	SEE PICTURE



Appendix 7: Variation of density in one lead-through block.



Densitet

Sample	ρ g/cm³	Sr %	e	ρ_s g/cm³
A1	2,073	59,2	0,477	2,780
A2	2,070	58,9	0,479	2,780
A3	2,100	61,6	0,458	2,780
B1	2,061	58,1	0,486	2,780
B2	2,063	58,3	0,485	2,780
B3	2,087	60,4	0,467	2,780
C1	2,063	58,3	0,484	2,780
C2	2,066	58,5	0,483	2,780
C3	2,091	60,8	0,464	2,780
D1	2,071	59,0	0,479	2,780
D2	2,072	59,1	0,478	2,780
D3	2,093	61,0	0,463	2,780
Average	2,076			

Appendix 8: Local permeability measuring system

José L. Garcia-Sineriz; Ignacio Barcéna; J.L. Fuentes-Cantillana
AITEMIN

LOCAL PERMEABILITY MEASUREMENTS IN A SELECTED ZONE OF THE BACKFILL

The purpose was to develop and test a dynamic pore water pressure sensor based on the piezocone principle, for the direct measurement of local saturated permeability in the backfill.

The dynamic pore pressure sensors and the measurement system required to control the sensors and to perform the pulse tests, were installed in 1999. The sensors were installed in section A4 of the backfill, and preferably in the areas where a higher density gradient may be expected (i.e. rock proximity), in order to measure hydraulic conductivity when saturated. In this way, a map of local permeability values will be obtained and will be compared with the global value estimated by backanalysis from the flow test in saturated conditions.

Once saturation is reached, a pulse pore water pressure will be applied, and the corresponding dissipation time will be measured. There is a relation between soil permeability and the shape of this dissipation curve.

1.1. The dynamic pore pressure sensors

A dynamic pore pressure (DPP) sensor is a specially constructed hydraulic piezometer, with a cylindrical ceramic filter of 60 microns pore size, and including a miniature pressure sensor inside. Figure 1 shows the DPP sensor configuration. Each piezometer has two metallic capillary tubes for water input and output, and an electrical cable for the pressure transducer signal.

The DPP sensors work in the same way as the "piezocone" testing method: A controlled positive pressure pulse will be applied to the sensors, and the evolution of the pressure drop in the sensor body, which is controlled by the local permeability of the surrounding material, will be analysed.

According to the initial calculations made by UPC, the compressibility of the water existing in the measuring circuit is a very sensible parameter, provided that the mechanical components (tanks, pipes, ...) are sufficiently rigid. As the expected range of the permeability to be measured may be very wide (from 10-8 to 10-11 m/s), the system has been designed so that the internal volume of the measuring circuit may be easily modified. Also the possibility of measuring the volume (flow) of water transfer to the backfill during the pulse test has been included in the system design, for the case of very permeable media.

1.2. Additional equipment and system description

The complete system comprises a number of DPP sensors (13 units), and a common measuring system, which is located outside the backfill area.

The measuring and control system performs the following three basic functions:

- Flushing and de-airing of the hydraulic circuit of each DPP sensor.
- Pressure pulse generation and control.
- Recording of the pressure variation at the DPP sensors.

The hydraulic/electric control system scheme is shown in Figure 2. The two hydraulic tubes of all the DPP sensors are connected to electric valves in a circuit-switching panel, so that only one sensor circuit is connected to the measuring system at any one time.

The data acquisition and control unit (DAC) controls the switching panel, which actuate the appropriate valves in the system, according to manually input commands. Electrical signals from all the DPP sensors are permanently connected to the DAC unit for data recording and storage.

The measuring system includes two basic hydraulic circuits:

1.The primary circuit, using de-aired Äspö water, which is the one to be actually circulated through the sensor circuit.
2.A secondary circuit, which uses compressed Nitrogen gas, used for pressure transmission and flow control purposes. This circuit does not mix with the primary.

The component of the measuring system called transfer, is a tank with a balloon inside, which is used as a pressure exchanger to apply a constant pressure pulse into the (primary) sensor circuit.

Other components of the system are:

- Return tank used for the storage of the water recirculated from the sensors.
- A vacuum pump for de-airing the Äspö salt water to be used in the primary circuits and to remove air from the primary circuits if necessary.
- A bottle of compressed Nitrogen, to generate the positive pressure.
- Auxiliary high speed solenoid valves, to control pulse generation.
- Three auxiliary tanks, one of 10 dm3 and two of 50 dm3 for changing the internal volume of the DPP sensor hydraulic circuit, as required by the test conditions.

The compressibility of the water in the measuring circuit is a relevant parameter for this type of test, and therefore the volume of water in this circuit must be reduced to the minimum required by the test. The volume of water estimated for the circuit (with 60 m long conduits) is about 1 l. As the range of permeability, which may be expected during the test, is very extensive (from about 10-8 to 10-11 m/s), it becomes necessary to increase the internal volume of the measuring circuit for the higher range of permeability. This will be accomplished by introducing in the primary circuit an auxiliary tank (designated as volume control tanks in Figure 2). The volume relations of these tanks are equivalent to the expected changes in permeability (2 orders of magnitude, 1/10/100).

However, the possibility exists that the permeability in the backfill would still be too high to be measured by a system such as the pulse test proposed. In this case, the measurement could be carried out by controlling the total water inflow into the backfill during the test. To enable this option, the system is equipped with a high accuracy flow meter.

1.3. Test procedure.

The initial situation for the operation of the system is the following:

- The return tank is almost empty of water.
- The transfer is full of de-aired salt water.

The test procedure for each DPP sensor is as follows:

1.Flushing of DPP sensor hydraulic circuit. The purpose of this operation is to completely fill the hydraulic circuit of the sensor (primary circuit), removing all the air that may exist in it. For this, the valve connecting the transfer and the circuit is opened and the input and output valves of the corresponding DPP sensor are opened. Sufficient pressure is applied to the transfer´s balloon by means of the bottle of compressed Nitrogen and a manual pressure regulator. Then this pressure is transmitted to the water inside the transfer, thus flushing salt water through the sensor circuit up to the return tank. The flush flow should be low enough to see the air bubbles in the circulated water in the return tank, which is made of transparent plastic. Salt-water circulation is stopped (closing the input and output valves of the DPP sensor) when no bubbles are observed at the return tank. It is estimated that a volume of water of about 6-7 times the circuit volume has to be circulated to remove all the air entrapped in the conduits, tanks, and sensor.

2.Pressure pulse generation. A pressure equal to the static pore pressure observed at the DPP sensor plus around 2-3 bars will be applied to the balloon by means of the compressed Nitrogen regulator, and this pressure will also be transmitted to the water inside the transfer. The input valve of the corresponding DPP sensor is then opened, the output valve being kept closed. A high-speed valve placed between the transfer and the DPP sensor input valve is opened and closed very quickly, in order to transmit a controlled pressure pulse to the DPP sensor. The evolution of the pressure at the piezometer during and after the pulse is measured by the pressure sensor installed in the DPP sensor and recorded by the data acquisition and control system (DACS).

In principle, the entire measurement sequence is carried out manually, although some of the operations are automated (specially valves control) to simplify the process, making it more accurate and repetitive, and avoid disoperation. Data is recorded automatically.

1.4. System layout

The system layout is shown in Figure 3.

All cables and tubing from DPP sensors have been taken from the ZEDEX to the Demonstration drift through a dedicated pipe inside a borehole drilled for this purpose. The hydraulic isolation between backfill and the pipe is performed by cable and tubing glands installed on a metallic flange at the ZEDEX drift end of the pipe.

Tubing from DPP are connected to circuit switch valves box at Demonstration drift, from where only two tubings connect with the measuring and control system, which is placed some 40 m away at the control room. The valve switch system makes that only one sensor circuit is connected to the measuring system at any one time.

All cables from DPP are connected to the measuring and control system by a multiwire cable using an electrical junction box placed also at the Demonstration drift.

The measuring and control system is composed by all the hydraulic components necessary for the DPP operation (tanks, transfer, vacuum pump, auxiliary valves, flow meter, etc.), which have been integrated inside a cabinet called “hydraulic panel”, and all the electric components (computer, data acquisition boards and interfaces,...) integrated inside a cabinet called “data acquisition system”.

1.5. Field installation

The sensors, as well as, the whole system were finally installed during March 1999.

The total number of sensors installed in A4 section of the 30/70 backfill is thirteen, and the final measuring points as well as their tubes and cables situation are shown in Figure 4. The location of some of the initial measuring points were changed during the installation due to:

- The risk of damaging other sensors installed in the previous compacted layer.
- The impossibility of drilling more that one compacted layer.

To reduce mechanical damage to the DPP sensors, these were installed in their positions after the corresponding 20 cm thick layer had been compacted, manually drilling or excavating a well-formed hole for DPP sensor insertion. Special precautions were taken to keep the walls of these excavations as uniform as possible, to avoid a low-density space around the DPPs.

Sensors horizontally installed in the layer were covered with a metallic perforated steel tube to protect them from any damage when compacting the next layer on top. For the same reason and once the sensors of one layer were installed, a channel for cables and tubes was dug from each sensor to the cable pass-through flange (see Figures 5 & 6).

The circuit switch valves box and the electrical junction box were located in the Demonstration drift near the pass-through pipe that connects this drift with the ZEDEX drift (see Figure 7).

The measuring and control system was situated in the control niche, in front of the Demonstration drift in the main gallery (see Figure 8). Part of the system was located inside the control room:

- The cabinets called “hydraulic panel” and “data acquisition system”
- The return tank.
- The compressed nitrogen bottle.
- The electrical distribution box.

The rest of the system is fixed to the metallic structure and located out of the control room (Figure 7).

The measuring and control system was started and adjusted to comply with all the system requirements. The acquisition and control software was developed under Windows 95 and it is based on commercial SCADA software called FIX DMACS from Intellution Inc. (USA).

The application includes a database where sensors readings are being stored. This software enables to precisely monitor and control the test, as well as to collect, deliver, and display data in graphical formats. As an example, a control screen is shown in Figure 9. Measurements from all sensors are being stored in the database every 10 minutes, except when a test pulse will be performed, in which case readings will be stored every second.

Nowadays, the system is working properly and sensors data are recorded to see the evolution of the pressure in the backfill.

The system is linked with AITEMIN´s office in Madrid by a standard telephone line. Data transmission between the system located in the Äspö laboratory and the AITEMIN´s office (Madrid), sensors measurements checking and control system are carried out through the telephone network, using modems (see Figure 10).

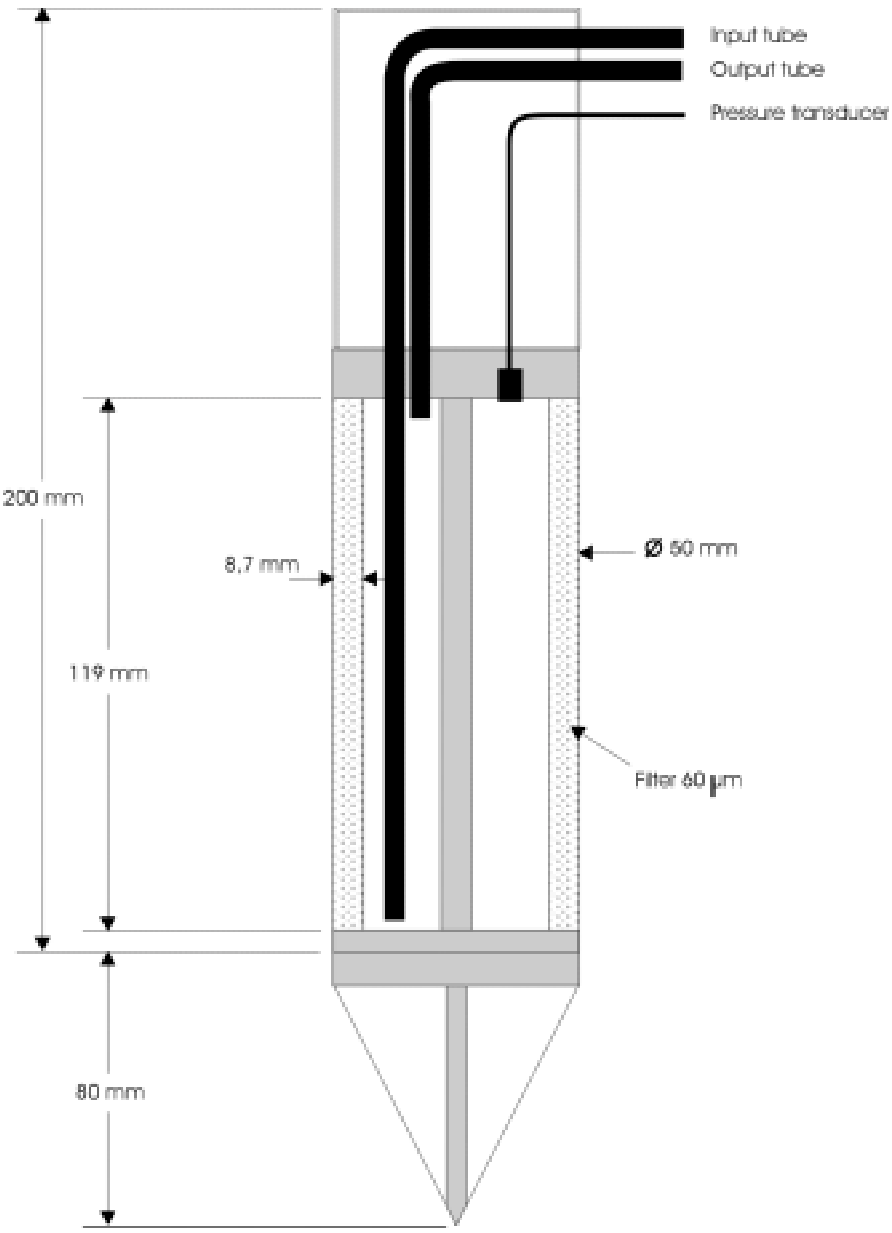


Figure1: DPP sensor description.

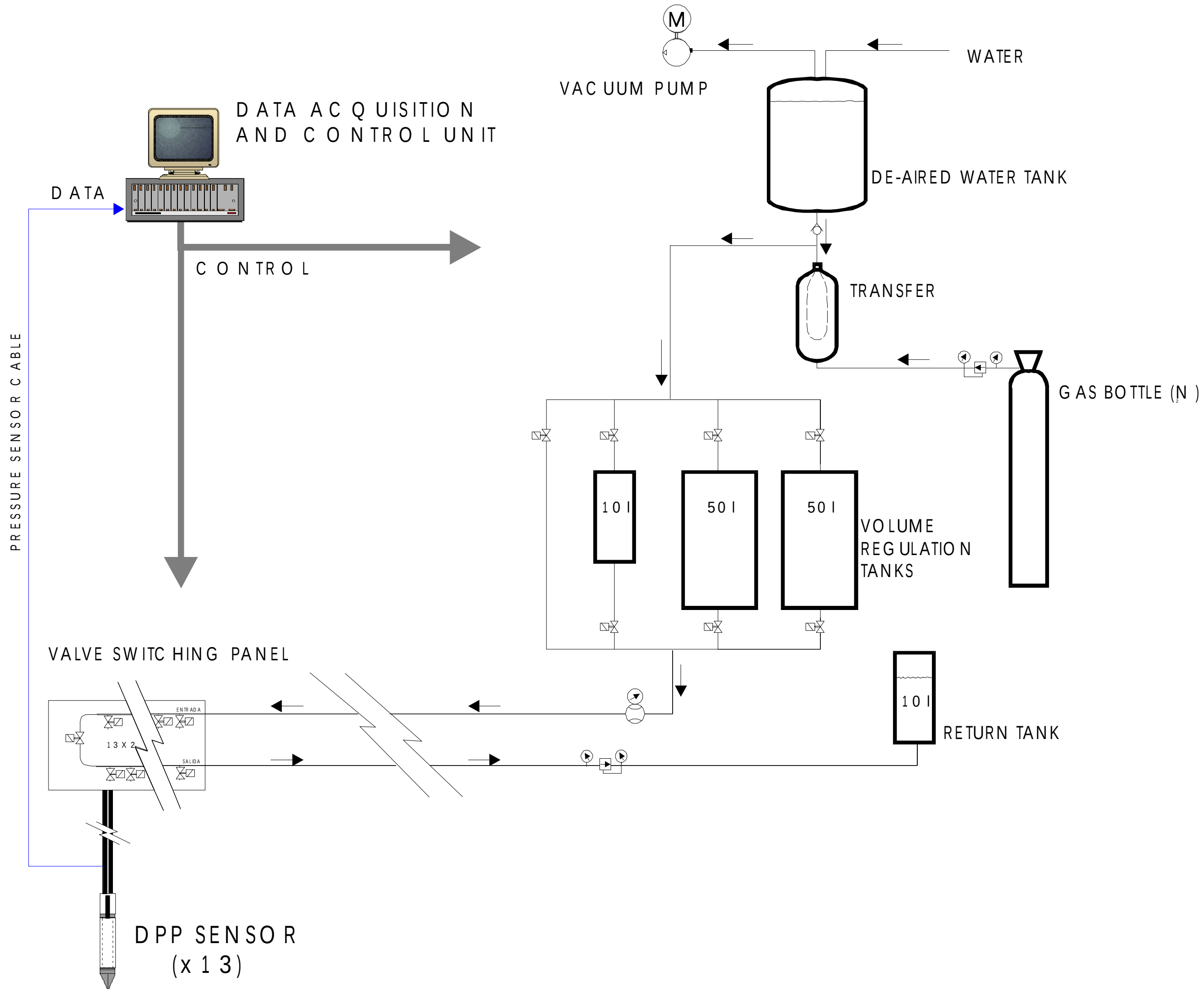
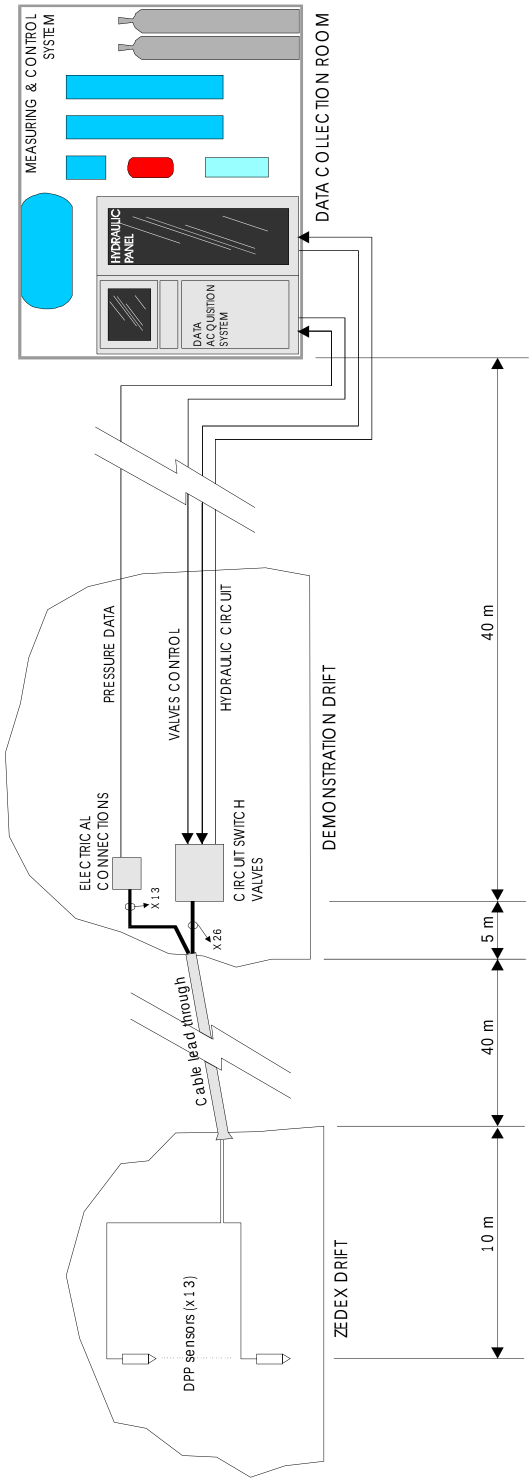
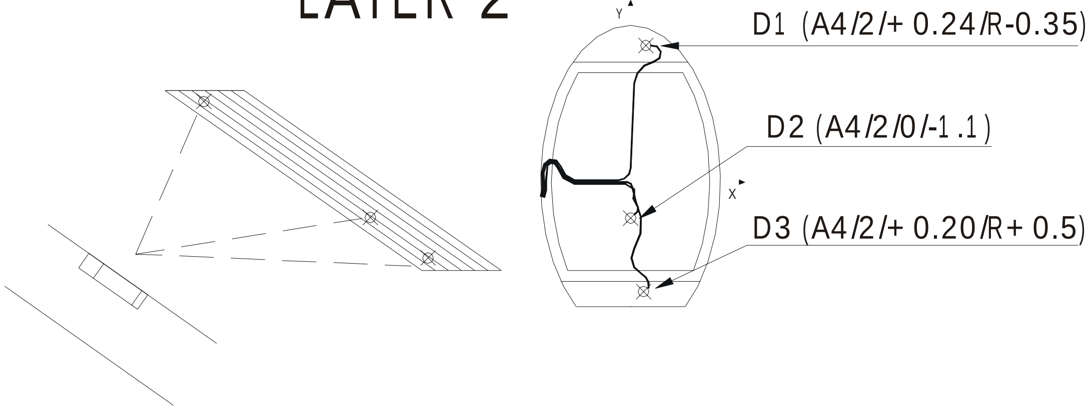


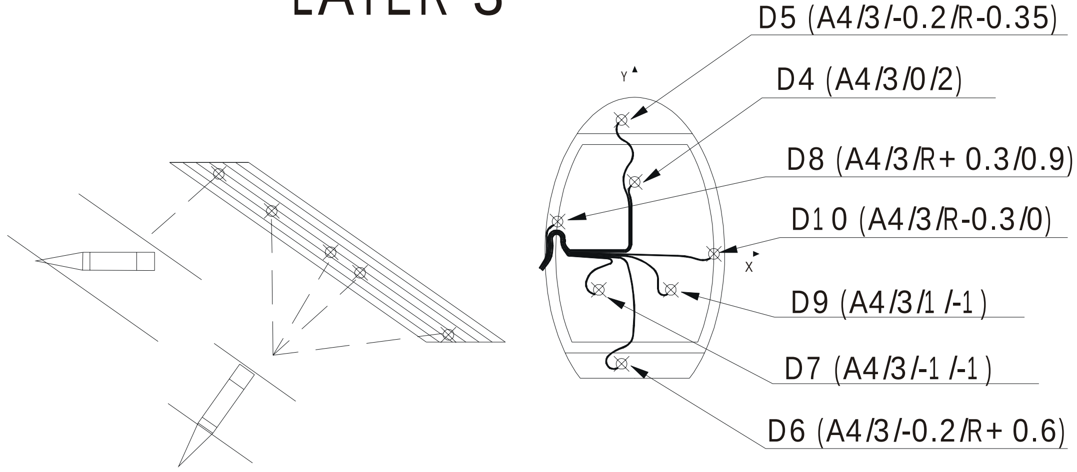
Figure 2: Control system scheme.


MEASURING & CONTROL SYSTEM
HYDRAULIC PANEL
DATA ACQUISITION SYSTEM
DATA COLLECTION ROOM
PRESSURE DATA
VALVES CONTROL
HYDRAULIC CIRCUIT
ELECTRICAL CONNECTIONS
x 13
CIRCUIT SWITCH VALVES
x 26
DEMONSTRATION DRIFT
40 m
5 m
Cable lead through
40 m
DPP sensors (x13)
ZEDEX DRIFT
10 m

LAYER 2



LAYER 3



LAYER 4

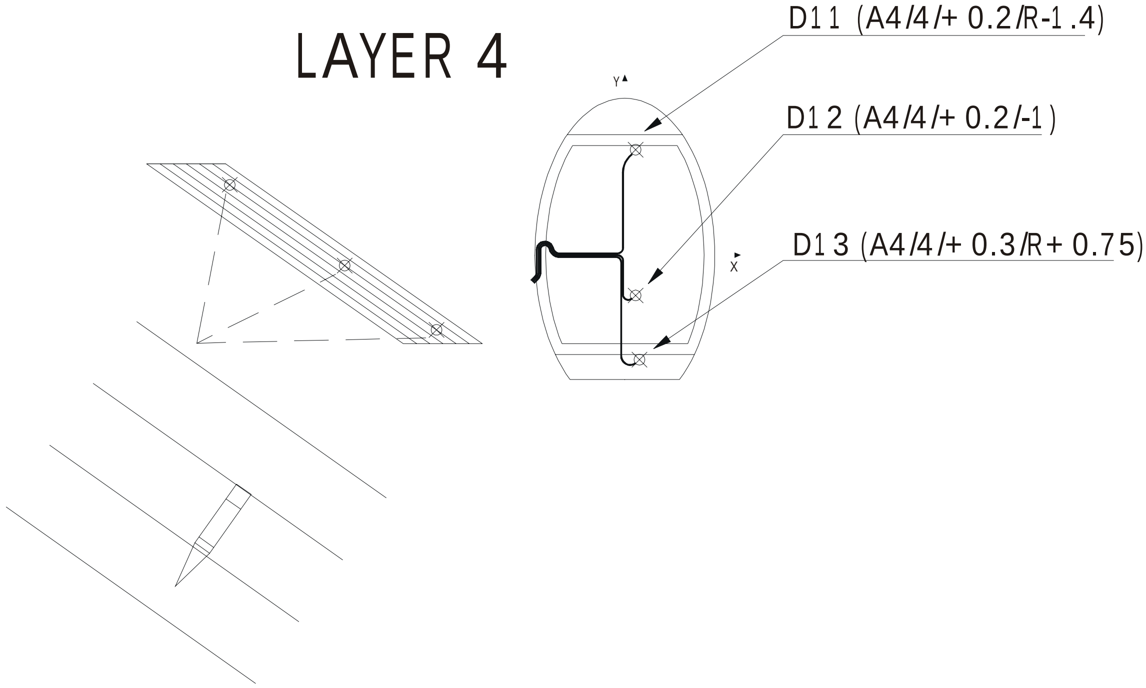


Figure 5: DPP sensor installed perpendicular to layer

Figure 6 DPP sensor installed parallel to layer.

Figure 7: Valves box (left) and junction box at Demonstration drift.

Figure 8: Measuring and control system (metallic structure outside control room).

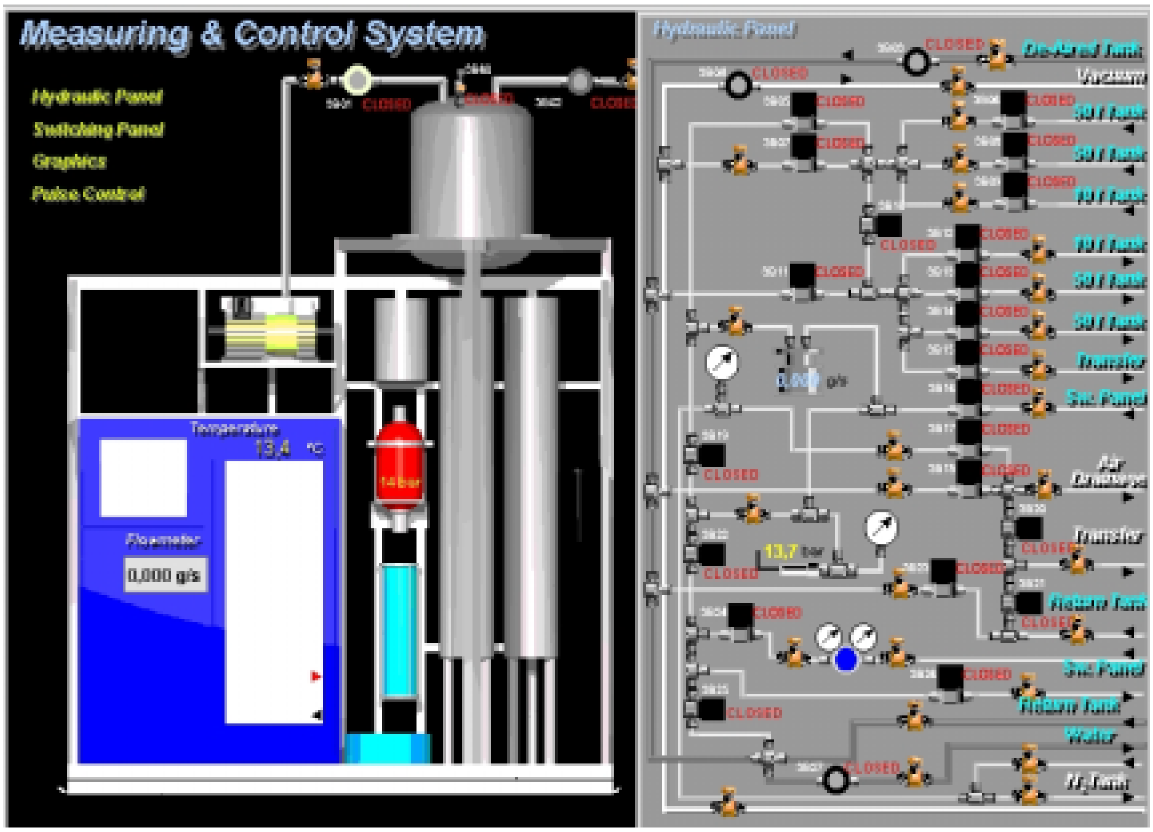


Figure 9: Example of control screen.

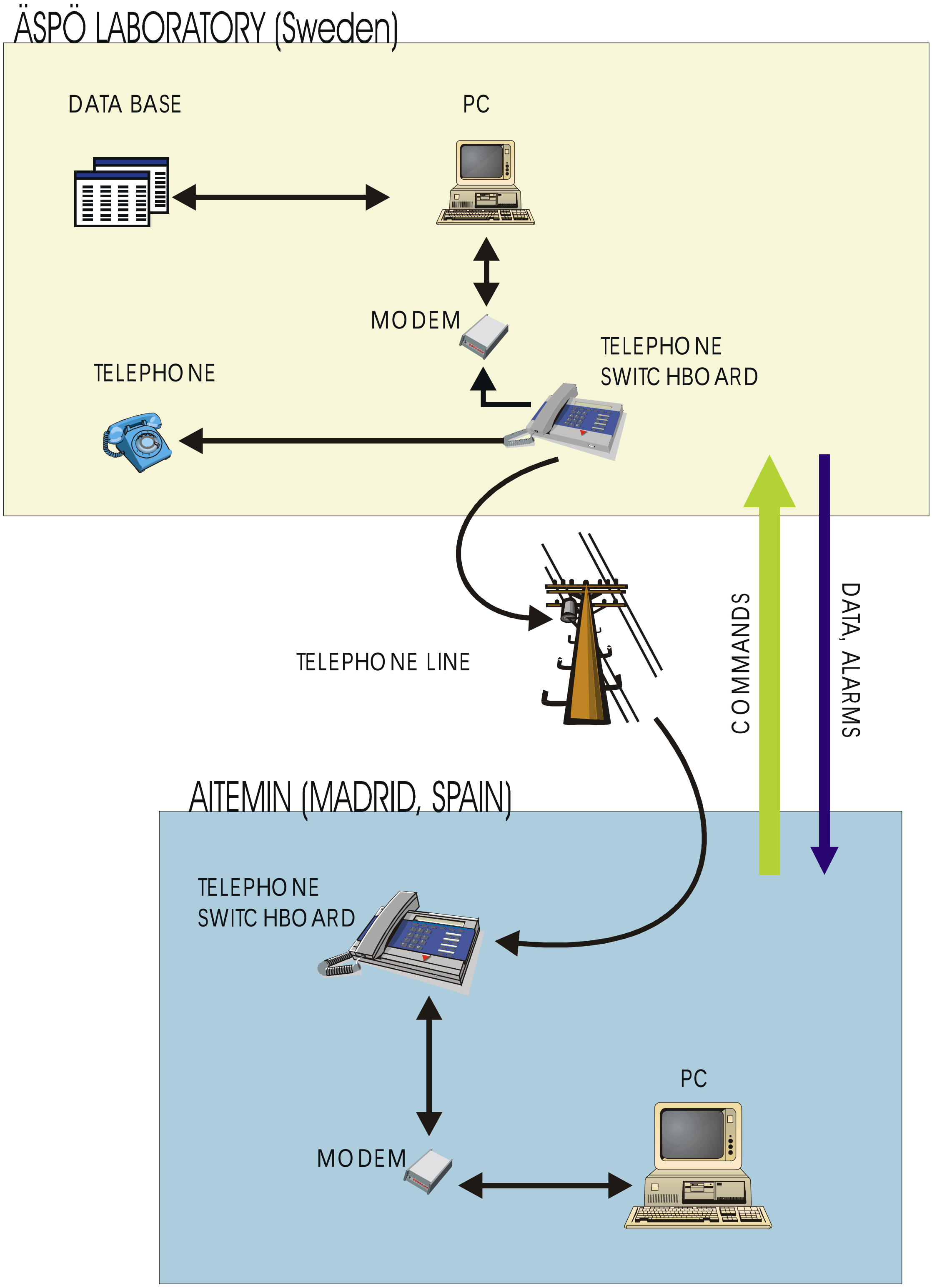
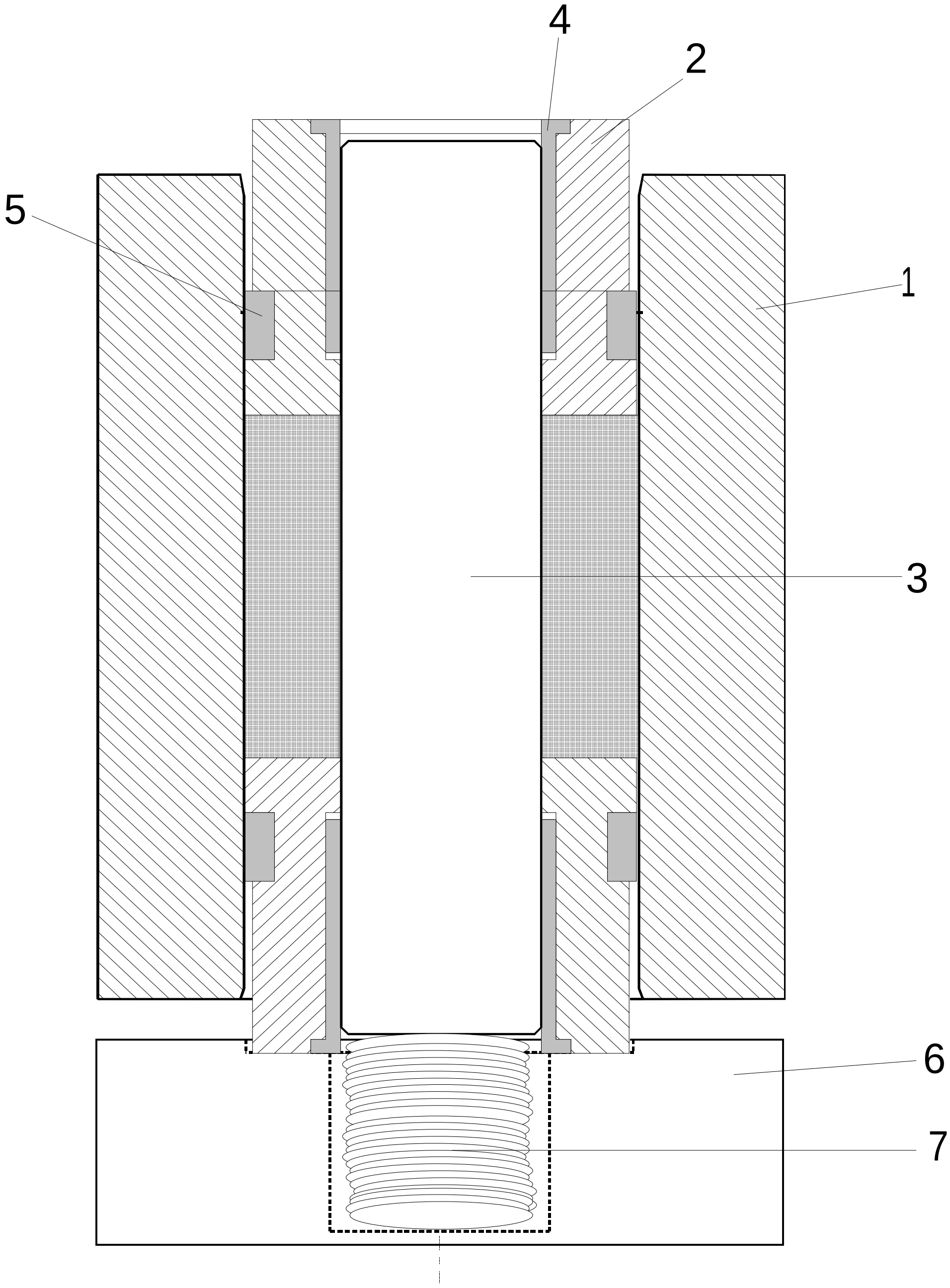


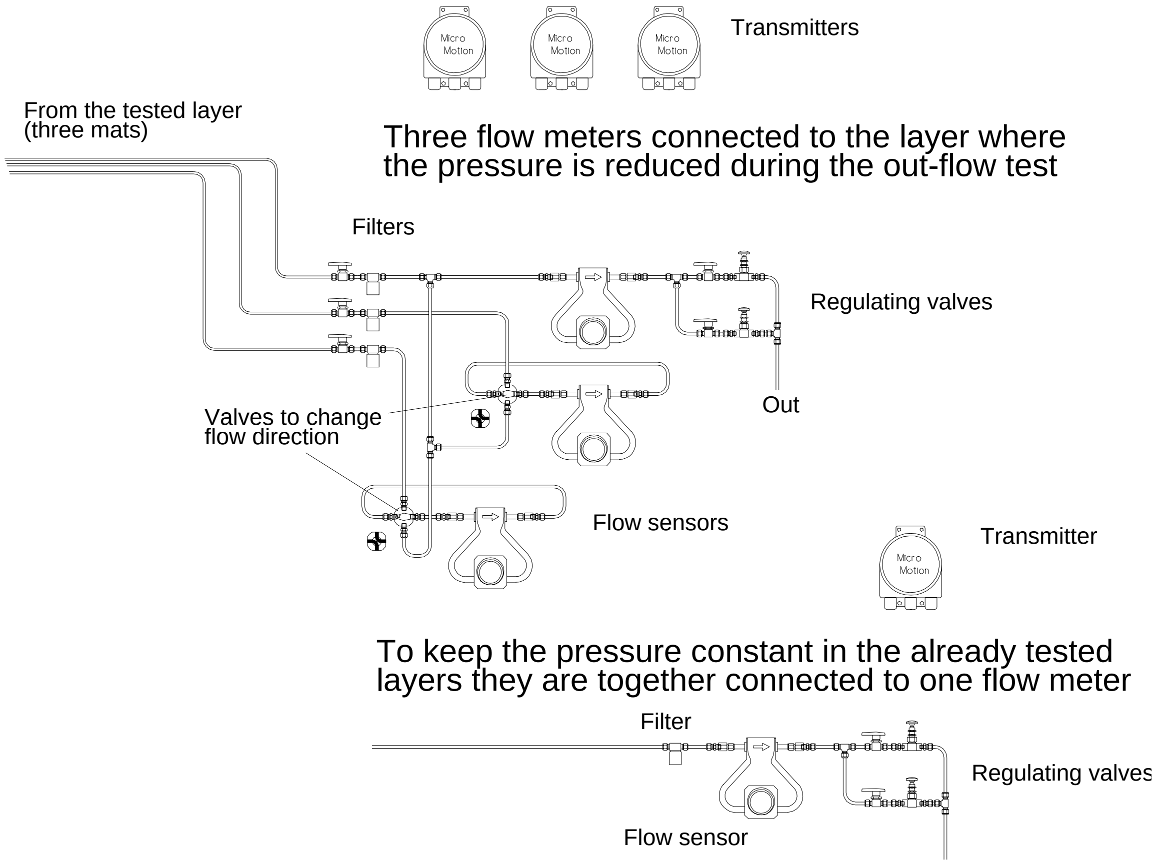
Figure 10 Communication system.

Appendix 9: Compaction form for the bentonite rings of the packers

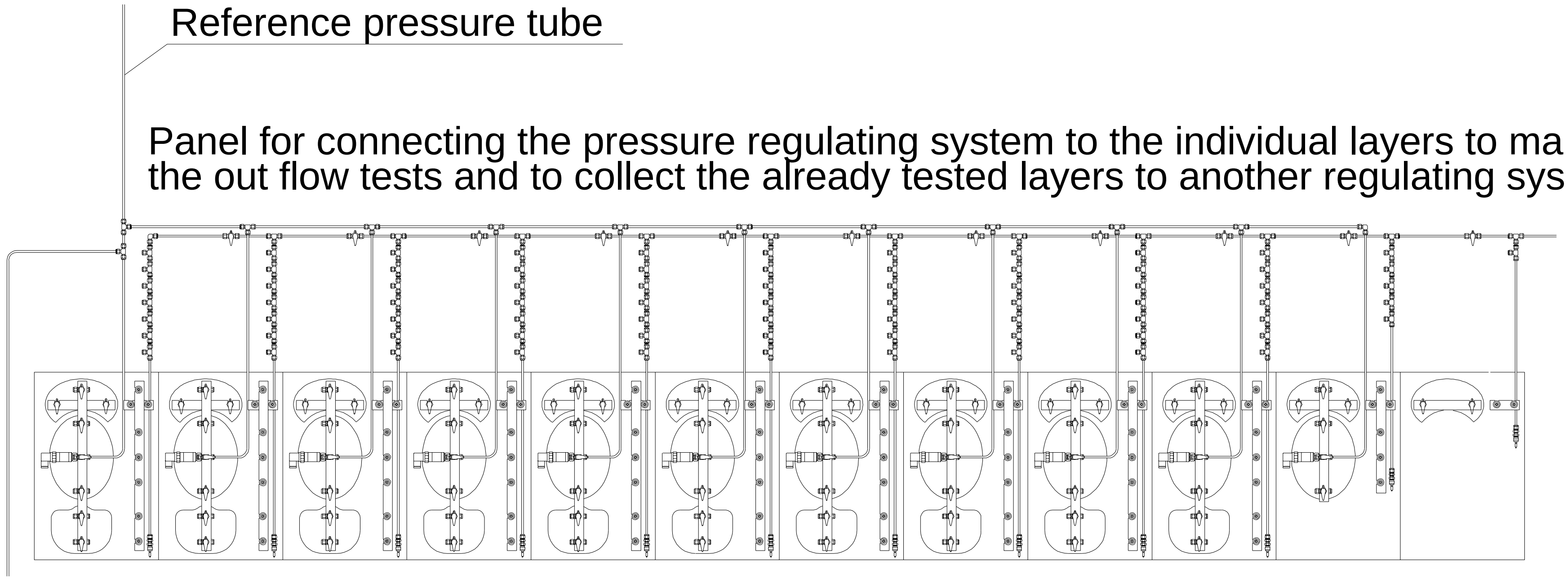


Nr	Quantity	Name for	Material
1	1	Form	SS 2541-03
2	2	Cylindrical piston	SS 2541-03
3	1	Inner piston	SS 2541-03
4	2	Casing	Brass
5	2	Steering	Brass
6	1	Bottom plate	SS 2541-03
7	1	Spring	

Appendix 10: Drawing of the flow / pressure regulating unit



Appendix 11: Control panel for the permeable mat control



Appendix 12: Micro-organisms

Microbiological investigations of migration, survival and activity of microorganisms in backfill material – Installation

Karsten Pedersen

June 2000

MICROBIOLOGICAL INVESTIGATIONS OF MIGRATION, SURVIVAL AND ACTIVITY OF MICROORGANISMS IN BACKFILL MATERIAL –

CONTENTS

Background

The backfill experiment is a full size copy of a repository with a backfilled tunnel and an ending plug. It aims at testing different backfill materials and techniques for backfilling and plugging, and studying the integrated function of rock, backfill and plugs. A backfill material will not be sterile and microbial activity is expected. The types of microorganisms that will inhabit a backfill are unknown, as is their possible degree of activity. Two scenarios or combinations of these scenarios are plausible:

1. Microorganisms present in *the bentonite and crushed rock* will multiply grow and consume organic components and available oxygen from the backfill material.
2. Microorganisms present in *the infiltrating groundwater* will multiply grow and consume organic components and available oxygen from the backfill material.

Today's limited knowledge about the microbiology of backfill does not foresee that microorganisms there should cause severe problems. Rather, it can be expected that they efficiently will remove oxygen from the backfill and contribute to achieving a low redox potential. The water intrusion into the backfill in this experiment is forced, and the backfill experiment is, therefore, atypical for the situation that will develop in a repository. Still, valuable information about microorganisms that are relevant for repository conditions can be expected, due to the very effective adaptability of microbial populations. There is the possibility that microorganisms may produce gases such as hydrogen sulphide and methane, and that fungi may develop in the backfill. Fungi are common in Äspö groundwater and in bentonite. They produce acids and strong complexing agents and should, therefore, be under observation. There may be more microbiological effects that are discovered during the experiment, which can be explored during the course of the experiment.

Experimental concept

Two main types of experiments are conducted:

1. Presence and activity of microorganisms that was embedded with the bentonite and crushed rock at start of the backfill experiment. Naturally occurring and introduced microorganisms will be investigated.
2. Presence and activity of microorganisms that have migrated with the saturating water into the backfill.

This concept requires good baseline data and viable counts of aerobic and anaerobic bacteria were therefore conducted at start of the experiment. Some of the bacteria that have been demonstrated to survive in bentonite during the LOT experiment were mixed with backfill. Activity experiments with radiotracer will be performed at end of the experiment.

The experiment consists of two phases. Phase 1 at start of the experiment and phase 2 that will be conducted at excavation. Test areas were selected for two backfill types and studied. After back filling of the chosen layer, samples were collected and analysed for naturally occurring microorganisms. The excavated parts (approximately 1 litre per part) were replaced by backfill material that was added with eight different microorganisms from lab cultures (Table 1).

Table 1. Bacteria with the following character were introduced in the backfill sections 1 and 4.

Microorganism	**Characteristics**
<u>Anaerobic bacteria</u>	
Desulfovibrio aspoeensis (DSM 10631)	Sulphate reducing bacterium
Desulfomicrobium baculatum	Sulphate reducing bacterium, from Äspö
Desulfomaculum nigrificans (DSM 574)	Spore-forming, sulphate reducing bacterium
<u>Aerobic bacteria</u>	
Deinococcus radiophilus (DSM 20551)	Desiccation and radiation resistant bacterium
Pseudomonas stutzeri (CCUG 36965)	Bentonite dweller
Bacillus subtilis (CCUG 163)	Spore forming bacterium
Bacillus sp., isolated from benonite (CCUG 36961)	Bentonite dweller

Installation procedure and results

Preparation of microorganisms

1. Four 500 ml bottles were prepared at the laboratory in Göteborg with growing cultures of each microorganism in table 1 and transported to Äspö HRL.
2. The number of viable cells in each culture was determined with plate count or most probable number count techniques before transport.

Mixing of microorganisms with the backfill

1. Two backfill sections were selected, section 1 with 100% crushed rock and section 4 with bentonite/crushed rock in a 30/70% mixture.
2. Seven holes with the volume of 1 litre were prepared in each section.
3. The bottom of the holes was covered by a sheet of plastic ensuring that the introduced microbes did not migrate during the mixing procedure.
4. Approximately 500 ml of each culture were poured and mixed with the removed bentonite in each hole. They were installed according to table 2.
5. The edge of the holes was marked with a titanium wire.
6. The top of the holes was additionally marked with yellow plastic. The bordering section was also supplied with a yellow plastic to announce the presence of the holes in good time for the sampling procedure.

Table 2. Installation of microbes in backfill sections A5 L4 and B2 L5. Holes are numbered from left to right, facing inwards in the tunnel. The number of viable cells at installation is also given, as determined by with plate count or most probable number count techniques.

Microorganism	Hole number	Viable cells ml^{-1} culture
Anaerobic bacteria		
Desulfovibrio aspoeensis	1	3.5×10^7
Desulfomicrobium baculatum	2	7.0×10^8
Desulfomaculum nigrificans	3	8.2×10^8
Aerobic bacteria		
Bacillus subtilis	4	4.5×10^8
Pseudomonas stutzeri	5	9.0×10^9
Bacillus sp., isolated from benonite	6	3.1×10^3
Deinococcus radiophilus	7	5.0×10^8

Analysis of naturally present organisms

1. Approximately 40 g samples of 30/70% and 0/100% (bentonite/rock) backfill from the installation sites were transported to the laboratory in Göteborg.

2. Most probable number determinations for sulphate reducing bacteria were performed with SRB and E media and inoculated at 30 and 55 °C. Plate count determinations of aerobic bacteria were performed on Nutrient broth agar inoculated at 30 and 55 °C. The composition of the medium E is described in: “Postgate, J.R. (1984) The Sulphate-Reducing Bacteria. Cambridge, UK: Cambridge University Press”. The SRB medium used is described in: “Widdel, F. and Bak, F. (1992) Gram-negative mesophilic sulfate-reducing bacteria. In *The Prokaryotes.* Balows, A., Trüper, H.G., Dworkin, M., Harder, W. and Schleifer, K.H., eds., pp 3352–78. New York, NY: Springer-Verlag”.
3. Aerobic and anaerobic bacteria were analysed and identified. The results are presented in Table 3.

Table 3. Bacteria isolated from backfill material in section 1 and 4. The isolates are deposited at the Culture Collection of University of Göteborg (CCUG).

Microorganism	**Characteristics**
<u>0/100 % bentonite/crusched rock (section 1)</u>	
Sulfate reducing bacteria	Cultured at 30 °C in SRB medium
Sulfate reducing bacteria	Cultured at 30 °C in E medium
Aeromonas encheleia (CCUG 42252)	Cultured at 30 °C in nutrient broth medium
Bacillus thermophilic (CCUG42250-51)	Cultured at 55 °C in nutrient broth medium
<u>30/70 % bentonite/crusched rock (section 4)</u>	
Sulfate reducing bacteria	Cultured at 30 °C in SRB and E media
Sulfate reducing bacteria	Cultured at 55 °C in E medium
Pseudomonas stutzeri (CCUG 42189)	Cultured at 30 °C in nutrient broth medium
Stenotrophomonas maltophila (CCUG 42190)	Cultured at 30 °C in nutrient broth medium
Bacillus thermophilic (CCUG 42191)	Cultured at 55 °C in nutrient broth medium