

LBL-7052
SAC-02

LARGE SCALE PERMEABILITY TEST OF THE GRANITE IN
THE STRIPA MINE AND THERMAL CONDUCTIVITY TEST

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Stockholm, March 1978

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for

Kärnbränslesäkerhet, KBS

(Swedish Nuclear Fuel Safety Program)

Reprinted by

Lawrence Berkeley Laboratory
Earth Sciences Division
University of California
Berkeley, California 94720
July, 1978

PREFACE

This report is one of a series documenting the results of the Swedish-American cooperative research program in which the cooperating scientists explore the geological, geophysical, hydrological, geochemical, and structural effects anticipated from the use of a large crystalline rock mass as a geologic repository for nuclear waste. This program has been sponsored by the Swedish Nuclear Power Utilities through the Swedish Nuclear Fuel Supply Company (SKBF), and the U.S. Department of Energy (DOE) through the Lawrence Berkeley Laboratory (LBL).

The principal investigators are L.B. Nilsson and O. Degerman for SKBF, and N.G.W. Cook, P.A. Witherspoon, and J.E. Gale for LBL. Other participants will appear as authors of subsequent reports.

Previously published technical reports are listed below.

1. *Swedish-American Cooperative Program on Radioactive Waste Storage in Mined Caverns* by P.A. Witherspoon and O. Degerman.

(LBL-7049, SAC-01)

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LARGE SCALE PERMEABILITY TEST OF THE GRANITE IN
THE STRIPA MINE AND THERMAL CONDUCTIVITY TEST,
SUMMARY

The granitic rock mass in a drift at 360 m level in the Stripa Mine has been investigated with regard to its permeability, effective fracture volume (porosity) and thermal conductivity.

A number of vertical holes, 10 m deep, were drilled in the bottom of the drift on two test places, 12 m apart. Each test place contained 16 peripheral holes in a circle of 3 m diameter and one with a large diameter in the centre.

The permeability of the rock mass

was determined by application of a constant water pressure in all peripheral holes and measurement of the flow of water from the centre hole under stable conditions. The measurements were performed at normal rock mass temperature (abt. $+10^{\circ}\text{C}$) at both places and also at increased bedrock temperature (abt. $+35^{\circ}\text{C}$) at one test place. The rock mass was heated with warm water circulating in the holes. The following permeability values were obtained with unheated rock mass $0.4 \cdot 10^{-10} \text{ m/s}$ and with heated rock mass $0.2 \cdot 10^{-10} \text{ m/s}$.

A conventional water loss test was made of each peripheral hole (ring hole), using both single and double packers. A comparison of the permeability values obtained with the packer tests and the large scale test shows that the water loss tests give 10 to 20 times higher values.

The effective pore volume of the rock mass

was determined by pressing water containing a tracer substance at constant pressure (500 kPa) from the ring holes and measuring the amount of tracer in the centre hole at evenly spaced time intervals.

By closely following the concentration increase of the tracer substance in the centre hole until the increase became constant, the effective pore volume of the rock mass, and thus its pore volume, could be determined. Amidorhodamine-G, a fluorescent compound, was used as tracer substance.

The effective pore volume, calculated in two different ways, was found to be abt. 5.4 l, equivalent to a porosity of 0.012 %.

The thermal conductivity of the rock mass

was determined by filling a 10 m deep, vertical drill hole with warm water, measuring the temperature decrease as a function of time. Two test runs were made, one resulting in a conductivity value of 3.5 W/m⁰C and the other 4.3 W/m⁰C.

Based on the results presented in this report the investigated properties of the granite bedrock at Stripa may be summarized as follows:

- The permeability is very low, $0.4 \cdot 10^{-10}$ m/s and independent of the pressure gradient.
- The permeability is reduced by 50 % at a temperature increase from abt. +10⁰C to abt. +35⁰C.
- The thermal conductivity was determined in situ to be abt. 4 W/m⁰C which largely agrees with laboratory determinations.

- The effective porosity was determined to be abt. 0.012 %.

Development and adaption of today's technology was required in order to perform the executed tests. This means that the necessary technology has been developed in order to plan and perform future field measurements of the hydrogeological properties of the rock mass.

LARGE SCALE PERMEABILITY TEST OF THE GRANITE IN
THE STRIPA MINE AND THERMAL CONDUCTIVITY TEST,
REPORT

Background

Commissioned by Kärnbränslesäkerhet, KBS, Hagconsult AB has concluded an investigation of the permeability properties of the Stripa Mine granite through in situ measurements at the 360 m level in an especially established test station.

The work was started during May 1977 and brought to an end in November. A preliminary report was submitted 1977-09-13. The planning of the permeability tests has been made in collaboration with Dr. techn. Sten G. A. Bergman. The objective has been to determine, in an almost impermeable rock mass, how the permeability varies with the pressure gradient, the effective pressure and temperature at a test site where the boundary conditions are definable. The final report of Nov, 1977 includes all basic data.

The investigations were made under strong time pressure considering the type of tests which fact has meant application of the following prerequisites for the planning of the test runs.

- Use of techniques available today
- Minimizing of time for preparation on test places and installations.
- Limitation of test runs, i. e. only the principle course of events is shown.

An investigation of the thermal conductivity and effective porosity of the rock mass was later added to the task.

1 TEST SITE

The drift for the testings has a length of abt. 50 m. It was divided into 4 alternative test places, each of them investigated with a \varnothing 76 mm drill hole. These holes were drilled vertically from the bottom of the drift to a depth of 15 m and were investigated by water pressure tests and television camera inspection. The water losses were very small and in order to get greater leakage, another hole with a length of 10 m was drilled on 1.3 m distance from the first one. Two of the places were selected for the permeability tests. On the selected test places, the distribution of fractures and water losses was the most even.

On one of the other places the investigation hole was used for the thermal conductivity test.

1.1 Layout of test places

Water movement within a granitic rock mass takes place through its fractures. The implication is that fracture geometry, orientation, roughness, width, filling material and extension are important parameters when analysing the water flow in the rock mass. The very complex mechanical and physical relations that influence the water flow are evidenced by the following.

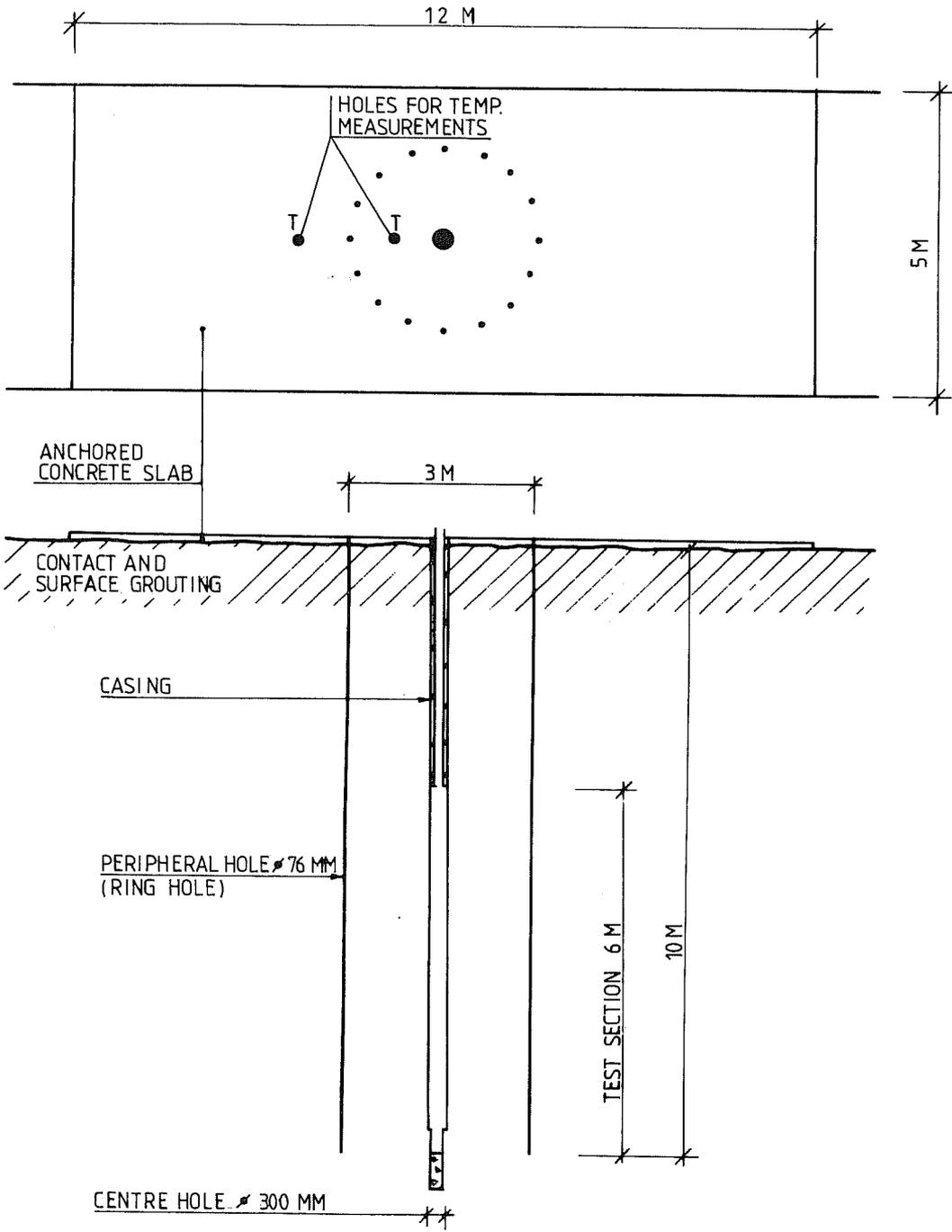
- Temperature changes influence the effective stresses in the rock mass as well as the kinematic viscosity of the water.
- The stress and strain changes cause a change of fracture widths.
- The water flow in a fracture is dependent upon the effective fracture width and the viscosity of the water.

- The waterflow through the rock mass is dependent upon the flow in individual fractures and the fracture characteristics.

The places for the permeability test were, in consequence, created with regard to the following:

- Possibility of measuring the flow at different gradients and total water pressure.
- Possibility of measuring the flow at different temperatures of the rock mass.
- The distance between water-carrying fractures should be small compared to the length of the investigated area.
- Well defined boundary conditions.
- Easy to analyse.

In order to comply with these conditions as far as possible, the test site was prepared and given the following design. In two places, 12 m apart, 16 vertical, \varnothing 76 mm and 10 m deep, peripheral holes were drilled in a circle of 3 m diameter, with a central \varnothing 300 mm vertical hole, see FIG. 1:1.



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FIG. 1:1 Layout of each test place

The tests were made as follows: water at constant pressure was pressed into each peripheral hole in each of which was mounted an injection pipe with a mechanical packer positioned abt. 1 m below the rock surface, see FIG. 1:2.

In the centre hole a casing pipe was cemented in place, equipped with a pressure lid. The water flow-rate to the centre hole was measured at counter pressure as well as zero pressure, see FIG. 1:3 and 1:4. The length of the casing was chosen to ensure that the influence by the tunnel floor proximity became as negligible as possible. The geological prerequisites were established through mapping of the drill cores acquired during the core drilling and through TV-examination of the drill holes in order to allow detailed design of the test places.

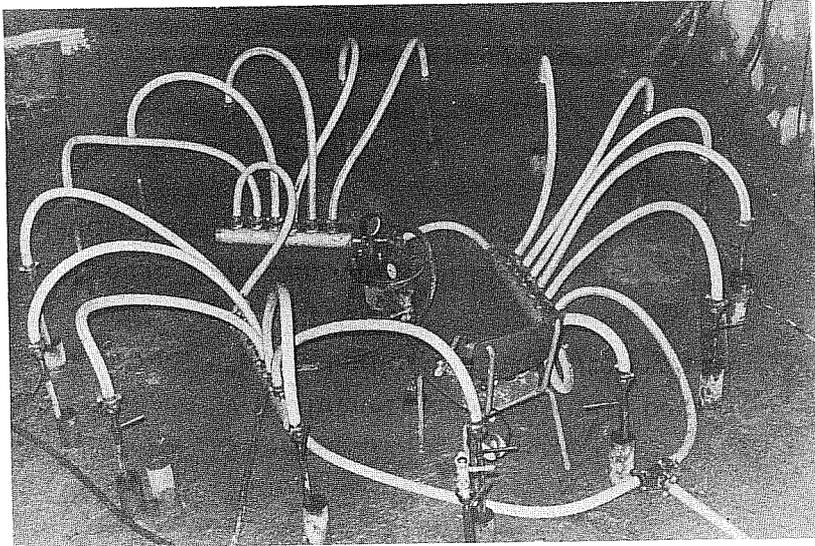


FIG. 1:2 Test place No. 1

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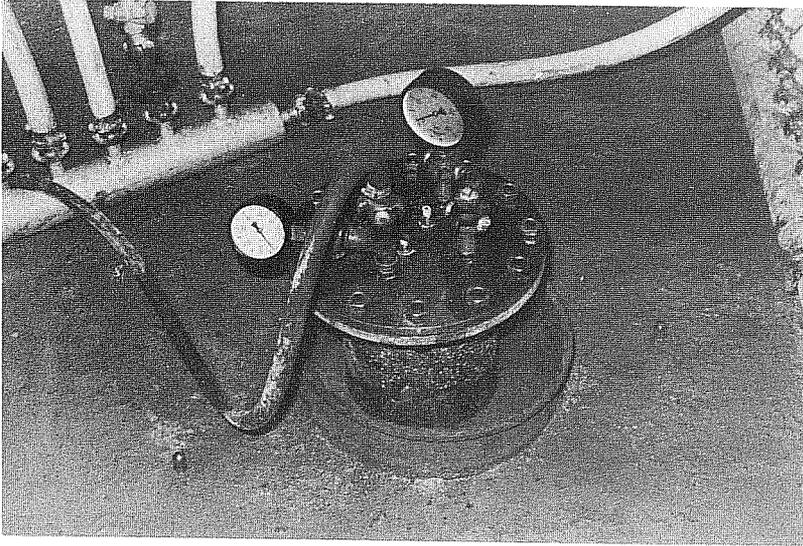


FIG. 1:3 Centre hole, casing pipe and pressure lid

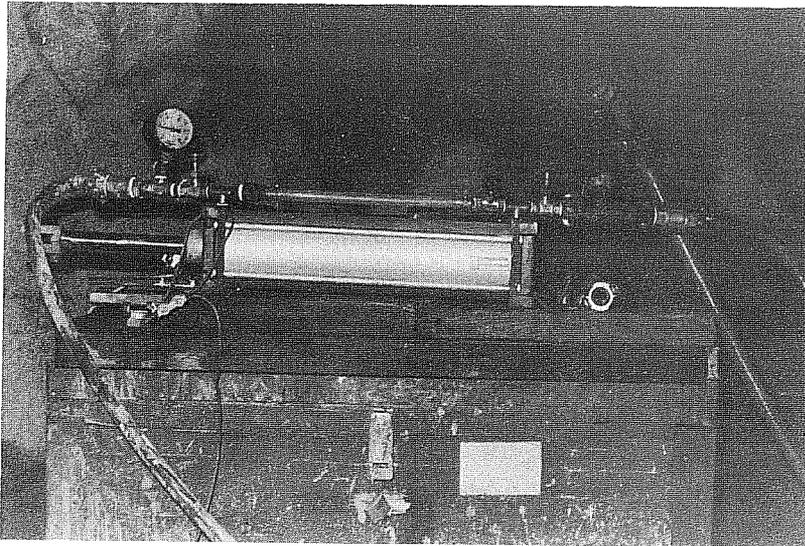


FIG. 1:4 Flow rate meter

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1.2 Geological conditions

1.2.1 Rock mass

The rock mass in the test site for the permeability test in the Stripa Mine is uniform and it consists mainly of an even-grained granite with a granitic texture. The granite is serogene, of Svecofennic age, homogeneous and with relatively few greater fractures and other discontinuities.

The mineral components are: grey-white quartz, red - grey potassium feldspar, mica (comparatively uncommon) and a grey-white sodium feldspar. The micas occur in the form of biotite and chlorites. No detailed study has been carried out regarding the mineralogical components or their distribution per centage nor has any chemical analysis been considered necessary.

Due to the relatively low age of the granite and its occurrence at the end of an orogenic period, gneissic structures do not exist.

1.2.2 Fracturing

A number of fractures has been observed in the drift belonging to different "families", i. e. fractures with the same orientation in space. These fractures are usually coated with chlorite, a platy mineral which through its occurrence in the form of mm-wide seams (=fillings of the fractures) gives the granite a certain degree of anisotropy. Some chlorite-coated fractures were observed to be open or partly open, i. e. they are possible water paths. From two of these, water leakage into the drift was noticed on the walls of the test site in the form of dripping and moisture, measured flow-rate abt. 10-20 ml per 24 hrs.

The geological study showed a clear dominance of fracture families with gentle dip or nearly horizontal. Medium-steep fracture families do occur, but less frequently. Sparsely there are also fractures with a steep dip, cf. FIG. 1:5.

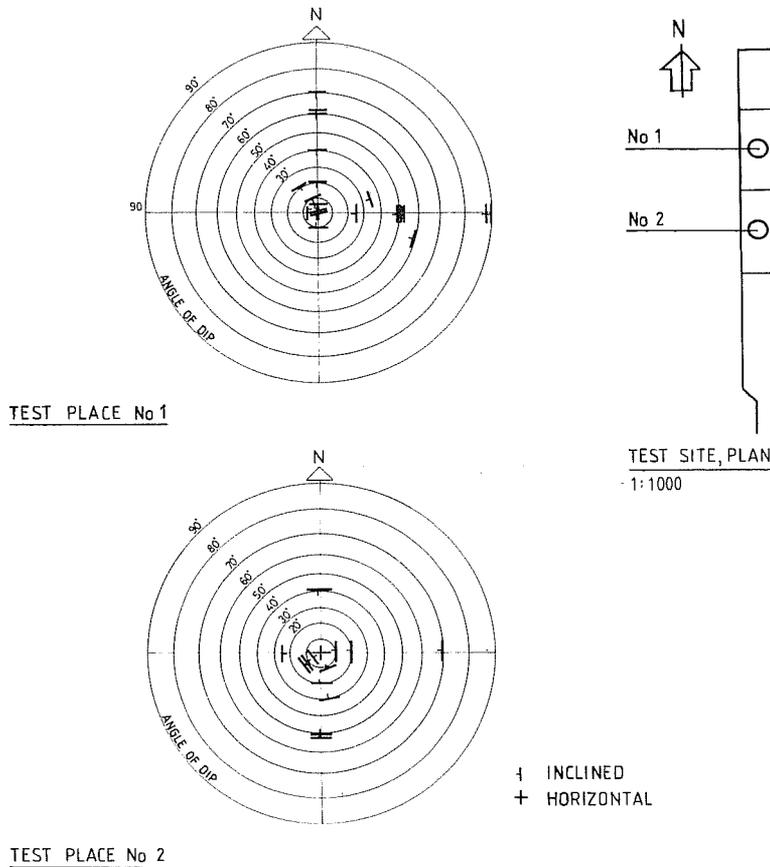


FIG. 1:5 Open fractures (tangential point on the lower half-sphere, stereographic projection)

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The strikes of the fractures are dominantly nearly N-S and E-W, sometimes NW-SE and sparsely NE-SW. In TAB. 1:1 all the fractures and cracks are compiled which were recorded at the inspection with the TV-camera. No distinction regarding filling material has been made. The strike values have been rounded to even 5 and 10 degrees. The distribution of the open or partly open fractures is shown in TAB. 1:2. It is obvious that also others of the observed fractures may be permeable, but this can not be estimated through inspection, though the bore hole camera used
tion of fissure widths of abt. 0.2 mm.

From the table it is evident that a limited number of open fractures exists with varying orientation. An analysis of their space orientation has indicated hydraulic connection through these fractures between different parts within each test place.

A calculation of the fissure frequency per space unit gives the following preliminary result.

There are abt.4 fractures per m³ rock mass as a mean value. Only a few of them seems to be open. The relatively great number of fracture families with varying fracture widths, filling material etc., will probably mean that the water-paths through the rock mass are irregular and complicated.

The water flow at the test places where the distance is limited should therefore be greater than in the mass as a whole; i. e. flow between two points of greater distance.

Orientation	Frequency			Dip
	0 - 5	5 - 10	10 - 20	
E-W/0-10°S			x	Gentle
E-W/20-40°S			x	
E-W/0-10°N	x			Steep
E-W/20-40°N		x		
E-W/60-80°S	x			Steep
E-W/50-75°N		x		
N-S/10-20°E		x		Nearly horizontal to medium-steep
N-S/20-40°E	x			
N-S/5-20°W		x		Steep
N-S/35-50°W	x			
N-S/55-85°E		x		Steep
N-S/70-85°W	x			
N45°E/15-30°NW		x		Gentle
N20°E-55-70°W	x			Steep
N25°W/40-60°SW	x			Steep

TAB. 1:1 Total number of registered fractures.

Orientation	Frequency	Dip
N-S/5°E E-W/5°S N10°W/5°E N75°E/10°SE	ca 35 %	Nearly horizontal
N-S/50°W E-W/50°N N20°W/40°NE N20°W/50°SW N60°E/25°SE	ca 40 %	Medium-steep
N-S/70°W E-W/60°W E-W/75°N	ca 25 %	Steep

TAB. 1:2 Distribution of registered fractures, open or partly open.

2 PERMEABILITY

2.1 Description of tests

In order to determine the rock mass permeability, the water flow-rate from the ring holes towards the centre hole was measured at a certain, overpressure in the ring holes, with and without counter-pressure at the centre hole.

By varying the pressure difference between peripheral and centre holes at different counter-pressures in the centre hole, the water flow dependency of effective pressure could be studied. Two different counter-pressures were applied, 160 and 460 kPa. The differences were 15, 30, 100 and 500 kPa.

In order to study the permeability temperature dependency, test place No. 1 was heated to abt. +35°C by circulating warm water in a closed system through peripheral and centre holes. The water flow measurements, after heating, were made at zero pressure in the centre hole and at 100 and 500 kPa in the ring holes.

All water flow measurements were made after stable conditions had been reached. Water loss measurements using single and double packers were made in the peripheral holes. For comparison, the permeability was calculated from these values also.

2.2 Drill hole tests

2.2.1 Analysis of measuring data

One pre-requisite if using the water loss values in calculations of the rock mass permeability is that the fracture spacing is essentially less than the test length. The results from the double packer

tests, where the test length is 1 m only, must therefore be handled with care. Even if the Stripa granite is comparatively fractured, 5 - 10 fractures per meter, a great deal of them are sealed or filled with quartz and chlorite(s) or their extension is limited.

A common way of calculating the permeability from measured flows has been described by i. a. Sanger(1953). The following equation has been taken from his work:

$$k_m = \frac{q \cdot C}{L \cdot p} \quad (1)$$

q is here the measured flow, L is the test length, p is the effective overpressure in meter water column and C is a constant dependent upon the length/width relationship of the test section.

$$C = \frac{1 + \ln \frac{L}{2r}}{2 \pi}$$

where r is the hole radius.

In this case only one parameter is not well-defined, namely the pressure.

No hydrostatic equilibrium exists due to the water percolation to the drift. Also, this flow has been disturbed by the holes drilled for the permeability tests.

These holes were open during the water pressure tests performed in each individual hole and could therefore have served as a drainage system. This means that the water pressure in the rock mass could have been lowered nearly to a hydrostatic pressure within each test place.

By closing the drill holes and recording the water pressure at different depths in some of the holes, an idea of the gradient was gained. At the test place No. 1, the gradient was 1.6. At the test place No. 2, these pore pressure measurements were uncertain and because of the very small flows of water and thus no gradient was obtained.

Principally, the boundary conditions of the effective pressure p in meter water column can be described as follows:

$$H \geq p \geq H - 1.6 Z$$

where Z is the depth from the bottom to the middle of the test section and H is the manometer pressure. Beside their sinking effect on the pore pressure, the drill holes also influence on the symmetric flow to the surrounding drill holes, the calculated permeability values will be too high.

Alternative methods to determine the permeability from the measured flow rates are analytic or graphic solutions, based on the differential equations that describe the flow. The flow near to a hole is of different types as follows:

- plan axi-symmetric radial flow from the hole
- three-dimensional spheric radial flow from each end of the hole.

These are, however, approximations of the reality, especially the flow from the end surfaces. The rock mass is not a homogeneous isotropic porous medium, the water percolate in the fractures only. But, if a spheric radial flow is assumed, this can give a fairly good estimate of this water flow.

The spheric radial flow from the end surfaces of a drill hole can be calculated by using the equation:

$$Z = h_0 + \frac{1}{k_m} \cdot \frac{q}{4\pi} \left(\frac{1}{r} - \frac{1}{R} \right) \quad (2)$$

where Z is the head of the water at a distance R from the hole end level, r is the radius of the hole and h_0 is the water pressure inside the hole (water column above the end surface). All dimensions are in meter.

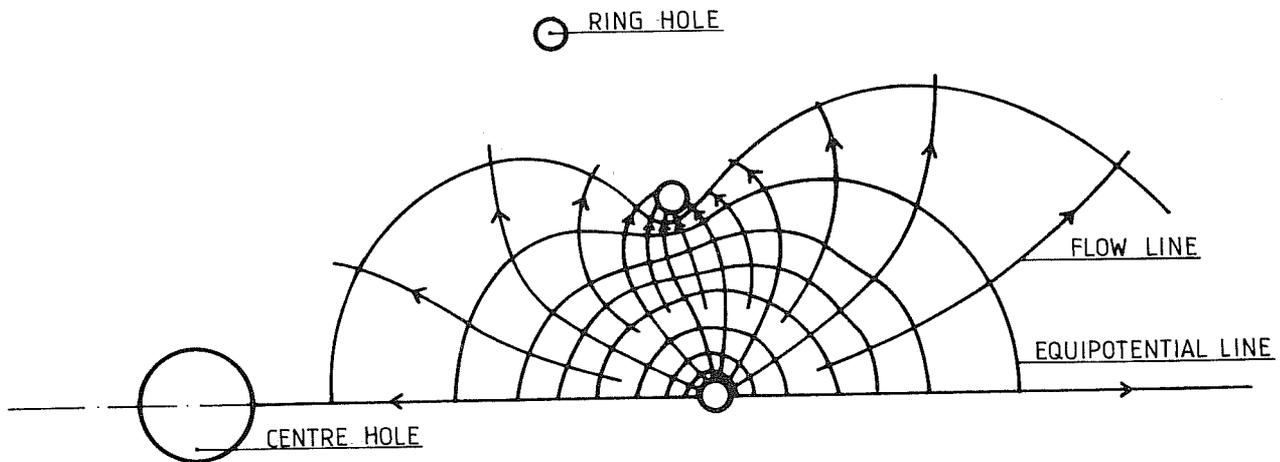
The plan axi-symmetric flow can be described accordingly as follows:

$$Z = h_0 + \frac{1}{k_m} \cdot \frac{q}{2\pi \cdot L} \cdot \ln \frac{R}{r} \quad (3)$$

where L is the length of the test section (m).

As the flow from one drill hole was influenced by the other ones and thus was not axi-symmetric, the boundary conditions put into the equation (3) must be specially considered.

An alternative method is to solve the problem graphically by constructing flow and equipotential lines. On FIG. 2:1 a flow network has been drawn showing the plan flow that will be created around one of the ring hole during the water pressure test is performed in this hole.



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FIG. 2:1 Flow network at plan flow from one of the ring holes

The rock mass permeability can be calculated from the measured flow-rate according to the following equation:

$$k_m = \frac{q}{h} \cdot \frac{n}{m} \cdot \frac{1}{L} \quad (4)$$

where m is the number of flow channels and n is the number of squares along the flow direction between two points with different heads h (m).

By comparing the graphic solution with the ratio permeability/plan radial flow calculated according to the equation (3), it was found that the other holes had a great influence. The analysis shows that a hydrostatic pressure in a distance equal to that between the peripheral holes can be used as a boundary condition when calculating the permeability of the water pressure test results in the individual holes.

By combining the equations (2), (3) or (4) with the assumption that the noticed total water loss is equal to the sum of the water losses at the hole ends and the part of the hole between them, the rock mass permeability can be calculated on the basis of the water loss measurements.

Inserting the boundary condition in equation (2) and (3) gives the following:

$$q_1 = k_m \cdot 4\pi \cdot p \cdot r \quad (2a)$$

$$q_2 = k_m \cdot p \cdot \frac{m}{n} \cdot L \quad (4a)$$

$$2 \cdot q_1 + q_2 = q \quad (5)$$

where p is the manometer pressure in meter water column counted from the tunnel bottom and q is the measured water loss.

2.2.2 Results

As described above, the permeability calculated in accordance with the equation (1) gives too high values due to the influence of the surrounding open drill holes.

The most correct values should be obtained when using the equation system above. Both the losses at the end surfaces and the surrounding holes will then be considered. Naturally the influence of leakage from the end parts is essentially greater at the double packer measurements than when using the single packer equipment.

The mean values of the permeability calculated from the single and double packer measurements have been compiled in TAB. 2.

Mean values from the single packer measurements

Test place No. 1

p	k_m
300 kPa	$0.96 \cdot 10^{-9} \text{ m/s}$
100 kPa	$0.88 \cdot 10^{-9} \text{ m/s}$

Test place No. 2

p	k_m
300 kPa	$0.51 \cdot 10^{-9} \text{ m/s}$
900 kPa	$0.55 \cdot 10^{-9} \text{ m/s}$

Mean values of some individual holes, comparison between double packer and single packer measurements

Test place No. 1

Drill hole	Double packer	Single packer
1	$3.1 \cdot 10^{-9} \text{ m/s}$	$2.0 \cdot 10^{-9} \text{ m/s}$
4	1.3	0.20
8	0.87	0.34
14	3.3	2.6

Test place No. 2

Drill hole	Double packer	Single packer
3	$1.6 \cdot 10^{-9} \text{ m/s}$	$0.57 \cdot 10^{-9} \text{ m/s}$
5	2.7	2.6
13	1.7	0.7
16	1.5	0.7

TAB. 2 Permeability values based on water pressure tests of individual drill holes.

When comparing the mean values of the permeability from the double packer measurements with corresponding single packer values it is evident that the former values are all greater, maximum abt. 7 times greater. This means that the end effects are, as assumed above, of considerable importance. When using a double packer system for permeability calculations this should specially be studied and considered.

The results in TAB. 2 show no significant difference for the pressure 300 kPa and 900 kPa.

The measurements with the double packer equipment show that the water losses normally gave an even variation along the holes without greater differences. Only a few 1 m sections were quite tight. These results also correspond with the geological structure achieved through mapping and drill hole inspection. Therefore it appears to be sound to assume a porous continuous medium at the analysis of the large scale permeability test.

2.3 Large scale test

2.3.1 Analysis of measuring data

As distinguished from the water pressure tests on the individual drill holes the large scale tests were carried out under better controlled conditions, especially regarding the boundary conditions.

By testing at a constant and higher pressure in the peripheral holes and measure the flow-rate to the centre hole, several of the objects that can be raised to usual water pressure tests in drill hole were eliminated. In addition to well-defined boundary conditions the tests were not sensitive for leakages in the inlet system (tubes, couplings and packers).

Equation (3) can be used for the calculations. Inserting the boundary conditions $R = 1.5$; $r = 0.15$ and $Z - h_0 = p_1$ (the pressure difference between the ring holes and the centre hole) gives the following equation:

$$q_1 = 2.7 \cdot k_m \cdot p_1 \cdot L$$

The water flow through the bottom of the centre hole can roughly be estimated by using the equation (2). The effective water pressure p_2 corresponds to the difference between the initial pore pressure in the rock mass (160 kPa) and the water pressure in the centre hole, h_0 .

$$q_2 = 1.8 \cdot k_m \cdot p_2$$

When testing with 0-pressure in the centre hole the pressure difference between the hole and the pore pressure near to the hole at the top of this is negligible. This means that eventual leakage paths through the rock upwards to the drift bottom also is negligible and thus the measured flow-rates are the sum of rates from the hole walls and the bottom.

However, at the tests with an applied counter-pressure in the centre hole there occurred a certain leakage up towards the slab on the outside of the casing as well as from the bottom of this hole.

This total spheric radial flow-rate, the so called loss flow-rate, was determined by applying the same pressure in the centre hole as in the ring holes.

The plan radial flow-rate was achieved by subtracting the loss flow-rate from the total flow-rate to the centre hole.

The loss flow will probable only to a minor part be effected by pressure changes in the ring holes, as the leakage mainly occurs near to the casing and therefore depends on the pressure differences inside and closely outside the casing at the drift bottom level.

The flow at small pressure differences between the ring holes and the centre hole was lower than the estimated accuracy of the measurement, this because of the very low flow-rates. This meant that these results had to be omitted.

2.3.2 Results

Because of the relatively speaking long time required for each separate permeability test, the test series had to be limited. Therefore, the results presented here may, at times, just give an indication of the hydrogeological relations. To closer elucidate these, further test series will be required.

At zero pressure in the centre hole, the permeability of the bedrock appears to be independent on the pressure gradient not only at normal bedrock temperature, (abt. $+10^{\circ}\text{C}$) but also at higher bedrock temperature (abt. $+36^{\circ}\text{C}$). The permeability value for normal temperature has been calculated to be abt. $0.4 \cdot 10^{-10} \text{ m/s}$. This value is based upon 9 separate tests, and is the same for both test places, see also FIG. 2:2 below. For heated rock, at abt. $+36^{\circ}\text{C}$, tested at place No. 1, a marked decrease in permeability was noted, to about half or $0.2 \cdot 10^{-10} \text{ m/s}$. At the same time, the kinematic viscosity has decreased to about half its original value, the implication being that the permeability would decrease about 4 times by temperature increase from $+10$ to $+36^{\circ}\text{C}$, if the temperature of water were to be constant at $+10^{\circ}\text{C}$.

The correlation between permeability and effective pressure has been studied, i. e. the net pressure between the blocks of rock material with due regard to the pore pressure of the water in the fractures, cf. FIG. 2:3.

As pore pressure, at the respective test, the mean value of the peripheral and centre holes has been used.

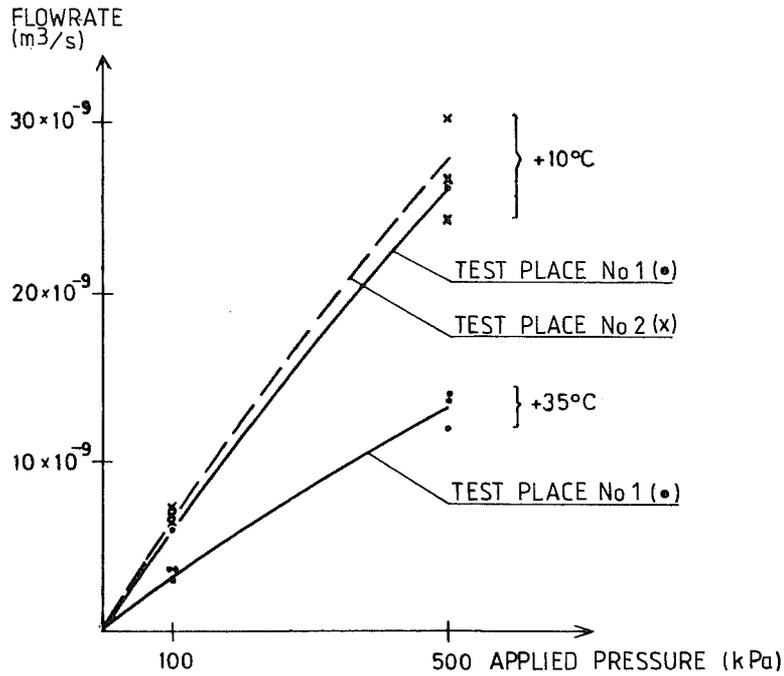


FIG. 2:2 Water flow as a function of the applied pressure measured at two different temperatures

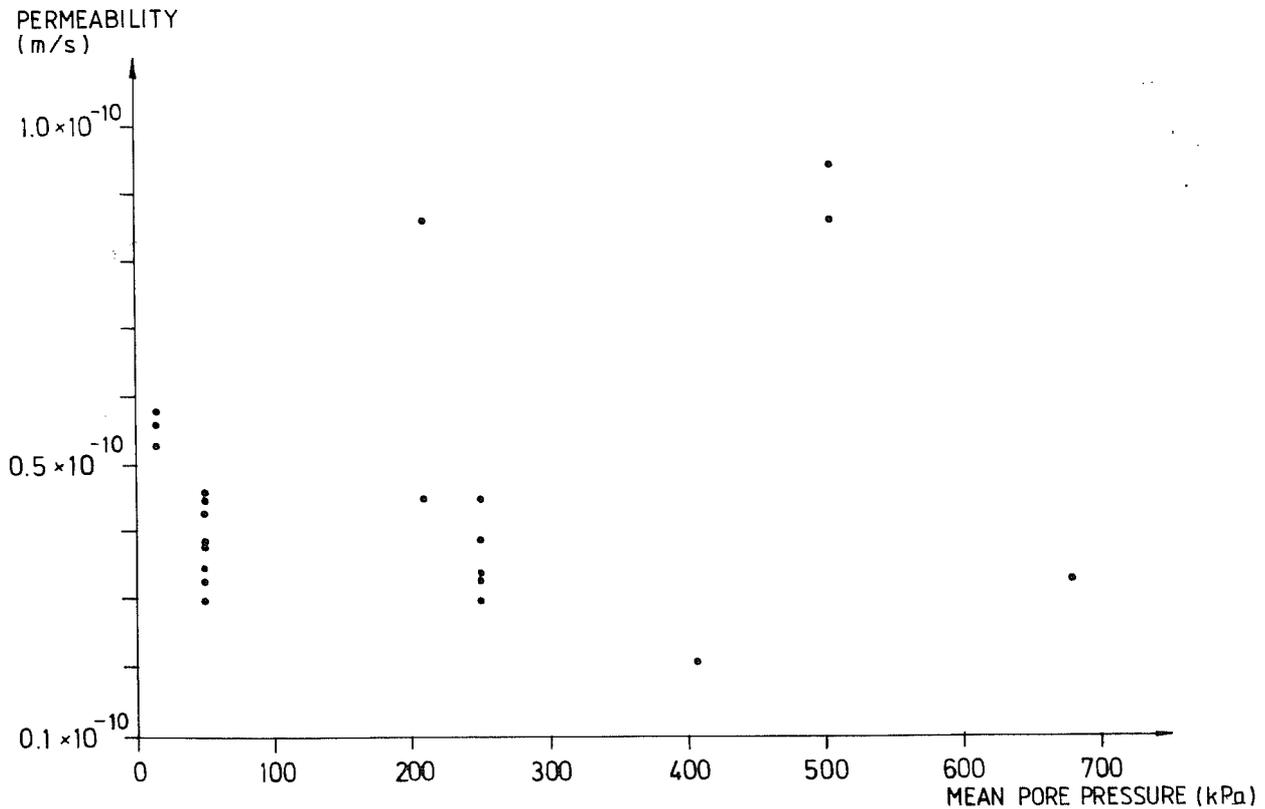


FIG. 2:3 Permeability as a function of pore pressure

As evidenced, there is no obvious correlation to be seen. The number of reliable test results is too small to allow this. However, no radical permeability change appears to have taken place at the applied pressures. All values are within a factor of 2, calculated from the earlier given k_m - value $0.4 \cdot 10^{-10}$ m/s.

3 POROSITY

3.1 Description of test

The determination of the effective fracture volume of the rock mass was made using tracer substance techniques. The fluorescent colour substance Amidorhodamine-G was chosen as tracer substance. The water of the peripheral holes was stained with the tracer and subsequently samples were taken from the centre hole for assaying. The amount of time for a specific volume of water flowing from peripheral holes to centre hole, may thus be determined. From the results, the effective fracture volume may be calculated.

The test run was commenced with flushing and cleaning of all holes, followed by addition to the water of the peripheral holes of a solution of the tracer substance to a concentration of abt. 100 ppm. The water in each hole was carefully stirred in order to get an even distribution of the substance.

Sampling of the water in the centre hole was made once every hour for 49 hours after addition of tracer. After an interval of 12 hours, further sampling was made, with longer intervals, to abt. 450 hours from the start.

Preceding each sampling in the centre hole, the water was mixed in a similar manner as in the peripheral holes. The samples were taken at 5 m depth in a 200 ml glass bottle. In order to check the concentration changes in the ring holes, these were sampled at varying times.

All samples have been assayed at SGU's geochemical laboratory, with a fluorescence-spectrophotometer, registering the intensity of fluorescence as a function of the wave length emitted.

Through analysis of solutions with known concentrations the correlation between intensity and concentration could be calculated. The sensitivity of the analysed instrument was measured at abt. 0.001 ppm.

3.2 Analysis of measuring data

The concentration of the tracer substance in the centre hole (c_2) as a function of time was determined. The concentration increase with the time gives a curve which first part is shown in principle in FIG. 3:1. c_0 represents the concentration in the ring holes at the time $t = 0$.

The fractures that carry water from the ring holes to the centre hole vary in width, orientation etc. This means that the flow-rate and flow velocity differ from fracture to fracture. The concentration increase of the tracer substance in the centre hole will therefore at the beginning be non-linear.

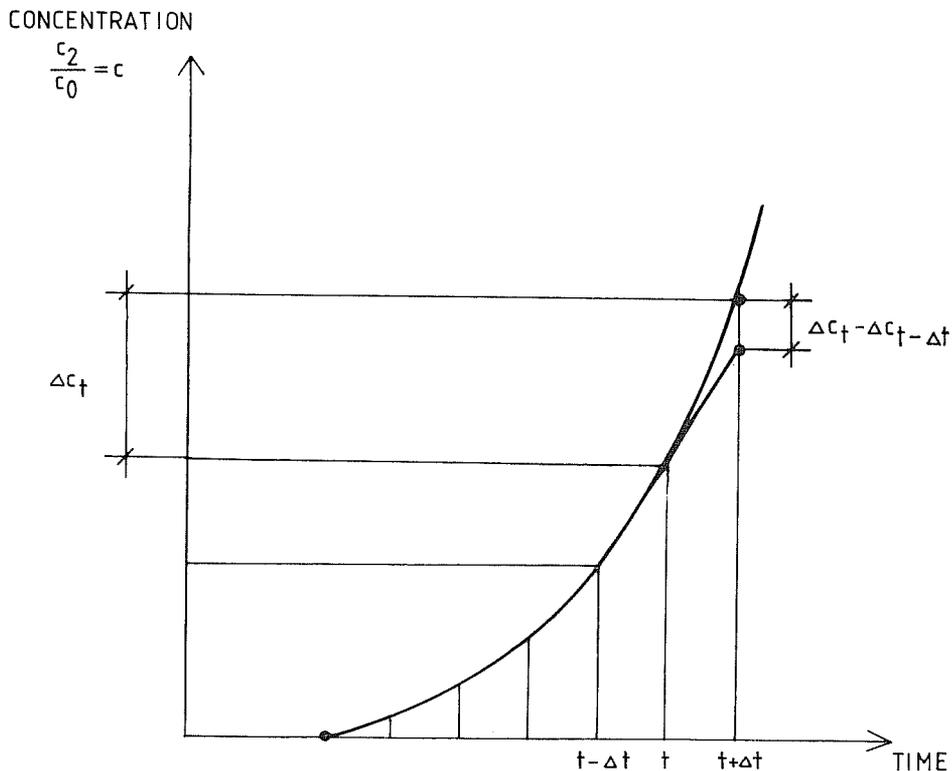


FIG. 3:1 Principle figure showing the difference-calculation method

The effective pore volume of the rock mass can be calculated by using a technique with difference-calculation. The curve is divided into certain time intervals (Δt). The increase of the tracer-carrying water flow from one interval to the next is a function of the corresponding fracture volume and of the travelling time. The increase of the tracer substance during the interval Δt can be signed in two ways:

$$\Delta c_2 \cdot V_2 = q \cdot c_0 \cdot \Delta t$$

This gives:

$$q = \frac{\Delta c_2}{c_0} \cdot \frac{V_2}{\Delta t} = \Delta c \cdot \frac{V_2}{\Delta t}$$

V_2 is the volume of the centre hole, q is the water flow that carries the tracer substance. c_0 is the concentration of tracer substance in the ring holes at the time $t = 0$, here assumed to be constant. Between the time t and $t + \Delta t$ the additional flow carrying tracer (Δq) will be as follows:

$$\Delta q = \frac{V_2}{\Delta t} (\Delta c_t - \Delta c_{t - \Delta t})$$

When all the fractures carry tracer marked water to the centre hole the Δq will be zero, which means a linear increase of the concentration of tracer substance.

The fracture volume which gives the additional tracer-carrying flow from t to $t + \Delta t$ can be determined.

$$\Delta V = \Delta q \cdot t$$

The total effective fracture volume was calculated by adding these.

$$V = \sum \Delta V$$

The analysis has also included an alternative method to calculate the fracture volume. In this case the mean travelling time of the water from the ring holes to the centre hole has been used as a basis for the calculations.

The water velocity in the pores can be signed:

$$v_p = \frac{k_m}{n} \cdot i$$

where n is the porosity, k_m is the permeability of the rock mass and i is the pressure gradient.

$$i = \frac{dz}{dR}$$

$$z = \frac{q}{2\pi \cdot L} \cdot \frac{1}{k_m} \cdot \ln \frac{R}{r}$$

The total travelling time t_o can be calculated from the equation

$$\int_0^{t_o} dt = \int_r^{R_o} \frac{dR}{v_p} = \int_r^{R_o} \frac{2\pi \cdot L \cdot n}{q} \cdot R \cdot dR$$

$$t_o = \frac{2 \cdot L \cdot n}{q} \cdot \frac{R_o^2 - r^2}{2}$$

where R_o is the radius of the tested rock cylinder, L is the length of the non-encased part of the centre hole and r is the radius of this hole.

As R_o is much greater than r this equation can be simplified.

$$n = \frac{t_o \cdot q}{\pi \cdot L \cdot R_o^2} \quad (2)$$

3.3 Results

The ratio of the concentration of tracer substance in the centre hole to that in the peripheral holes was plotted in a diagram as a function of time, see FIG. 3:2. As the leakage path and the time vary for each separate fracture carrying water from peripheral to centre hole, the tracer concentration increase in the centre hole will not be linear with time, initially. When all fractures carry tracer stained water, the increase will be linear with time.

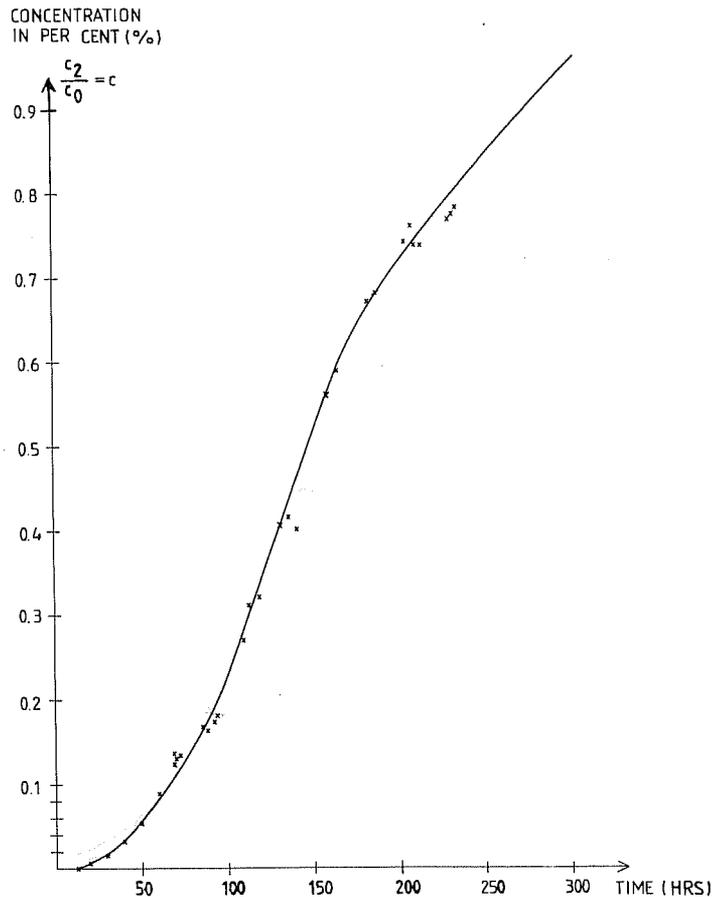


FIG. 3:2 Tracer substance concentration in the centre hole in per cent of the initial concentration in the ring hole as a function of flow time.

By dividing the curve in time intervals up to the time when all fractures carry stained water, the water flow which causes an increase of tracer substance may be determined by using difference calculation technique. From this water flow, the fracture volume as a function of the travel time may be calculated.

The results show that a tracer-carrying flow of 27.5 ml/h only, has been had, which is considerably lower than the earlier measured flow of abt. 90 ml/h.

The determination based on the difference-calculation described in Chapt. 3:2 gave an effective fracture volume of 1.65 l.

Should there be a systematic error in the measurements of the concentration, the tracer-carrying flow instead being the observed total flow, abt. 90 ml/h, then the fracture volume will be 5.4 l. The alternative method based on the mean travelling time of the water gives the following results.

The first tracer substance arrived at the centre hole 13 hours after the start and a linear increase, i. e. all the open fractures carried tracer stained water, was attained after 100 hours. This gives a mean travelling time of 53 hours.

The figures $t_0 = 53$ hrs, $q = 90$ ml/h, $L = 6$ m and $R_0 = 1.5$ m put into the equation (1) Chapt. 3.2 gives the porosity $1.3 \cdot 10^{-4}$ and the total fracture volume abt. 5.5 l.

4 THERMAL CONDUCTIVITY

The thermal conductivity of the rock mass is normally determined in the laboratory on small pieces of rock. However, such testings give no information of the fracture influence etc. The values are not necessarily representative for the rock mass which often seems to be lower.

In connection with the large scale permeability tests a relatively new method to determine the rock mass conductivity was tested.

A drill hole was filled up with warm or hot water and the lowering of the temperature of the water was measured as a function of the time. One of the investigation drill holes on the test site in the mine was used.

4.1 Description of test

In order to determine the rock mass thermal conductivity, a $\varnothing 76$ mm, 10 m deep vertical drill hole was filled with warm water. The water temperature was checked at even time intervals, using 3 resistance-thermometers (Ni 100), mounted centrally in the hole at 5.8, 7.8 and 9.7 m depth.

The initial bedrock temperature was determined before the warm water was poured down the hole, whereupon the hole was blown empty from water with compressed air. The bottom of the hole was insulated with a 200 mm layer of ball sinter (type Leca). The upper part of the hole was also insulated with Leca, abt. 300 mm layer thickness, see FIG. 4:1. Two tests were made, one using 40°C water and another using 80°C water. The water temperature of the mentioned levels were recorded each minute during the first 15 minutes, thereafter each 5th minute during 45 minutes, after addition of warm water.

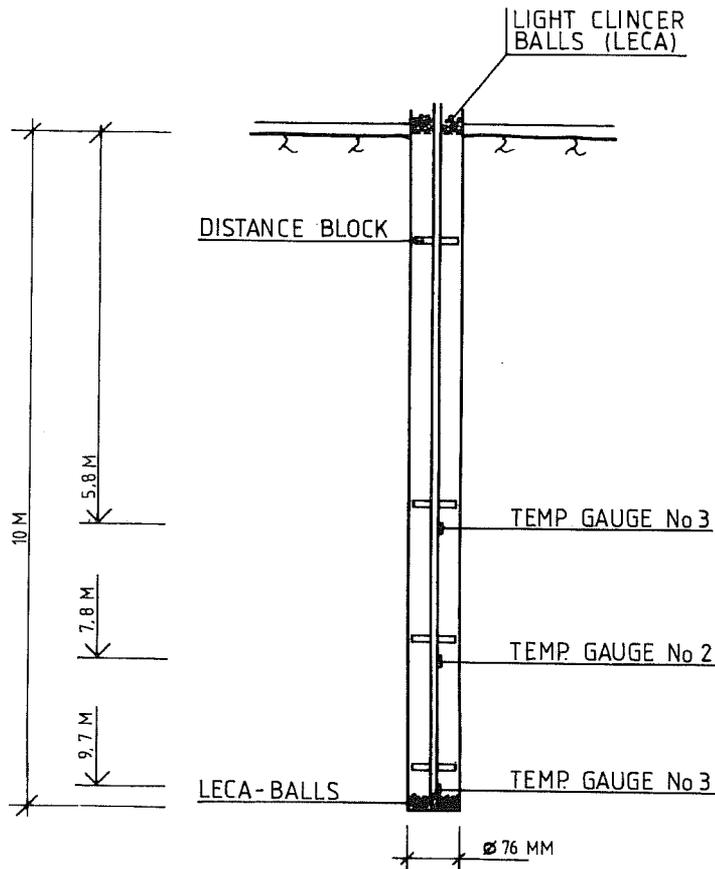


FIG. 4:1 Layout of thermal conductivity test

XBL 786-9211

4.2 Analysis of measuring data

The following relationship has been formulated by Beck, Jaeger and Newstead:

$$\frac{V}{V_0} = F(\alpha, \tau)$$

V_0 is the temperature difference between the water and the rock mass at the time $t = 0$, V is the corresponding value at the time t . Their ratio is thus a function of α and τ . α is dimensionless and a constant determined from

$$\alpha = \frac{2\pi a^2 \cdot \rho \cdot c}{S}$$

where a is the radius of the hole, ρ is the density of the rock, c is the thermal heat capacity and S means the thermal heat capacity of the water per meter hole.

The test hole has $\alpha \sim 1.0$ ($a = 38$ mm, $\rho = 2650$ kg/m³, $c = 800$ Ws/kg⁰C and $S = 18953$ Ws/m⁰C).

↓
1.191 cal/gm⁰c
The parameter τ is dependent on the time and is dimensionless. Its magnitude is determined from the equation

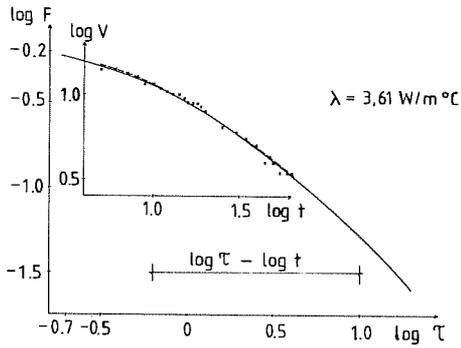
$$\tau = \frac{\sigma \cdot t}{a^2}$$

where σ is the thermal heat diffusivity of the rock. t is counted from the start point of the test.

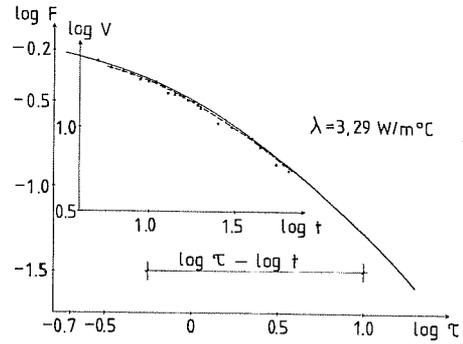
Using the values given by Jaeger a curve has been constructed showing $\log F(\alpha, \tau)$ as a function of $\log \tau$ when $\alpha = 1$. This curve has been fit into the diagram that has been plotted on the basis of the field test results, cf. FIG. 4:2. These diagrams show the $\log V$ as a function of $\log t$.

The difference $\log \tau - \log t$ in the figure gives $\log \frac{\sigma}{a^2}$.

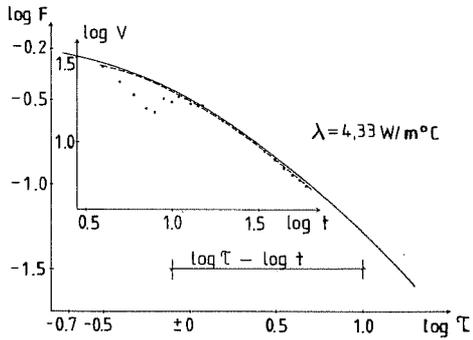
The thermal conductivity of the rock mass (λ) can be determined from the relationship $\lambda = \sigma \cdot \rho \cdot c$.



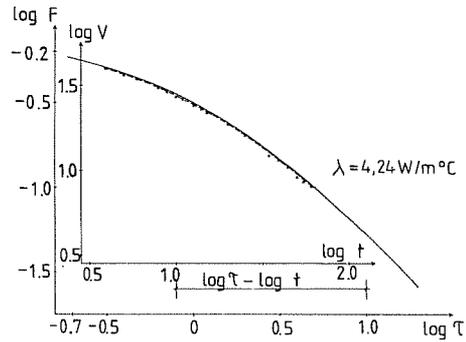
TEST RUN 1, GAUGE No 2



TEST RUN 1, GAUGE No 3



TEST RUN 2, GAUGE No 2



TEST RUN 2, GAUGE No 3

— $\log F = f(\log t)$
 - - - - $\log V = g(\log \tau)$

FIG. 4:2 Thermal conductivity test.
 The curve $\log V = f(\log t)$ fit to the
 curve $\log F = g(\log \tau)$, $\alpha = 1$.

This report is part of a cooperative Swedish-American project supported by the U.S. Department of Energy and/or the Swedish Nuclear Fuel Supply Co. Any conclusions or opinions expressed in this report represent solely those of the author(s) and not necessarily those of The Regents of the University of California, the Lawrence Berkeley Laboratory, the Department of Energy, or the Swedish Nuclear Fuel Supply Co.

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