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PLAN OF COUPLED THERMO-HYDRO-MECHANICAL EYPERIMENT AT THE KAMAISHI MINE

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TOKAI WORKS

POWER REACTOR AND NUCLEAR FUEL DEVELOPMENT CORPORATION

(I)

PLAN OF COUPLED THERMO-HYDRO-MECHANICAL EXPERIMENT AT THE KAMAISHI MINE

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要　旨

地層処分における技術開発の観点からは，工学規模での試験によりニアフィールド環境である周辺岩盤の挙動が人工バリアに与える影響の把握および周辺岩盤を含むニアフィールド性能の定量的評価と室内および原位置における大型試験による人工バリアの品質性能の確認を行い，地層処分技術の信頼性向上を図ることが重要となっている。そのため，動燃東海の地層処分基盤研究施設等における工学規模の試験と並行して，釜石原位置試験場において，人工バリアの品質性能の確認およびその実岩盤条件下でのニアフィールド連成挙動を評価することが必要となっている。

そこで，実条件でのニアフィールド環境を把握するため原位置における粘土膨張・熱負荷による緩み領域の影響評価の原位置試験を実施することとし，試験計画案を策定した。

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1. INTRODUCTION

It is an important part of the performance assessment of near field to evaluate coupled thermo-hydro-mechanical (T-H-M) phenomena, e.g., thermal effects on groundwater flow through rock matrix and water seepage into the buffer material, and generation of the swelling pressure of the buffer material, and thermal stress potentially affecting porosity and fracture aperture in rock. In-situ T-H-M experiment which is named 'Engineered Barrier Experiment' has been planned at the Kamaishi mine of which host rock is granodiorite in order to establish coupled T-H-M conceptual models and to make a confidence of the mathematical models and computer codes.

This report describes the T-H-M experiment including the test site features of the Kamaishi mine.

2. OBJECTIVES

The major objectives for the T-H-M experiment at Kamaishi are ;

to observe near-field coupled T-H-M phenomena in situ and to make a confidence of coupled T-H-M models,

to develop excavation technology of disposal pit excavation and to evaluate the effect of the pit excavation on surrounding rock mass,

to evaluate applicability of the engineered barrier technology.

The concrete test programme on the first objectives includes the input data acquisition for coupled T-H-M models and the experimental measurement for comparison with the prediction from the coupled T-H-M numerical simulations. The properties of bentonite which have been given will be available as the input data. Then, the hydraulic and mechanical rock properties of the test site, and mechanical properties of the core sample will be measured in the programme. The distribution of temperature, moisture, pore water pressure, strain and stress in the bentonite and the rock mass under coupled T-H-M condition will be measured as the data for comparison with the calculated results derived from coupled T-H-M models.

In order to develop the excavation technology of disposal pit excavation, large diameter boring machine will be used for test pit opening. It is expected to minimize a disturbance of the surrounding rock mass and improve surface finish in the test pit. Mechanical and hydraulic properties of rock mass should be measured before and after opening the test pit.

Bentonite buffer will be filled in the test pit. Then, the quality of bentonite should be controlled on density, water content, and so on.

3. KAMAISHI EXPERIMENTAL SITE [1]

PNC has conducted the activities of geoscientific R&D program. In order to understand the deep geological condition in fractured crystalline rock, PNC initiated the in-situ experiments in the Kamaishi mine, where early Cretaceous granodiorite hosts the experiments [2].

The Kamaishi in-situ experiments have been carried out in two phases. Phase 1 (1988~1993) of the programme had been completed and Phase 2 (1993~1998) has been carried out. The in-situ experiments of Phase 2 have the following main objectives :

- To understand deeper geological environments in fractured crystalline rock,

- To improve technology and methodology to characterize geological environment.

Phase 2 is divided into five tasks as follows ;

Task 1 : Characterization of geological structure, groundwater flow, groundwater chemistry and geomechanics in a deep rock mass

Task 2 : Evaluation of excavation disturbed zone around the drift

Task 3 : Solute transport experiments in single fractures.

Task 4 : Engineered barrier experiments ; T-H-M experiment and grouting experiment

Task 5 : Study on earthquake

3.1 Geology [3]

The Kamaishi mine is located approximately 600km north of Tokyo (Figure 1). The Kamaishi mine has been a major source of iron ore for steel plants in the city of Kamaishi from the beginning of industrialization to the latter half of this century. The area surrounding the Kamaishi mine is geologically divided into two terrain near the boundary, called Hayachine-Goyo Tectonic Zone ; one with Southern Kitakami terrain with abundant limestone, and the other with Northern Kitakami terrain with abundant chert (Figure 2). The Kamaishi mine is situated in the north east part of the Southern Kitakami terrain. The bedrock in the area consists of Paleazoic sedimentary rock, Cretaceous sedimentary rock, and igneous complexes (Figure 3).

The Paleozoic sedimentary rocks have a nearly north-south strike, and are mainly composed of slate, sandstone, and limestone.

The Mesozoic sedimentary rocks are distributed in a north-south direction on the west side of the Kamaishi mine, and unconformably covers the Paleozoic rocks.

The igneous complexes are classified into followings according to rock type, distribution, and period of intrusion.

(1) Peridotite (ultrabasic rock) and the metagabbro or metadiabase (basic rock) that intrude along the Hayachine tectonic zone.

(2) Ganidake igneous complex, which is mainly composed of granodiorite (acidic rock), including diorites and monzonites. It is mainly distributed around Ganidake mountain (elevation 967 meters) located near the Kamaishi mine.

(3) Kurihashi granodiorite, which is mainly composed of granodiorite and is found near the Ganidake igneous complex.

The intrusion period of each igneous rock mass is in the sequence of (1),(2), and (3). The ages of these are: (1), about 450 million years ago, (2) and (3), about 120 million years ago.

The facilities of T-H-M experiment have been developed at the drift 550m meters above sea level (EL 550m drift) in the

Cretaceous-age Kurihashi granodiorite (Figure 4). The overburden thickness at this location in the mine is about 260 m.

3.2 Geological structure [4]

(1) Fracture zone

The fracture zone at Kamaishi have been estimated with existing geological-structure map made during mining activity, the lineament with the aerial photograph, and with the fracture observation of core and walls in the drifts. There are three fracture zones which have EW strike and steep dip at 550m level (Figure 5). The experimental area locates between Zone 2 and Zone 3.

(2) Fractures

The fracture mapping has mainly been done at the walls in the drift, drill core and borehole TV. The fractures, which have trace length not less than 3m, have been selected for the measurement on the walls in the drifts. The data on fractures observed on the walls in the drift consist of 17 items, for example, locations, strikes, dips, apertures, roughness, shapes, inflow rate, degree of alteration, widths of alteration zone, filling materials etc [5]. Figure 6-(a) is a equal area lower hemisphere projection of 886 fracture normals, sampled in the NW drift. Many fractures in the NW drift show NE strike and steep dip. Figure 6-(b) is a map of the fracturing near the T-H-M experimental site in the NW drift. A notable feature of the map is the large percentage of fractures which extend over more than two, and sometimes three walls of the drift.

3.3 Hydraulic properties [1]

The distribution of the pore water pressure along the depth from the surface do not correspond to a hydrostatic pressure gradient with depth, but gradually decrease as shown in Figure 7. The pore water pressure in the test area will be measured in detail in the test programme.

Hydraulic test was carried out in a borehole around the KD-90 drift using low pressure lugeon test. Rock mass in the test area has hydraulic conductivity ranging from 1.07×10^{-8}cm/sec to 1.51×10^{-5}cm/sec. The crosshole test proposed by Hsieh was performed in three horizontal boreholes. The results of the test indicated that the principal hydraulic conductivities K1, K2 and K3, were estimated to be 5.92×10^{-5}, 1.94×10^{-5} and 5.49×10^{-6}cm/sec, respectively.

The relationship between frequency and transmissivity distribution of conductive fracture around the KD-90 drift is estimated by the FracMan code with the results of packer test at some boreholes. The results indicates the conductive fracture frequency of 1.01 m^{-1} and the distribution for the transmissivity of lognormal distribution of which mean is 4.05×10^{-7} m^2/s and standard deviation is 2.82×10^{-5} m^2/s [4][6].

3.4 In-situ stress

Rock stress measurements have been performed in some boreholes at the 550m level drift and the 250m level drift with various methods, i.e. the overcoring method (the conical bottom method, the borehole deformation method), the hydraulic fracturing method, the AE method, the DRA method and the DSCA method. Table 1 shows one of the results performed by the overcoring method (the borehole deformation method) in three boreholes with different directions at 550m level. The maximum principal stress is about 2.4 times as large as 8MPa of the overburden at 550m level and nearly horizontal, N25°E.

3.5 Laboratory tests on rock samples

Mechanical and thermal properties of rock sample near the experimental area at 550m level are measured by laboratory tests. The specifications of tests and the properties are summarized in Table 2 and Table 3, respectively.

4. PROGRAMME OF T-H-M EXPERIMENT

The programme is divided into five phases.

- Excavation of Drifts
- Measurement of Rock Properties
- Excavation of Test Pit
- Setting up Bentonite
- T-H-M test

(1) Excavation of Drifts

The test area is shown in Figure 8. At first, the experimental drifts will be opened, 5m x 10m in square and 7m in height, by drill and blast method. After excavation, the experimental drifts will be fracture-mapped in detail including orientation, filling, water inflow, and surface characteristics.

(2) Measurement of Rock Properties

The objective of this phase is measurement of properties of rock mass before excavation of a test pit.

Borehole set 1 : Six boreholes for hydraulic test will be drilled. These boreholes will be 66mm in diameter, 8m depth, and 60cm apart at one direction (Figure 9). A borehole television system will be used to determine the geometry of joint. Five piezometers will be set in each inner borehole. Hydraulic test will be performed at several points in the outer boreholes using low pressure lugeon test and the pressure will be monitored in each inner boreholes in order to provide the basis for determining the continuity of joint. After hydraulic test, five piezometers will be set in each outer borehole. The pressure will be monitored in all boreholes continuously during the drilling of other boreholes and the opening of test pit.

Borehole set 2 : A borehole for crosshole hydraulic test and borehole expansion test will be drilled at the center. These boreholes will be 76mm in diameter and 8m depth after drilling of Borehole set 1 (Figure 9). A borehole television system will be used to determine the geometry of joint. After low pressure lugeon test and setting of the piezometers in Borehole set 1, crosshole hydraulic test will be carried out in order to provide the basis for determining the continuity of joint. The strength - displacement property of rock mass will be measured using borehole jack system at three depths.

Borehole set 3 : The boreholes for the strain and temperature measurement will be drilled with a diameter of 66mm and a depth of 8 meter at 120º apart (Figure 9). A borehole television system will be used to determine the geometry of joint. The strain gauges and thermocouples will be set in the boreholes. After that, the packers will be put in Borehole set 3.

Borehole set 4 : Two boreholes for monitoring of rock deformation will be drilled (Figure 9). A borehole television system will be used to determine the geometry of joint. The extensometer will then be installed in the boreholes.

Borehole set 5 : Two boreholes for monitoring of joint deformation will be drilled. Each borehole will be located in the position crossed by a joint. A borehole television system will be used to determine the geometry of joint. The joint deformeters [7] will then be installed in the boreholes, by which a linear variable displacement along three directions can be simultaneously monitored.

Laboratory tests with core samples will be carried out to set input data of the calculation, shear strength of joint and so on.

(3) Excavation of Test Pit

A test pit will be drilled, 1.7 m in diameter and 5 m in depth, by the shot boring method using large diameter boring machine to avoid a disturbance of the surrounding. The shot boring method is that shot, which is small iron beads sent with water into the crown of the casing pipe, is ground by the crown thickness with the casing pipe rotation and rock mass is drilled by the edge of the ground shot (Figure 10). The test pit will be fracture-mapped in detail including orientation, filling, inflow rate, and surface characteristics. The information on the pressure responses and logging data at the surrounding boreholes will provide the basis for determining the continuity of joint.

Borehole set 6 : If the groundwater will flow into the test pit, some boreholes for pumping up of groundwater in order to fill the bentonite will be drilled. These boreholes will be 66mm in diameter and 7m in depth (Figure 9). Each borehole will be located in the position crossed by a joint. A borehole television system will be used to determine the geometry of joint. The packers will then be installed in the boreholes. The groundwater in Borehole set 6 will be pumped up to fill the bentonite in the test hole.

(4) Setting Up Bentonite

The mixture of sodium bentonite (KUNIGEL-V1 [8]) of 70 % and quartz sand of 30 % will be packed directly in the test pit with monitoring sensors and an electric heater. The mixture is compacted to an average wet density of 1.86 g/cm^3 (1.6g/cm^3 in dry density) at a water content of 16.5%. Figure 11 shows the schematic view of test pit.

Sensors to monitor temperature, heat flux, water content, pore water pressure, displacement of the heater, swelling pressure and strain will be installed in the mixture. The measurements will be made hourly with a personal computer. These instruments are arranged within the three vertical cross sections through a center axis of the test pit. An arrangement of the sensors set in the mixture is shown in Figure 11.

The electric heater will be set in the mixture, which will be about 1 m in diameter and about 2 m in height.

After setting up the mixture with the heater and the sensors, a concrete lid will be put on the test pit.

A fence will be constructed with a height of 60 cm at the entrance of T-H-M experimental area and the experimental area will be curtained off. The flow rate at the fence and humidity in the experimental area will be monitored to specify the upper hydraulic boundary condition. Temperature and heat flux will be monitored on the surface of rock till the end of the test to specify the upper heat boundary condition (Figure 12).

(5) T-H-M Test

The heating will be started in the middle of 1996.

If groundwater will not flow into the test pit in spite of stopping the pumping up of groundwater, groundwater will be injected into the boreholes till the end of the test. If the test area will be unsaturated from the Setting Up Bentonite stage, some boreholes will be drilled and groundwater will be injected.

Temperature, water content, and swelling pressure of the bentonite mixture under coupled T-H-M condition will be monitored till the end of test. Hydraulic test will be performed at the end of T-H-M test to understand an effect of coupled T-H-M process on rock mass.

(6)Others

The investigation of microbe in the test pit and corrosion damage at the surface of the heater will be made before and after T-H-M test.

A preliminary schedule is given in Table 4.

This plan will be optimized with a progress of the test.

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Table 1 Magnitude and Direction of Principal Stress

Item	Stress (MPa)	Direction
Maximum principal Stress	19.2	N25°E2S
Medium principal Stress	17.1	N20°W30S
Minimum principal Stress	6.0	N52°W60N

Table 2 Specification of Laboratory Tests

Test	Sample (mm)	Method	Item
Uniaxial compressive test	60(ϕ)x 120(h)	displacement controlled (1μ/sec)	Uniaxial compressive strength, Young's modulus, Poisson's ratio
Triaxial compressive test	60(ϕ)x 120(h)	displacement controlled (1μ/sec) compressive cell pressure; 10,15,30 MPa	Triaxial compressive strength, Young's modulus, Poisson's ratio, Volumetric modulus, Cohesion, Friction angle
Shear test		ISRM guide normal stress; 0.075, 0.3, 0.5 MPa	Shear stiffness Cohesion, Friction angle
Braziliam test	60(ϕ)x 30(h)	ISRM guide	Tensile strength
Thermal conductivity test	50(ϕ)x 250(h)	Box probe method, 20°C 60°C 90°C	Thermal conductivity
Heat capasity test	50(ϕ)x 250(h)	Mixed method	Heat capasity
Thermal expansion test	50(ϕ)x 250(h)	Lamp scale method	Coefficient of thermal expansion

Table 3 Mechanical and Thermal Properties

Uniaxial compressive strength U_c	120 MPa
Young's modulus E	5.7×10^4 MPa
Poisson's ratio ν	0.3
Tensile strength U_t	11 MPa
Volumetric modulus β	2.5×10^4 MPa
Cohesion (intact) c	22 MPa
Friction angle (intact) ϕ	52 °
Cohesion (joint) c	0 ~ 0.291 MPa
Friction angle (joint) ϕ	41.3 ~ 71.1 °
JRC	2.9 ~ 9.2
Thermal conductivity λ	2.71(20°C) 2.61(60°C) 2.54(90°C) W/mK
Heat capasity C	0.199(dry) cal/gK
Coefficient of thermal expansion α	8.21×10^{-7} /K

Table 4 Schedule of T-H-M Experiment

		1993	1994	1995	1996	1997	
Planning							
Excavation of Drifts							
Measurement of Rock Properties	Opening boreholes and borehole TV						
	Hydraulic tests						
	Borehole expansion tests						
	Install monitoring sensors						
	Monitoring						
	Laboratory tests						
Excavation of test pit	Excavation of test pit and Mapping joints						
	Pumping						
	Manufacture of heter						
Setting up Bentonite	Filling Bentonite						
	Install heter and monitoring sensors in bentonite						
	Pluging						
T-H-M test	Heating						
	Monitoring						
	Sampling						
	Hydraulic tests						

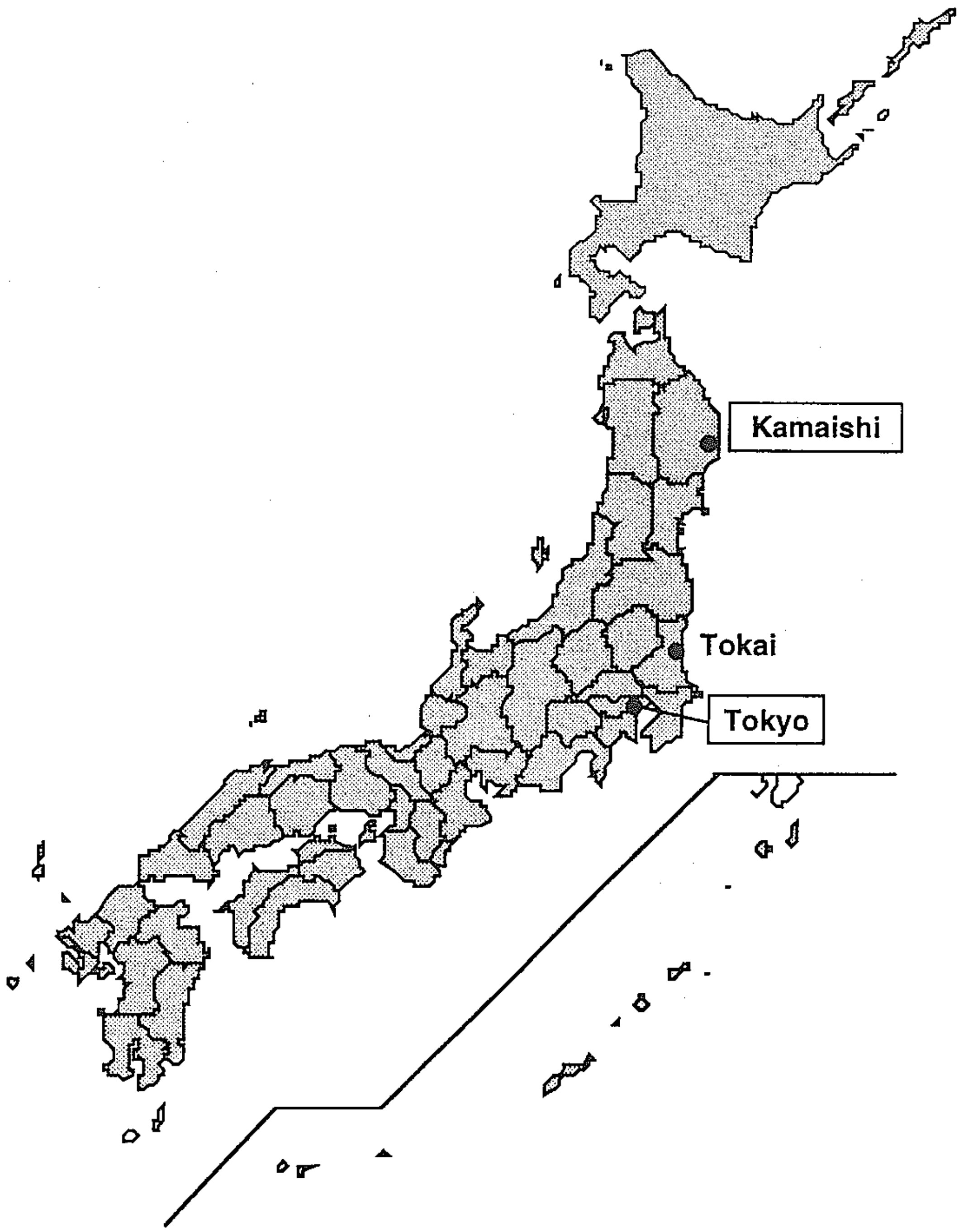


Figure 1 Location of Kamaishi

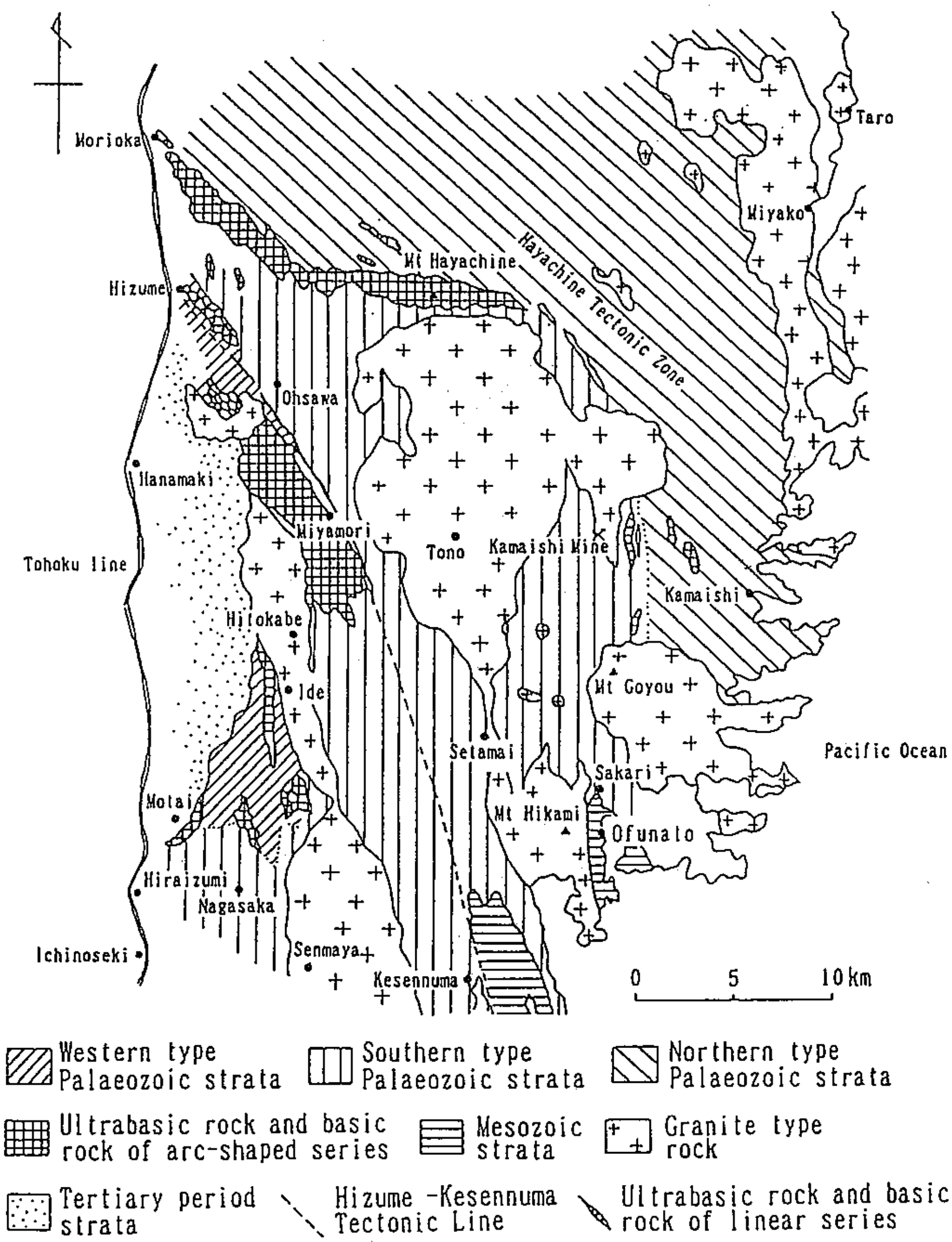


Figure 2 Geologic map of Kamaishi and environs

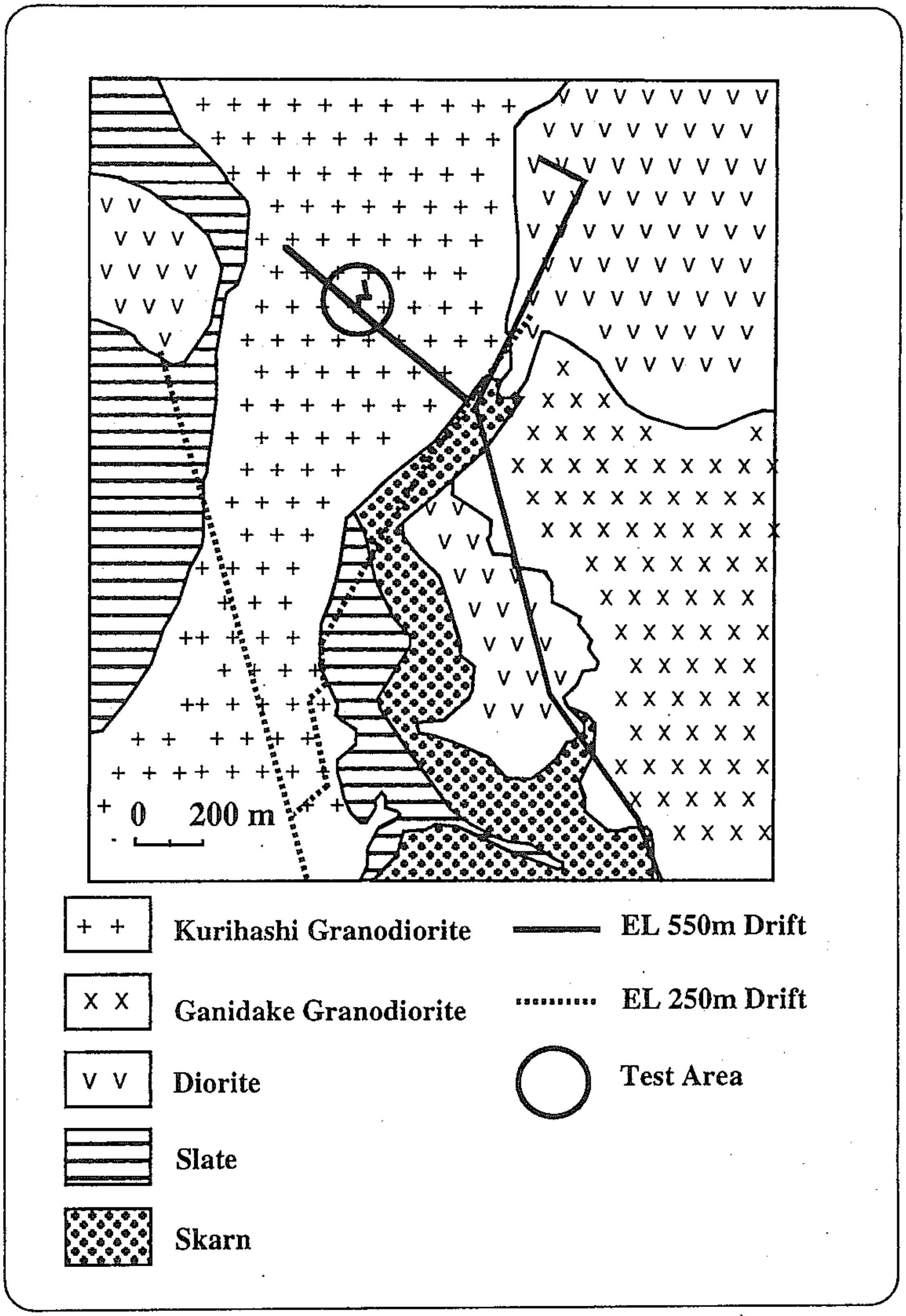


Figure 3 Geological Map(550 m level)

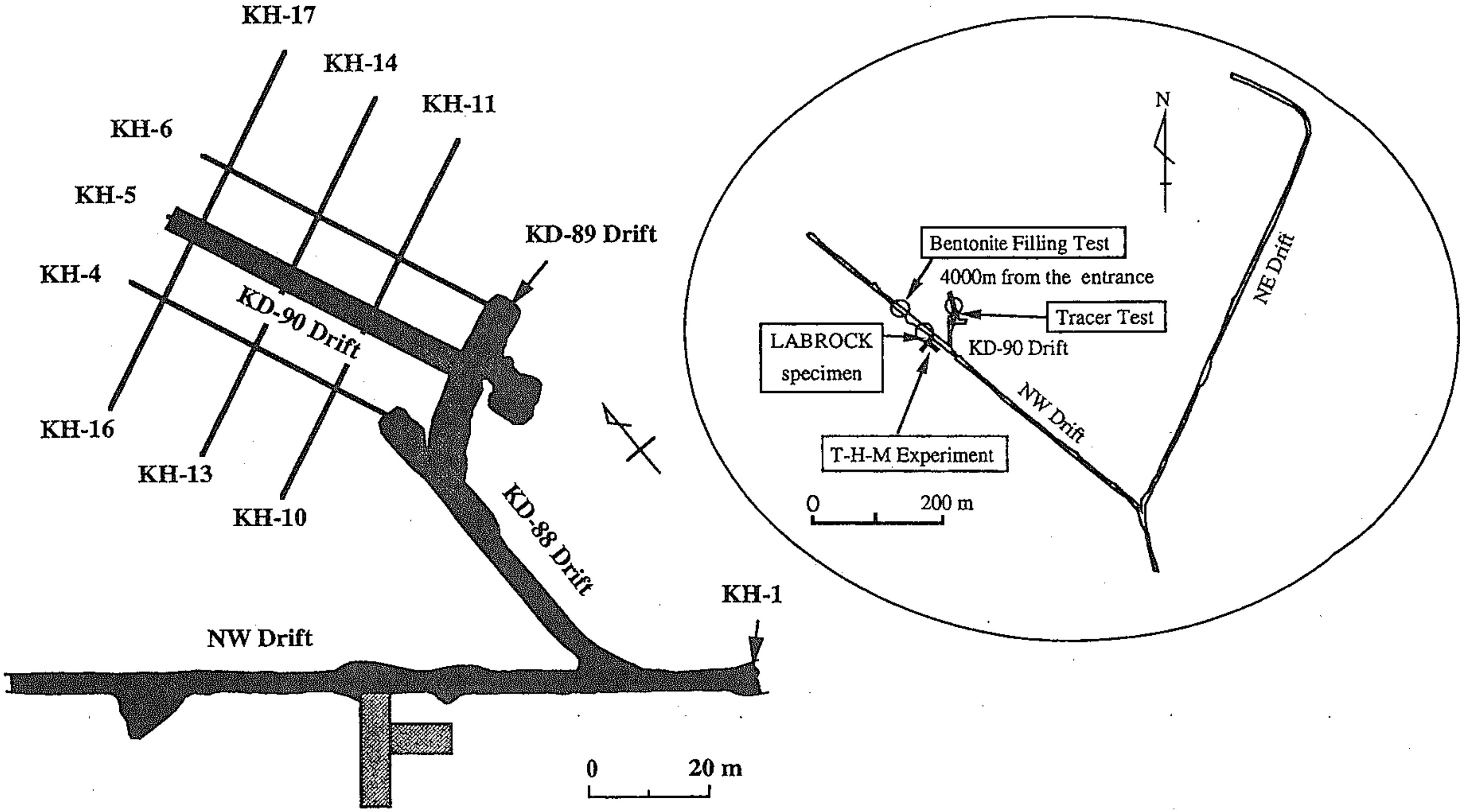


Figure 4 Location of the Test Drift and the Boreholes

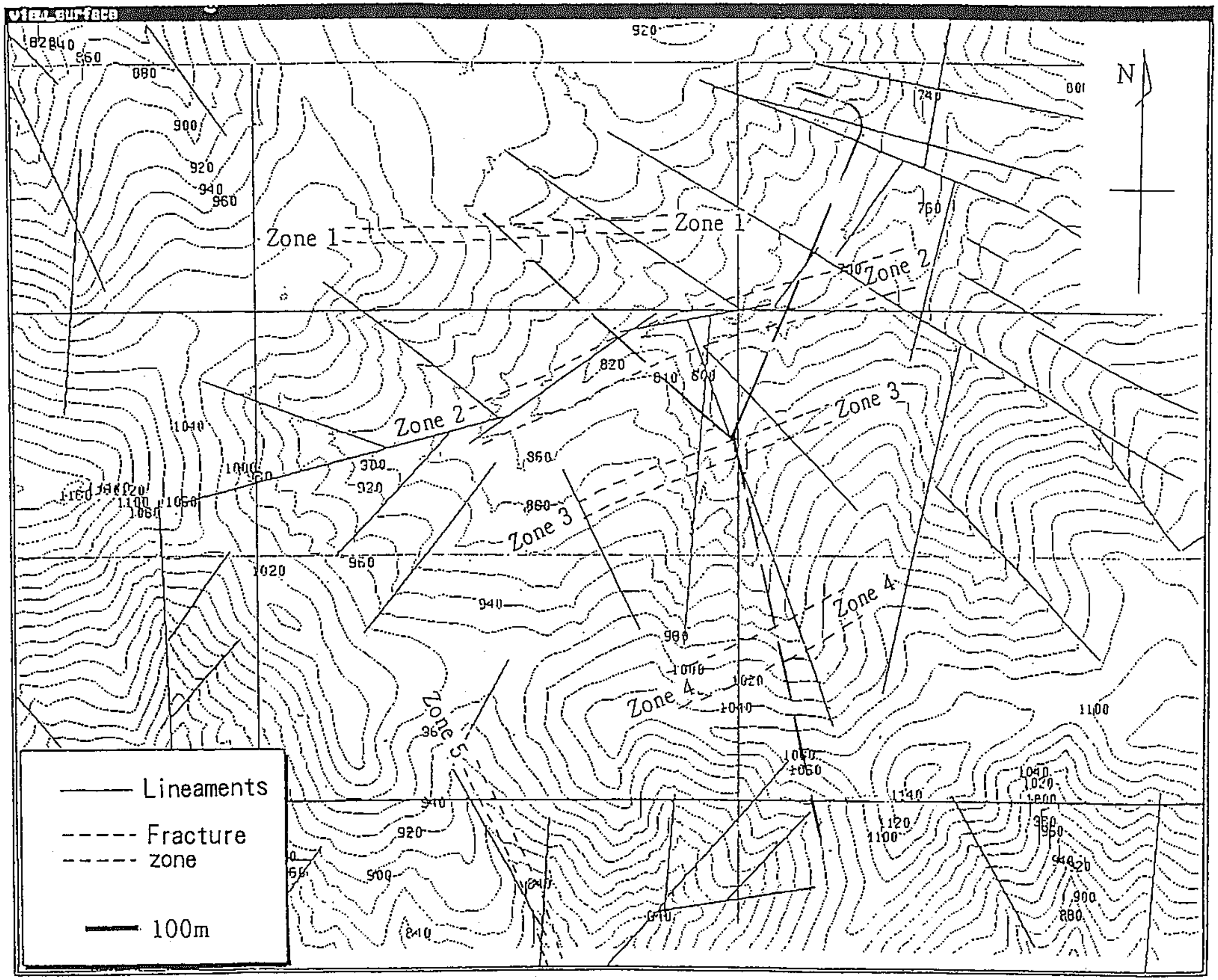


Figure 5 Fracture zone at the 550m level

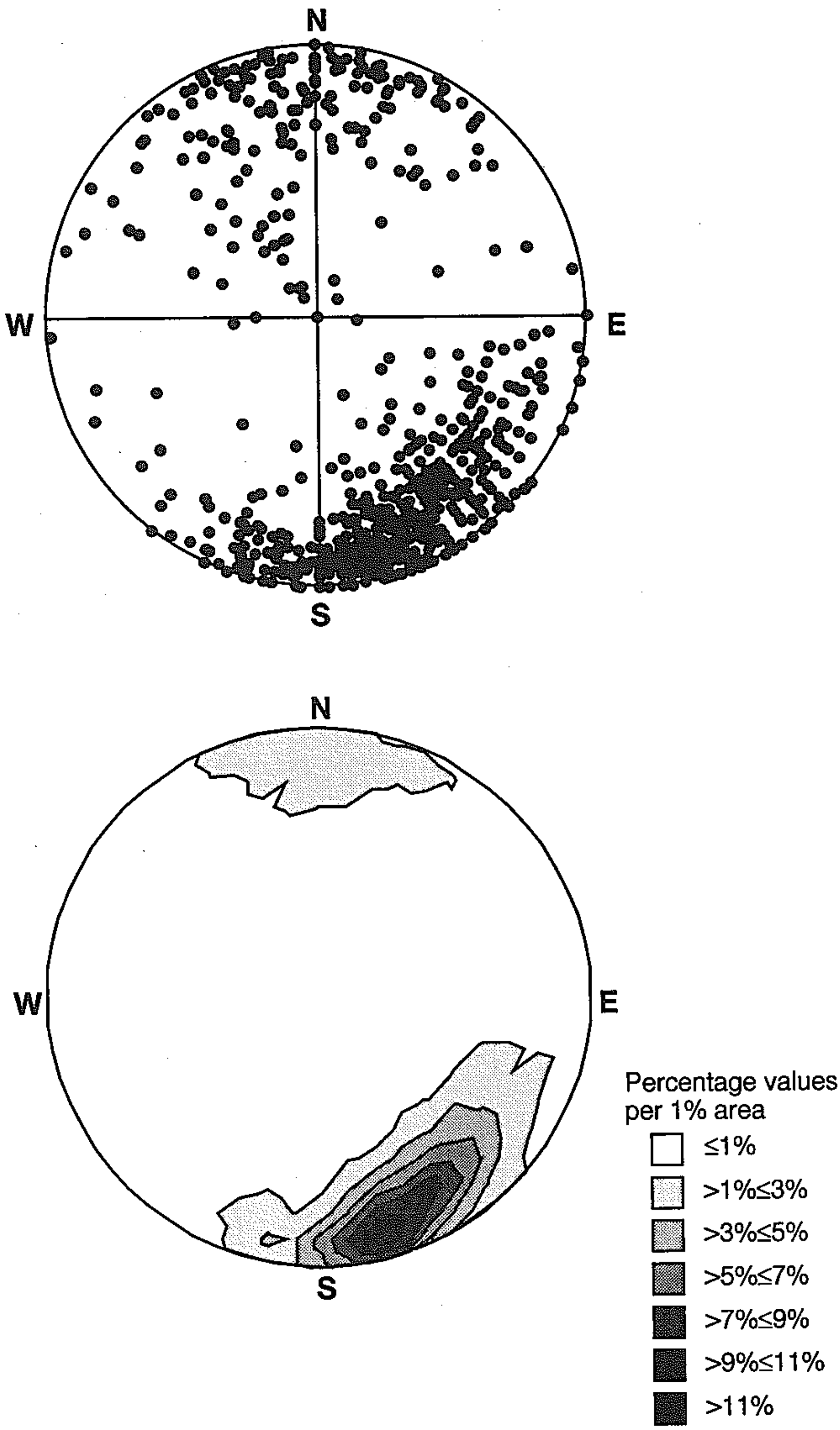


Figure 6-(a) Equal area lower hemisphere projection of 886 fracture normals at NW drift

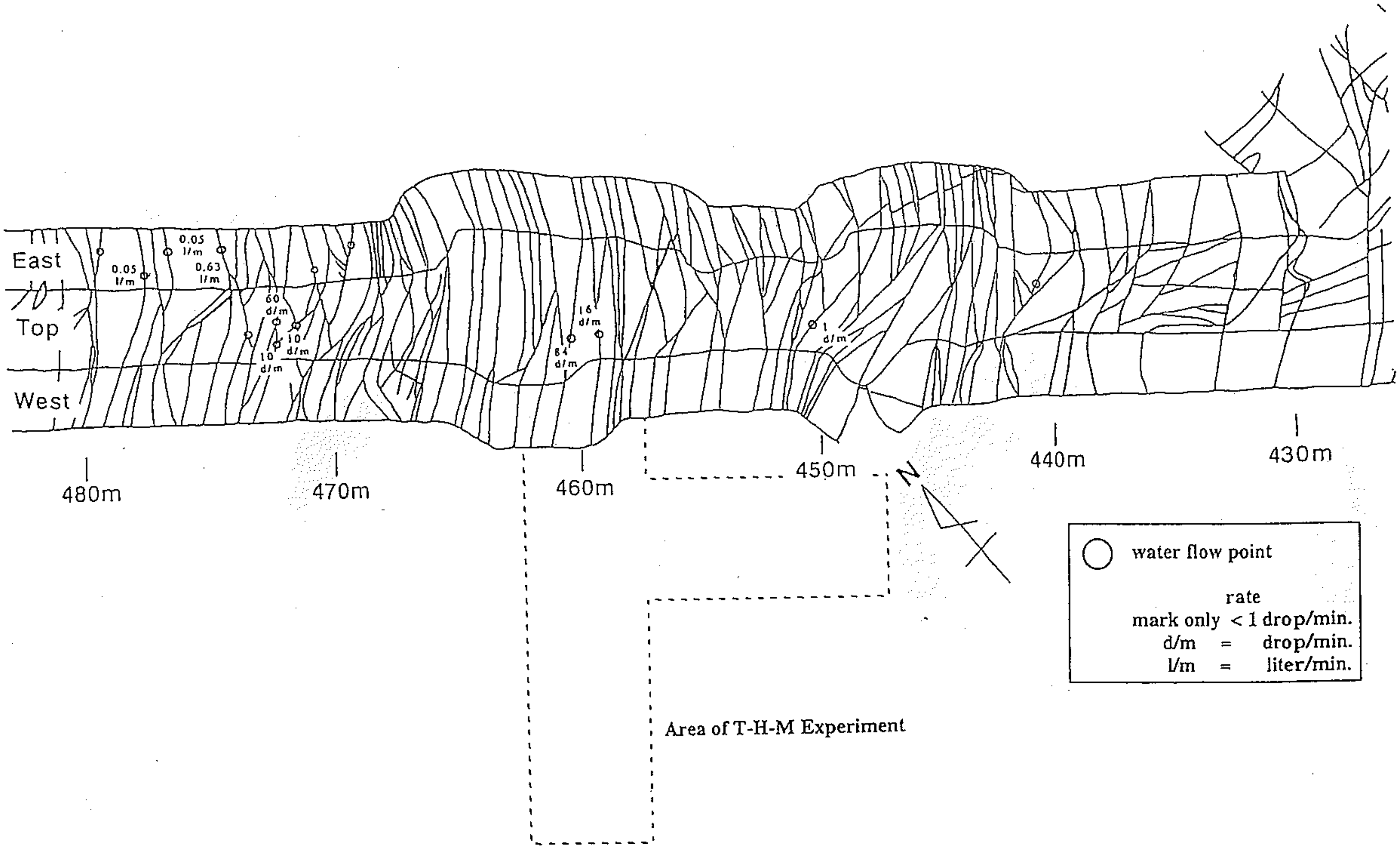


Figure 6-(b) Trace map of NW drift

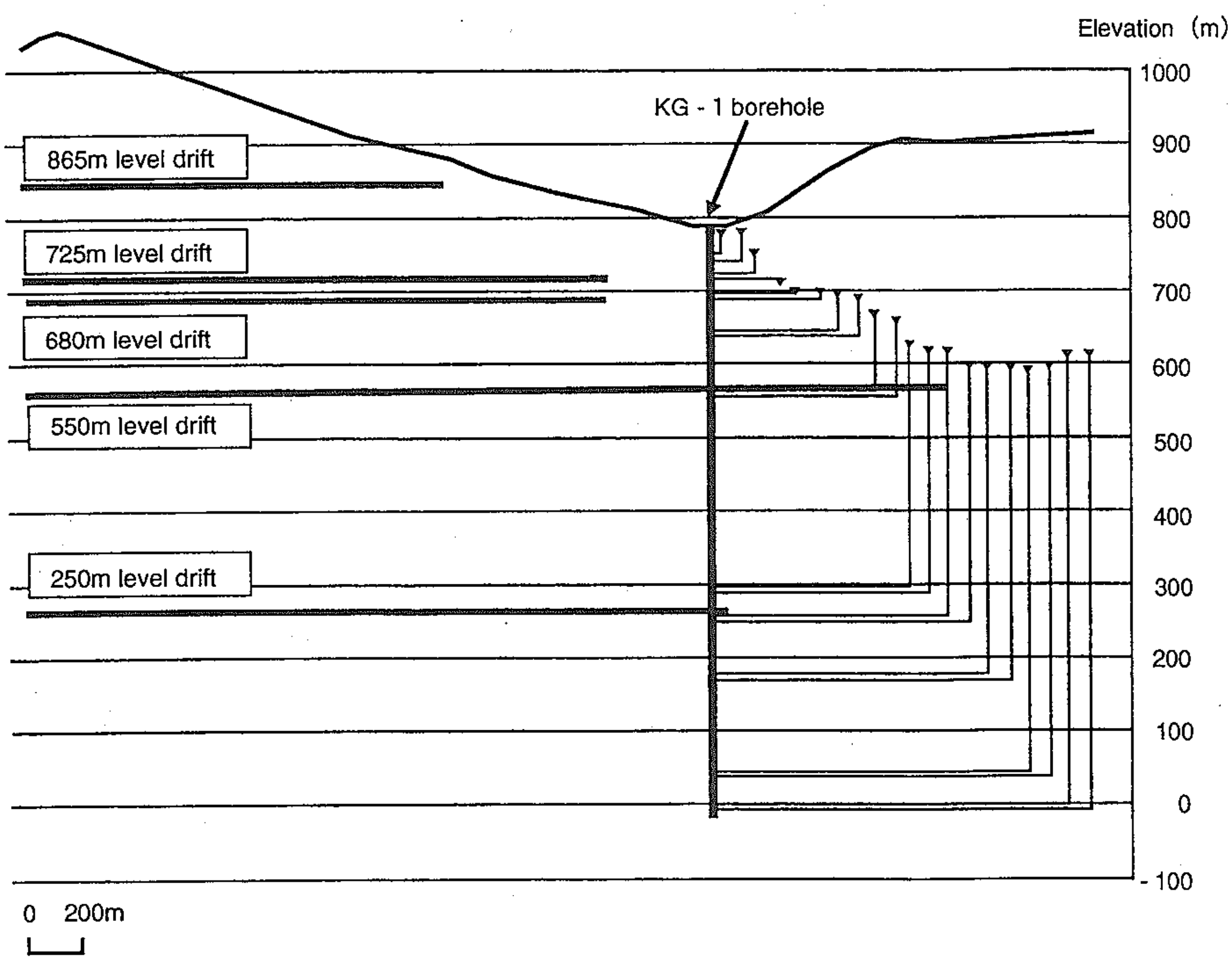


Figure 7 Distribution of pore water pressure along the depth from the surface

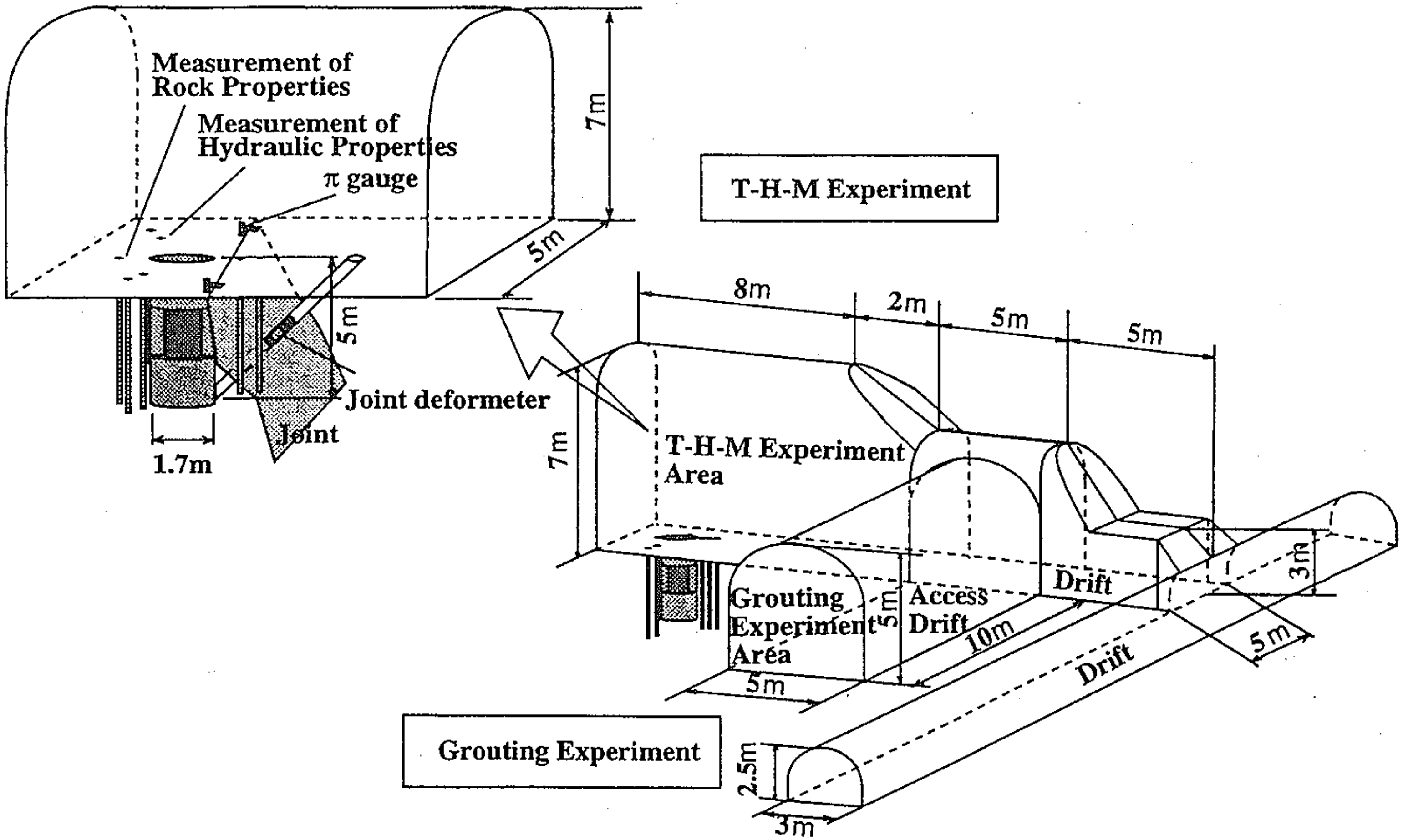


Figure 8-(a) Layout of Engineered Barrier Experiment

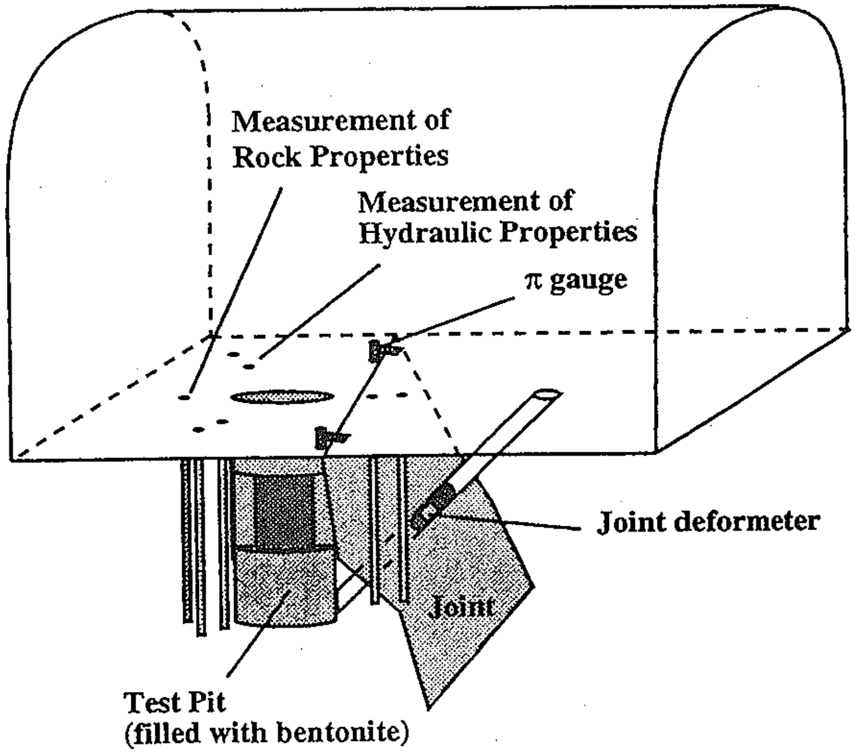
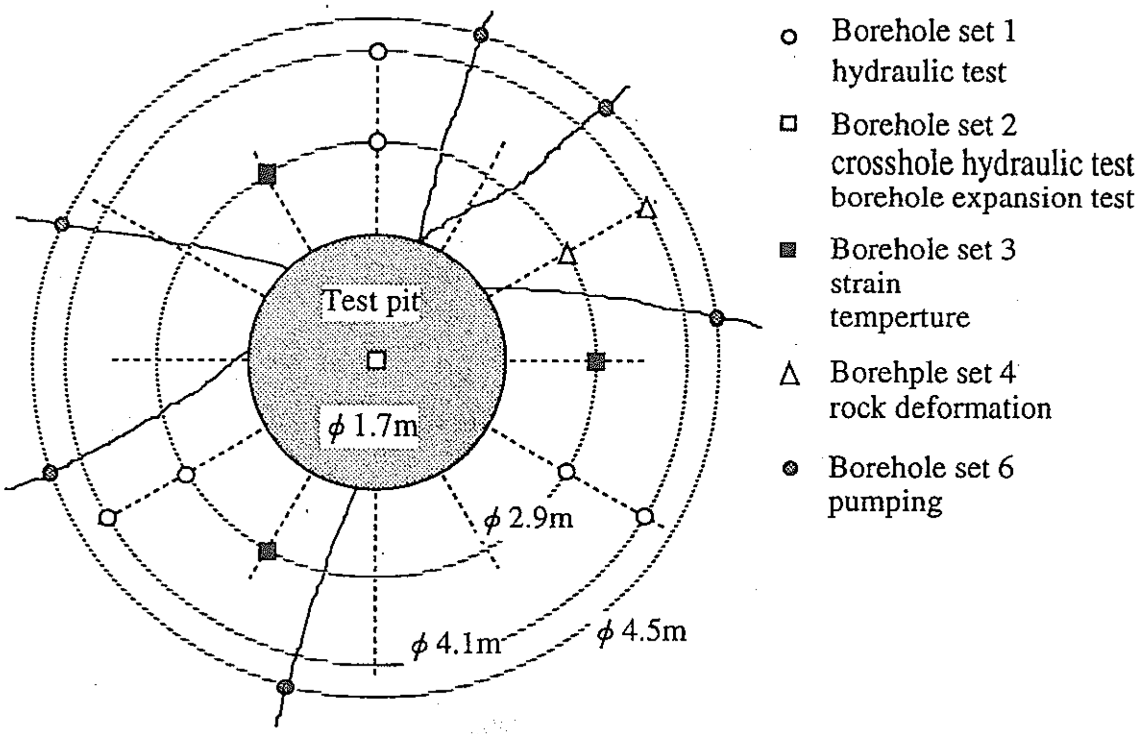


Figure 8-(b) Layout of T-H-M Experiment



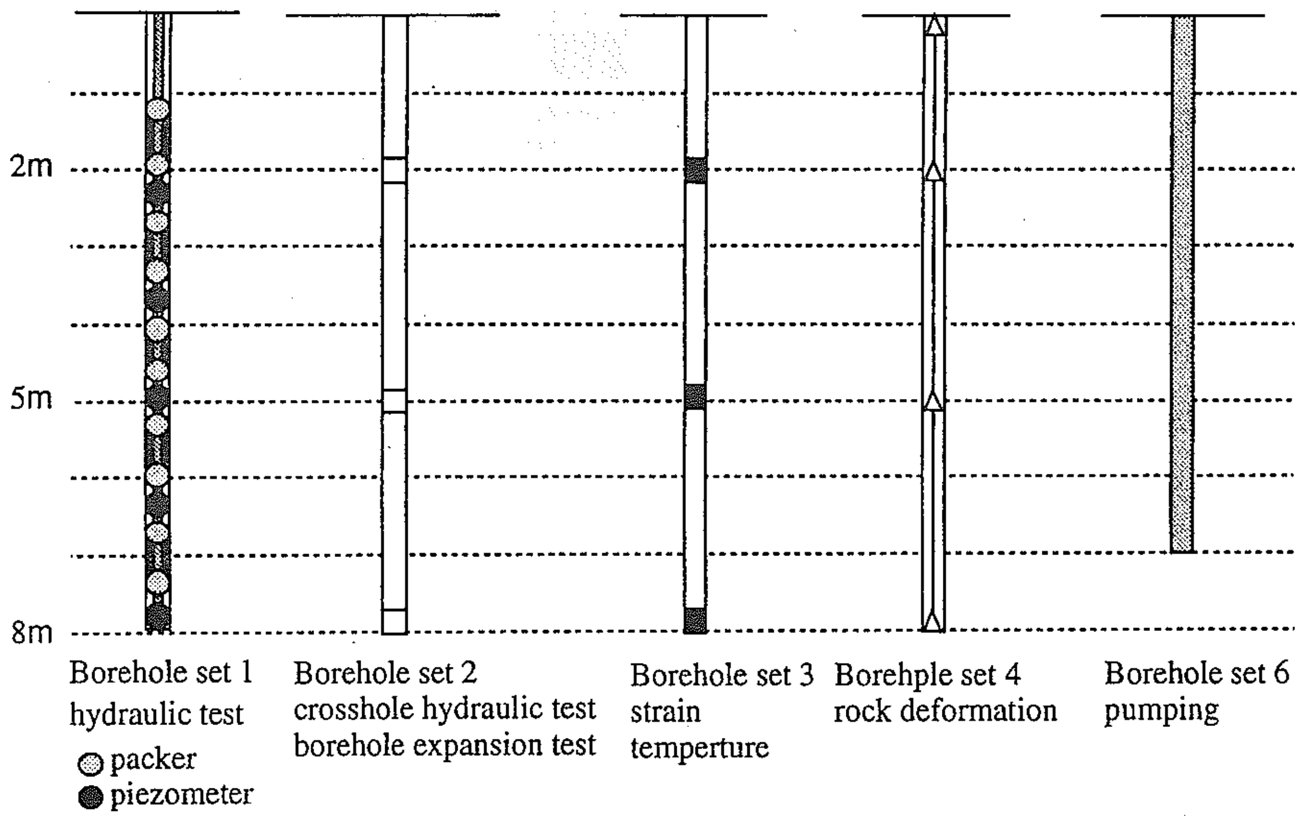


Figure 9 The Sensors in the Rock mass

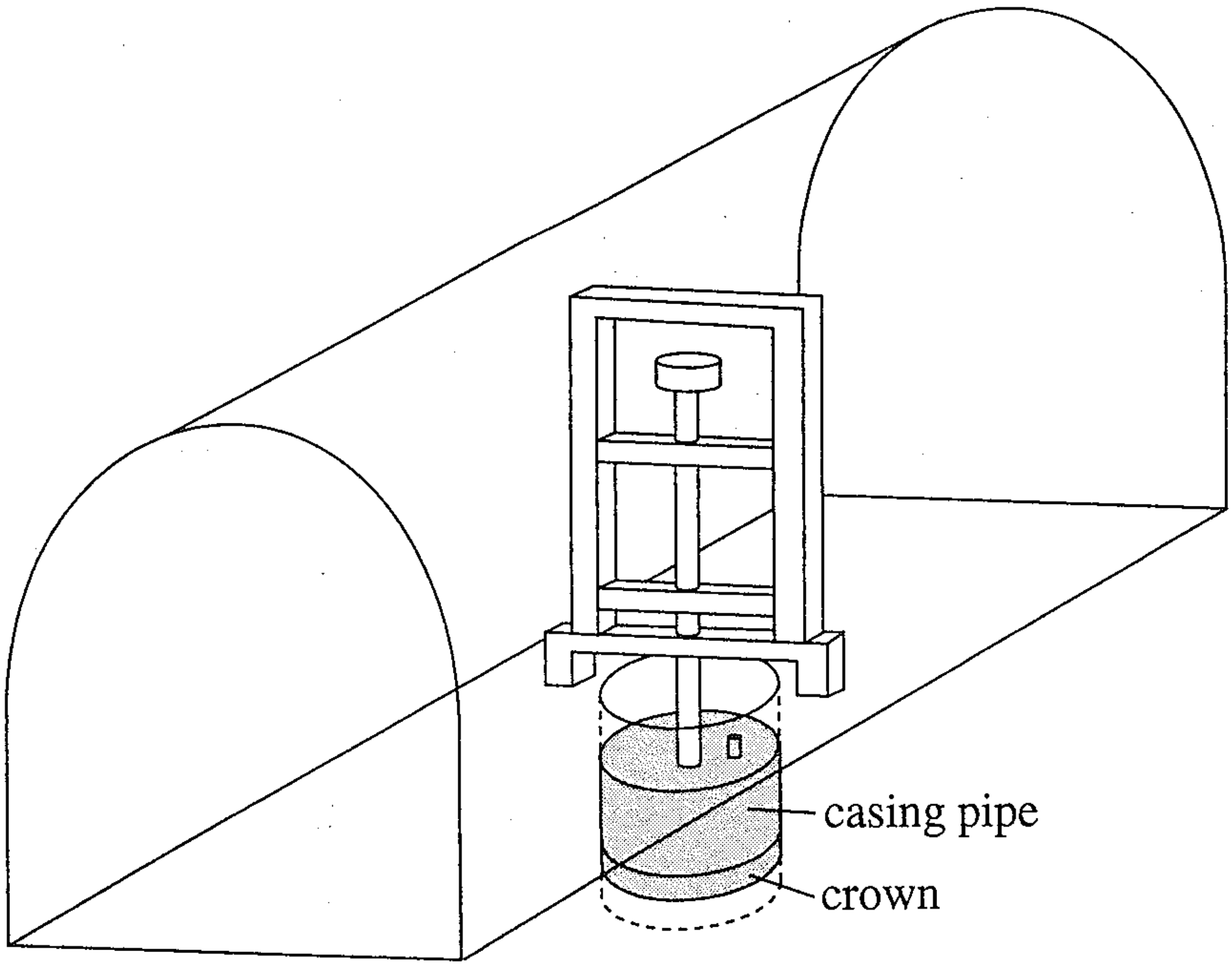


Figure 10 Large diameter boring machine

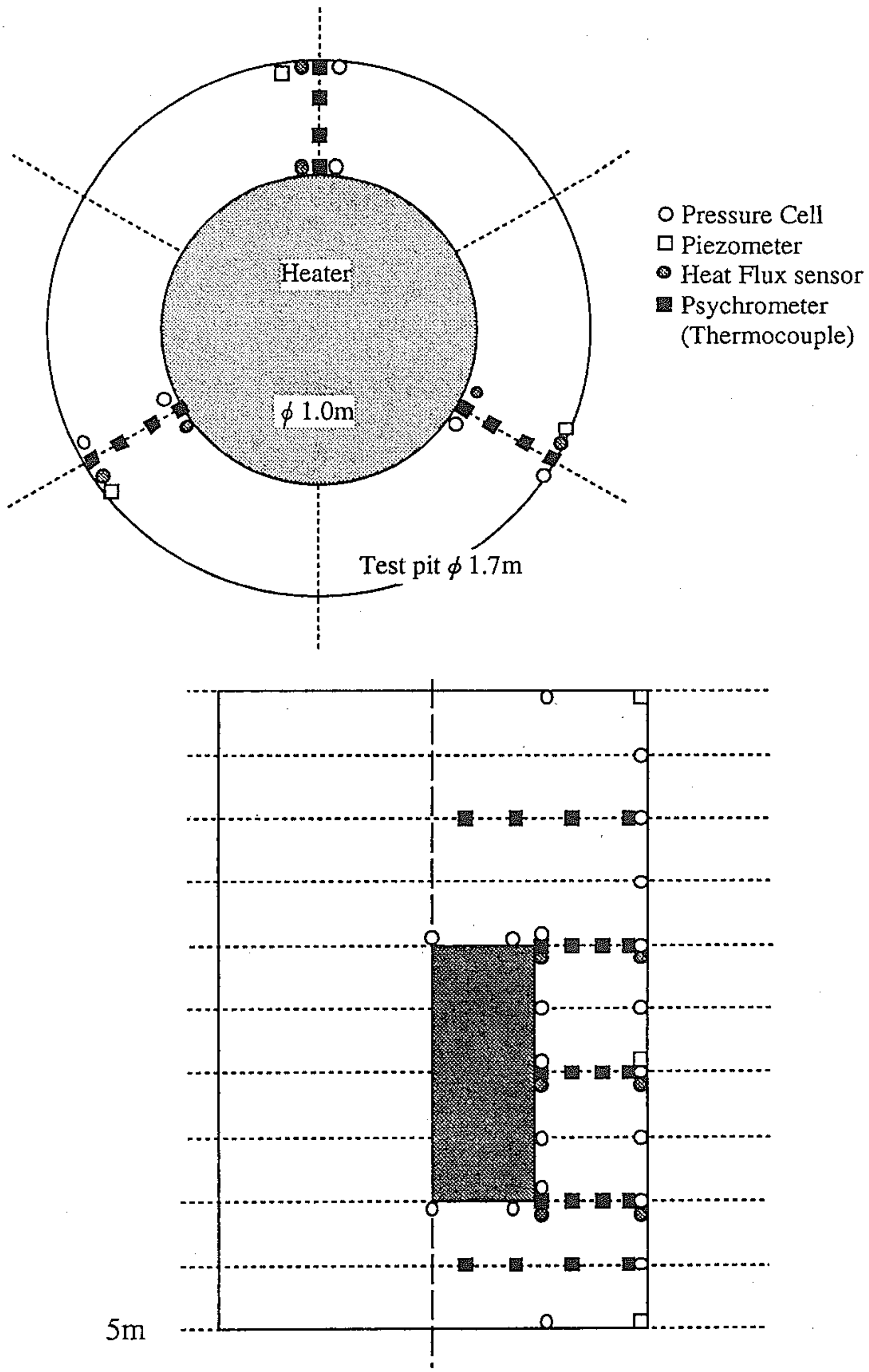


Figure 11 The sensors in the Buffer material

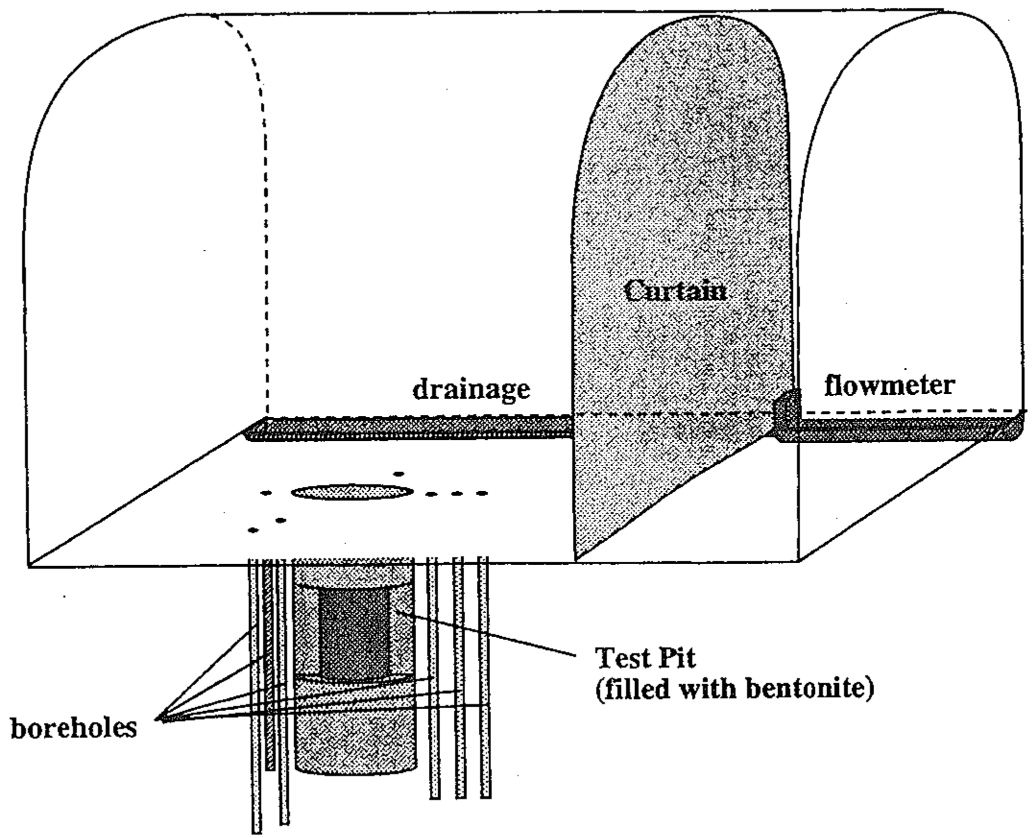


Figure 12 Schematic view of boundary condition