KBS TEKNISK RAPPORT

Groundwater movements around a repository

Geological and geotechnical conditions

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KÄRN – BRÄNSLE – SÄKERHET

GROUNDWATER MOVEMENTS AROUND A REPOSITORY

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Hagconsult AB september 1977

TECHNICAL REPORT 1

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KBS – Kärnbränslesäkerhet

GROUNDWATER MOVEMENTS AROUND A REPOSITORY

Technical Report 1: Geological and geotechnical conditions

Hagconsult AB
in association with
Acres Consulting Services Ltd
RE/SPEC Inc.

FOREWORD

This report was prepared as one of a series of technical reports within
a study of the groundwater movements around a repository for radioactive
waste in the Precambrian bedrock of Sweden. The contract for this study
was between KBS - Kärnbränslesäkerhet (Project Fuel Safety) and Hagconsult AB
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to the KBS project.

The opinions and conclusions in this document are those of the authors and
should not be interpreted as necessarily representing the official policies
or recommendations of KBS.

Stockholm September 1977

Ulf E. Lindblom
Study Director
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1.   INTRODUCTION

This assessment is intended to provide basic geotechnical data for the
analysis of groundwater flow around a repository situated in the Fennoscandian
shield of Precambrian bedrock. These data include properties and conditions
that are representative of the intact rock, the rock mass in general, and the
groundwater regime. As there exist a considerable range in the mineralogy of
potentially suitable plutonic rocks and since a specific site has not yet been
selected, all of the parameters presented in this report must be based on
presumptive geological and hydrogeological conditions. Where possible, data
for two potential site areas, namely Oskarshamn and Forsmark, are presented.

This report is divided into four parts. First, a brief description of the pro-
cedure for modelling groundwater movements is presented, along with a tabulation
of the important parameters. Secondly, a description of the geological and hydro-
geological conditions of the Fennoscandian shield, as well as of the two general
site areas,is given. The final two sections of the report provide thermo-
mechanical and geohydrological characteristics and properties of the host   rock.

2.   SUMMARY OF RELEVANT PROPERTIES AND CONDITIONS

During each of the four time spans of a repository, namely those of pre-mining, mining (but pre-emplacement of radioactive waste), short-term conditions, and long-term conditions, the factors affecting the groundwater flow are different and must be treated separately. The basic philosophy of modelling in this project at this time is to study discrete but related phenomena using existing computer programs,with only relatively simple modifications to the programs-and model development as required. There is not sufficient time for complex program development, and the sparsity of data precludes use of one comprehensive model containing all desirable features. The model used in this project is based on uncoupled programs for heat transfer, rock mechanics, and groundwater flow. Full coupling is developed between the programs for heat transfer and groundwater flow, with partial coupling between heat transfer, rock mechanics and groundwater flow. The groundwater flow is coupled to the heat transfer through thermal convection and advection, and to the rock mechanics through the permeability of the joints,which depends on the deformation of the joints.

The geotechnical parameters considered to be of importance in assessing the thermomechanical and hydrogeological behaviour are listed below. All the parameters can depend on several factors, including stress, time, temperature and the geological conditions of a specific site.

(a) Termal properties and conditions for the rock mass

- Thermal conductivity
- Specific heat capacity
- Density
- Coefficient of thermal expansion
- Convective film coefficient for heat transfer between the rock matrix and pore water

(b) Mechanical properties and conditions for the rock mass

- Young's modulus of elasticity
- Poisson's ratio
- Density
- Quasi/static compressive and tensile strengths of the intact rock mass including the failure envelope

(c) Mechanical properties and conditions for joints in the rock mass

- Cohesion and friction angle (or, the failure envelope)
- Shear stiffness
- Normal stiffness
- Creep behaviour

(d) Hydrogeological properties and conditions of the rock mass

- Joint widths and spacings
- Density and viscosity of pore water
- Permeability of the joint
- Permeability of the fractured rock mass

The properties of the jointed rock mass when treated as a continuous material can be evaluated from the properties and conditions of the intact rock and the joints. The pertubation of the groundwater flow will depend on the heat-generating characteristics of the radioactive waste, the repository geometry, and emplacement schedules, and on the thermomechanical behavior of the rock mass due to excavation and thermal loading.

The initial conditions are necessary for the calculation of the groundwater flow coupled to the thermal loading and rock mechanics. The following conditions have to be established.

(e) Initial conditions

- Initial stress field
- Initial flow field
- Initial temperature field
- Topography
- Initial permeability and porosity distributions

(f) Gross geological and environmental changes

- Changes in the stress field
- Changes in the flow field
- Changes in the temperature field

Many of the listed parameters are strongly depending on the geological and hydrogeological conditions at a specific site and of course they will also depend on the history of the rock mass.

In Table 1, the properties have been listed in conjunction with such geological considerations as

- Rock type
- Joints
- Geological and hydrogeological history
- Geological future

TABLE 1   Geotechnical parameters for various geological considerations

Geotechnical parameters / Geological considerations	Thermal properties of the rock	Mechanical properties of the rock	Mechanical properties of the joint	Hydrogeological properties	Initial conditions
Rock type	Specific heat Thermal conductivity Thermal behaviour of rock matrix	Young´s modulus Poisson´s ratio Density Failure envelope Creep behaviour			Density
Joints:					
Filling material	Film coefficient		Failure envelope Stiffness	Hydraulic conductivity	
Width			Failure envelope Stiffness	Hydraulic conductivity	
Roughness and undulation	Thermal conductivity from rock block to rock block		Failure envelope Normal stiffness Shear stiffness Creep behaviour	Hydraulic conductivity	
Spacing				Hydraulic conductivity	
Geological and hydrogelogical history		Anistropy		Water conditions	Initial temperature field Initial stress field Initial flow field
Erosion Glaciations Weathering Tectonics		Creep behaviour	Creep behaviour		Changes in: stress field flow field temperature field

3.   GEOLOGICAL CONSIDERATIONS

3.1   Geological Isolation

Historically,the scientific study of geology has been based upon the
"Principle of uniformity" which states that the present is the key to
the past. The logical complement to this principle is that the past and
present are the key to the future. It is on this latter premise that the
study of geological isolation of waste must be based.

There are a number of examples of natural isolation available for study.
Such isolations, oil and gas deposits rock chambers and tunnels, which have
been made by the nature itself or by the human beeing and have been stable
during very long time.

The experiences from mining in Swedish bedrock have shown that in specific
cases rock chamber with a width of the cavern up to 100 m has been made.
Many of the natural made rockslopes have such an height and steep slope
that it is remarcable that they stand stable.

Their existance thus indicates that, under favourable conditions, stable
underground structures can be acheived for time scales which are significant.

3.2   The Baltic Shield

The Fennoscandian shield of Precambrian bedrock dates back more than 1700
million years and is composed of the "roots" of very old eroded rock
formations.

Dominating rocks in the shield are those of the metamorphic type, e.g.
gneisses, and magmatic type, e.g. granites, and recrystallized surface
rocks. In addition, there are sedimentary rocks, e.g. sandstones, shales
and limestones consolidated to quartzites, and Precambrian limestones,

dolomites, shales and leptites, and volcanics, such as tuffs. These rocks exemplify the earth's oldest rock formations. There are many areas with these types of rocks on the earth's crust in spite of several periods of mountain formation.

The Precambrian is classified according to the main age of deformation. The earliest orogenic event is the Svecofenno - Karelian which has been dated at 1950-2600 my. Rocks of this age constitute the major portion of the Baltic Shield, underlying most of Central Sweden east of the Caledonides, from Blekinge in the south to Finnish Border in the north.

The oldest Svecofennian rocks belong to the leptite - hälleflintas series. They are metamorphosed acid volcanic extrusives (tuffs and lavas) and intercalated sedimentary strata (dolomites and limestones).

The leptite - hälleflintas series is succeeded by meta-sediments. Metamorphosed greywackes are typical of this period, with lesser amounts of sandstones, quartzites, schists, and volcanics, both acid and basic. The Mälar formation belongs to this period. The sediments are typical of geosynclinal and miogeosynclinal depositional environments. Intense folding took place as the geosyncline evolved. At depth, the rocks reached fusion point (anatexis). The supra crustal rocks thus became metamorphosed to granites, granodiorites and tonalites. The Dala - Uppland batholith is typical of these synorogenic plutonics (Stephansson, 1   ). Late Sveofennian regional metamorphisan resulted in the formation of gneisses and migmatites from the remaining supracrustal rocks. At the same time, serorogenic diapiric intrusions resulted in granitic plutons. The Stockholm and Fellingsbro granites are typical of this period.

The Svecofennian event was succeeded by the Gothian, which has been dated at 1200-1750 my. They occur in the southeastern part of Sweden with an

extension north-northeast to the Norwegian border with
another area extending north from Göteborg to the Norwegian border. The
oldest Gothian rocks consist primarily of acid lavas (rhyolites, andesites,
porhyries) with some sedimentary rocks (quartzites and conglomerates).
These rocks are succeeded by granites, granodiorites and tonalites with
minor dioritic and gabbrioc intrusions, known as the Smålands – Värmlands
and Åmåls – Kroppefjälls granite group. Dating of some of the early Gothian
rocks is complicated by the Dalsandian Regeneration under which the Gothian
rocks of southwestern Sweden (the Åmål – Kroppefjäll suite) were re-
metamorphosed. Age dating by radioisotope methods identifies only the
latest metamorphic event.

Following the intrusion of the granitic suite, there was intense tectonic
activity resulting in mylonization and formation of schists. This was
followed by further upplifting and plastic folding.

The Gothian era culminated with the intrusion granites, including the
Karlshamns – Spinkamåla group and the rapakivi like granites of Ragunda
and Nordingrå in Central Sweden. The Götemar pluton near Oskarshamn has
also been interpreted as belonging to this period (Magnusson, 2).

The Gothian era was succeeded by the Dalslandian, dated at 850 to 1200 my.
During the Dalslandian, rocks ascribed to the Gothian and Svecofennian
age in southwestern Sweden were re-metamorphosed (Dalslandian regeneration).

Diabase hyperite dykes are widely developed south from Vättern to Skåne.
The dykes are oriented mostly north-south. In Dalsland area west of Lake
Vänern, supracrustal of the Dal formation is well preserved. They are
primarily sedimentary, (sandstone, slates, arkose conglomerates) with
interbedded spilitic basalts. The Dalslandian era terminated with the
intrusion of granitic plutons of the Bohus group in southwestern Sweden.

The tectonic events occuring during the evolution of the Baltic shield have been described above. It should be noted however, that the orogenic activity in a particular area would also affect surrounding regions to a lesser extent.

Within the Precambrian era, a zone of intense mylonization extending from southern Sweden north to Lake Vättern and from Lake Vänern to the Norwegian border was formed. During the Jotnian – Eocambrian period, the Vättern graben was developed. The tectonics of the lower Palaeozoic in Scandinavia are dominated by the Caledonian orogeny of Silurian – Devonian time. Extensive low angle overthrusting and nappe movement are typical of the Swedish Caledonides. Within the shield area, normal faulting in the Silja, Närke, Vättern and Skåne regions is ascribed to the Palaeozoic (Stephansson & Carlsson[3]) date. During the late Palaeozoic and early Mesozoic, formation of the Oslo Graben and continued normal faulting in the Skåne area took place. Volcanic rocks of Permian age occur in the Oslo region. The development of these tensional features, together with the Rhine Graben may be associated with the onset of opening of the Atlantic.

In geologically recent times, Scandinavia has been glaciated. Isostatic rebound from the melting of the ice is still taking place at a rate of 8 mm/year in the area between Piteå and Luleå, and at 4 mm/year in the Stockholm area (Lundegårdh et.al.[4]). In the south of Sweden, the land is sinking at about 1 mm/year. There is evidence of recent reactivation of old faults in the northern Sweden Lundqvist & Lagerbäck (5), possibly associated with isostatic rebound.

3.3   Tectonics

Within the Baltic shield, most of the faults and shear zones appear to be primarily of Precambrian age. However, some of them may have been reactivated. At present, knowledge of these features is insufficient to adequately determine their genesis and subsequent history.

Thus, the precambrian shield is traversed by failure planes, crushed
and weathered zones, separating blocks of competent rock. The lateral
distance between these zones can vary from hundreds of meters to several
kilometers, or to even tenths of kilometers. Along many of these failure
zones, movements have occured. Nature itself has often healed crushed
zone products, developed during the shear motions; thus, grained as well
as coarse crushed materials were regenerated into hard rocks (e.g.,
plonite, breccia). However, failure zones are often zones of weakness
composed of low quality, fractured and altered rock material and with
a high permeability.

Deep groundwater movements that are caused by local varia-
tions in relief will probably be mostly dependent on the rock
jointing down to one or two hundred meters' depth (6).

The experience from excavations and tunneling in Swedish granite forma-
tions indicate that in many cases horizontal jointing is more frequent
than vertical, see figure 1.

FIGURE 1.   SHEET JOINTING IN GRANITE

This flat joint system is generally parallel to the ground surface. The name "topographic jointing or sheet jointing" which has been used for the phenomenon describes the main features quite well.

These horizontal joints are frequently open but can also be filled with sand and silt, Morfeldt (65). This circumstance can be explained from theory of pressure release. Stephansson (66) has also explained this effect with pressure release of the ice and material transport by the water during the glaciation period.

Mapping during excavations and tunneling shows that the distances between the horizontal open joints increase with depth, Morfeldt (68),(69).

Observations from well drilling in granite also show that the horizontal open joints are water bearing and springs in rock slopes occur where these joints exit, Morfeldt (70).

Investigations of water leakage into a tunnel in Forsmark, Carlsson & Olsson (67), showed that the largest leakage occured through flat joints with the explanation that these joints in the gneiss-granite more cut through the rock mass than other joints. Similar results have been shown by Hausen (70), see figure 2, where he shows the percolation and water movement in granite with nearly horizontal joints.
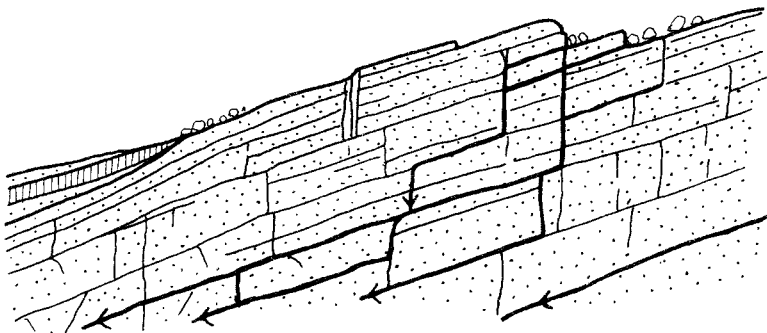

FIGURE 2. PERCOLATION AND GROUND WATER
         MOVEMENT IN GRANITE WITH A
         NEARLY HORIZONTAL JOINT
         SYSTEM, HAUSEN (71)

This means that granite with a dominating sheet jointing system can give a higher value of the permeability in the horizontal directions than in vertical.

The stress situations will also influence the capacity of water bearing of the joints. The two horizontal stresses are very often higher than the vertical one, see figure 7, which gives that the vertical joints generally are more closed than horizontal joints. However, since the ratio between major and minor horizontal stress can be very high, the water bearing capacity can be different in different directions.

Since the fracture system and stress situation will vary from place to place it is impossible to make any general assumption. Instead local tectonic and stress analyses must be done for every site.

3.4   Site Areas

Investigations are in progress at two sites, Forsmark and Oskarshamn, to determine geological and geohydrological conditions typical of the rock in which a repository may be located.

The area around Forsmark exhibits subdued local relief of about 10 m. The major low lying areas are occupied by lakes. Drainage is generally poorly developed, with the principle drainage courses developed along glacially deepened structural features in the bedrock.

From the coast inland the average elevation rises about 1 m in 2 km. At a distance of about 100 km between Sandviken and Avesta the relief becomes more pronounced, typically 100 m and locally up to 300 m. This topography continues westwards to the Norrland mountains area.

The area around Oskarshamn exhibits a highly dissected relief of about 20 m. Inland from the coast, the average ground elevation rises at about 1 m per km to the Götaland Highlands. Drainage from this uppland area is principally to the south-south east along valleys sub-parallel to the Vättern graben. Relative relief in the Götaland Highlands is generally 20 to 50 m, and locally over 100 m.

Precipiation for both the Oskarshamn and Forsmark regions increase from
about 500 m per year at the coast to about 700 mm per year inland.

The geology of the candidate areas is being studied by SGU for KBS. Only
a brief ontline is given below: further details are given in Technical
Report No. 3 (6)

It should be noted that a criterion for preliminary site selection required
an area of uniform granitic bedrock adjacent to the Baltic coast.

The bedrock of the Forsmark area comprises Precambrian Svecofennian
gneisses and gneissic granites with some leptites and minor areas of
youger granitic and basic intrusions. Offshore, to the north east, the
Precambiran basement is overlain by Jotnian sandstones, Cambro-Ordovician
shales and sandstones and Ordovician Limestone (7).

The surficial deposits of the area comprise tills overlain by postglacial
clays in the valley bottoms.

The structural geology of the area is extremely complex, reflecting a
tectonic history extending back to the Precambiran. First and second
order lineaments in the region have been identified from air photographs
and satellite imagery by Stephansson & Carlsson (3) and more recently
by SGU. The site under investigation is bounded on three sides by linear
structural features.

The bedrock of the Oskarshman region comprises Precambrian Gothian rocks
known collectively as the Småland-Värmland intrusions, primarily granitic
and grandodioritic in composition. Younger granites of sub-jotnian age
have intruded the Småland-Värmland suite at some locations. The candidate
repository site in the Oskarshman area is located within such an intrusion,
the Götemar massif. The granite is primarily massive, coarse grained,
with some medium, fine grained, and porphyritic varieties.

Offshore, the Precambrian rocks are understood to be overlain by Cambrian
and Silurian sandstones and limestones. Rocks of this age are exposed in
Öland, som 25 km offshore from the Götemar area.

The principle lineaments in the Oskarshman area have been identified from air photos and satellite imagery (8). The major joint directions in the granite massif have been interpreted by Kresten and Chyssler (9) as representing classic joint patterns of granitic plutons. Four joint sets have been identified: radial, tangential (concentric) diagonal and flat lying. A north-south reverse fault with a proven lateral extension of more than 25 km is located about 500-600 m west of the area being studied.

4. THERMOMECHANICAL CHARACTERISTICS OF HOST ROCK

The recovery of core from vertical drillholes can provide intact rock
samples for laboratory determinations of elastic, strength and fracture,
creep and thermal properties. The elastic properties, i.e., Young's
modulus of elasticity and Poisson's ratio, the peak strength and
residual strength can be obtained without difficulty under uniaxial
loading conditions. The evaluation of these properties under confine-
ment stress is slightly more difficult and becomes increasingly so
when elevated temperature and fluid flow within the sample are intro-
duced. The lack of available equipment to accomplish these latter tests
on a production basis is of particular concern. Generally speaking,
determination of the thermal properties of rock, namely thermal con-
ductivity and specific heat, can be accomplished by established labora-
tory techniques.

From tests on small samples in the laboratory, the strength and stiffness
of joints can be evaluated. This of course assumes that "intact" and
representative samples with joint planes can be recovered from the field
and satisfactorily prepared in the laboratory.

In order to evaluate the properties of the in situ rock mass, the behavior
of an intact rock sample and joint of field scale must be considered,
together with the effect of size. This requires in situ testing, or testing
of very large samples in the laboratory.

4.1 Strength of intact rock

Determination of the strength of a rock is dependent upon the physical
dimensions of the specimen, partly due to the microfractures and partly
due to the techniques of data accumulation. In general, test results for
uniaxial rapid compression of rock specimens at room temperature indicate
a reduction of up to 75% in compressive strength with increasing specimen
dimensions (10).

Large-scale tests on granite rock, reported by Singh and Huck (11),
show that a 0,80 m diameter sample of granite failed at stress
levels only 50 to 75% of the strength achieved from small cores.
These tests also indicated a negliable effect of size on the de-
formation modulus and Poisson's ratio. Londe (12) also observed similar
reduction of strength due to the scale effect. The theory of scale effect
from Weibull (13) and from the empirical approch of Protodyakonov (14)
are attempts to describe the scale effect.

It seems reasonable to assume that the strength of a large specimen is
about 50 to 75% of the strength of a small sample which agrees quite well
with Protodyakonov approch. In addition, test results have demonstrated
a reduction of 20 to 40% in strength when the rock is saturated with
water as compared to dry samples. This effect has been observed by
several investigators, including Broch (15), Farmer (16) and Wawersik (17).

The degree of weathering will also influence the strength. Serafim (18)
showed test results where the compressive strength decreased to about
17 to 25% of the strength of intact granite. Therefore, it is very im-
portant to determine weathering susceptibility.

The most difficult tasks in the determination of rock properties are
those related to assessments of creep and creep rupture characteristics
and of bulk (or macroscopic) in situ properties of all types, particu-
larly under simulated conditions of confinement stress, elevated
temperature, and fluid flow. For relatively short-term loadings under
ambient temperature conditions, the creep deformation of a specimen of
granite will be negligible. With elevated temperatures and sustained
states of stress near the yield strength of the rock, the creep deforma-
tions over long periods of time may be significant and may lead to creep
rupture.

Experience with other materials, such as clay, indicates that the quasi-
static yield stress state for rocks appears to be a lower limit of
"creep inducing" stress. Under this stress level, the creep rate will
only decrease with time and the tested sample may not rupture. Data
from such long-term tests of granite are very limited. Tests of the

influence of different loading rates as performed by Lundborg and
Almgren (19) have shown that the unconfined compression strength de-
crease with increasing test time, or decreasing loading rate. Similar
results have also been reported by Wawersik (17) from an investigation
of time-dependent rock behavior in uniaxial compression. According to
Figure 3, the results of both investigations indicate that the upper
limit of stress is about 70% of the strength from a common rapid com-
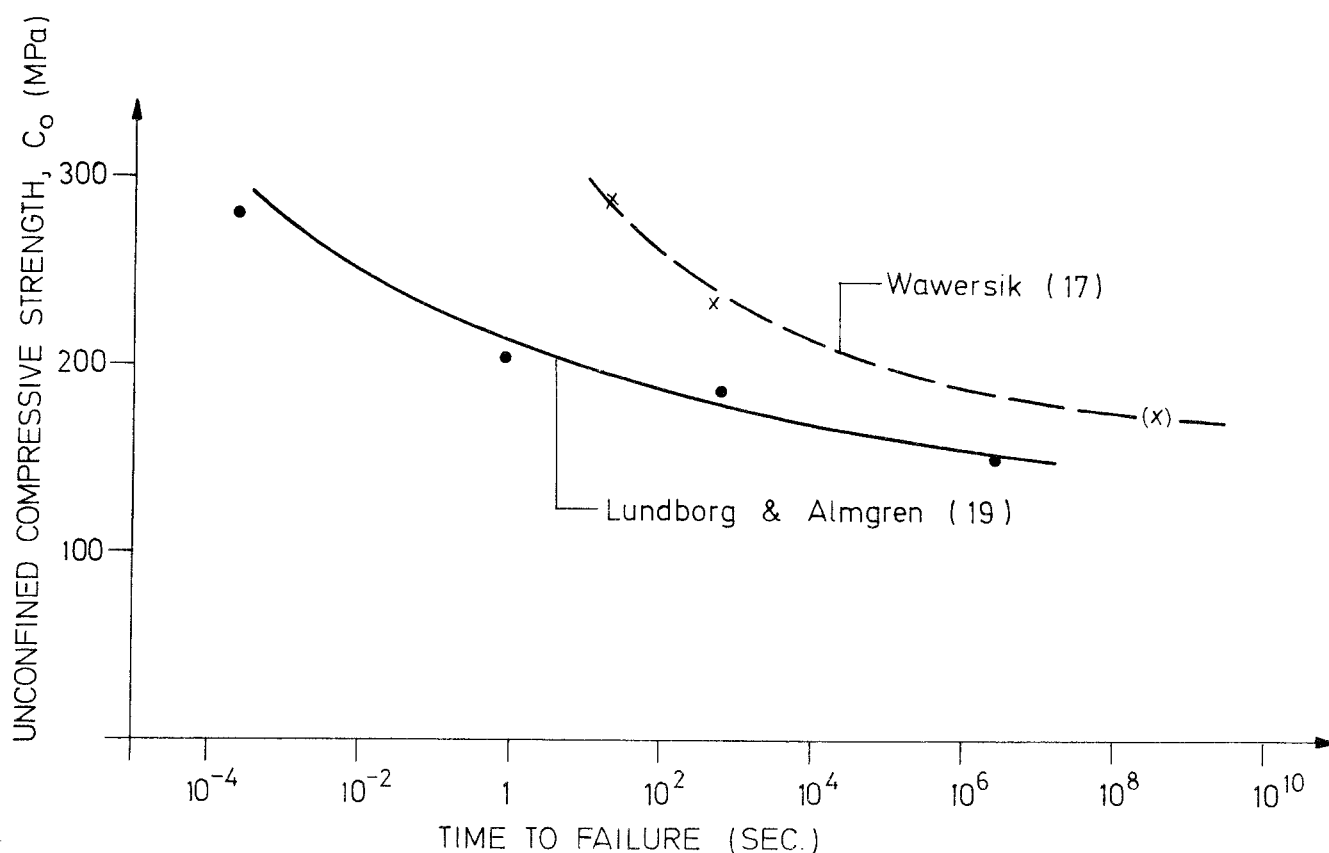pression test.

FIGURE 3. COMPRESSIVE STRENGTH VERSUS TIME TO FAILURE.

Quick, dry tests on small samples have shown that the uniaxial compressive
strength of granite will vary from 100 to 300 MPa with a mean value
of about 200 MPa (20). For saturated intact rock of large dimensions
and loaded for a relatively long time, the compressive strength can
only be estimated to be about 20 to 100 MPa, neglecting the effects of
weathering.

Tests of Stripa granite carried out by Swan and Stephansson (21) have
shown that the compressive strength is about 200 MPa and is not tempera-
ture dependent in a temperature interval of 20 to 150°C. Tensile fracture
tests on Stripa (17) show a mean value of 15 MPa.

The problems of tensile strength for intact rock material are similar to those related to compressive strength, and no data have been found which would invalidate the empirical assumption that the tensile strength is of the order of 5 to 10 percent of the compressive strength, or about 1 to 10 MPa. However, it is quite clear that these values are very uncertain due to the fact that problems associated with determining tensile strength have not been fully investigated.

The results of triaxial compression experiments are commonly represented by a sequence of stress circles on a Mohr diagram to which a failure envelope may fit. The failure envelope may be represented mathematically in terms of principal stresses and a failure parameter in the sense of a yield condition, similar to the procedures used in plasticity theory. A parabolic failure envelope can be stated in the following equations:

$$\tau^2 = 4B \ (\sigma + T_o) \tag{1}$$

where τ is the shear stress, σ the normal stress, T_o the tensile strength, and

$$4B = + (2T_o + C_o) - 2 \sqrt{T_o(T_o + C_o)} \tag{2}$$

with C_o being the uniaxial compressive strength.

With the strengths cited earlier (T_o = 1 to 10 MPa and C_o = 20 to 100 MPa), it follows that:

$$4B = 13 \text{ to } 54 \text{ MPa}$$

The failure condition of equation (1) will coincide with the failure criterion proposed by Griffith (22) when the ratio between the uniaxial compressive and tensile strengths is 8.

In Figure 4, the failure condition $\tau^2 = 42 \ (\sigma + 6)$ is graphically represented. It corresponds to "average" compressive and tensile strengths of C_o = 60 MPa and T_o = 6 MPa, respectively. In the stress interval of 0 to 50 MPa the linear Mohr-Coulomb failure criterion can be used as a good approximation to the parabolic criterion; viz.:

$$\tau = 19 + \sigma \tan (34^o) \tag{3}$$

The residual strength for the rock material has merely been choosen to be:

$$\tau_r = 1.9 + \sigma \tan (34°) \qquad (4)$$

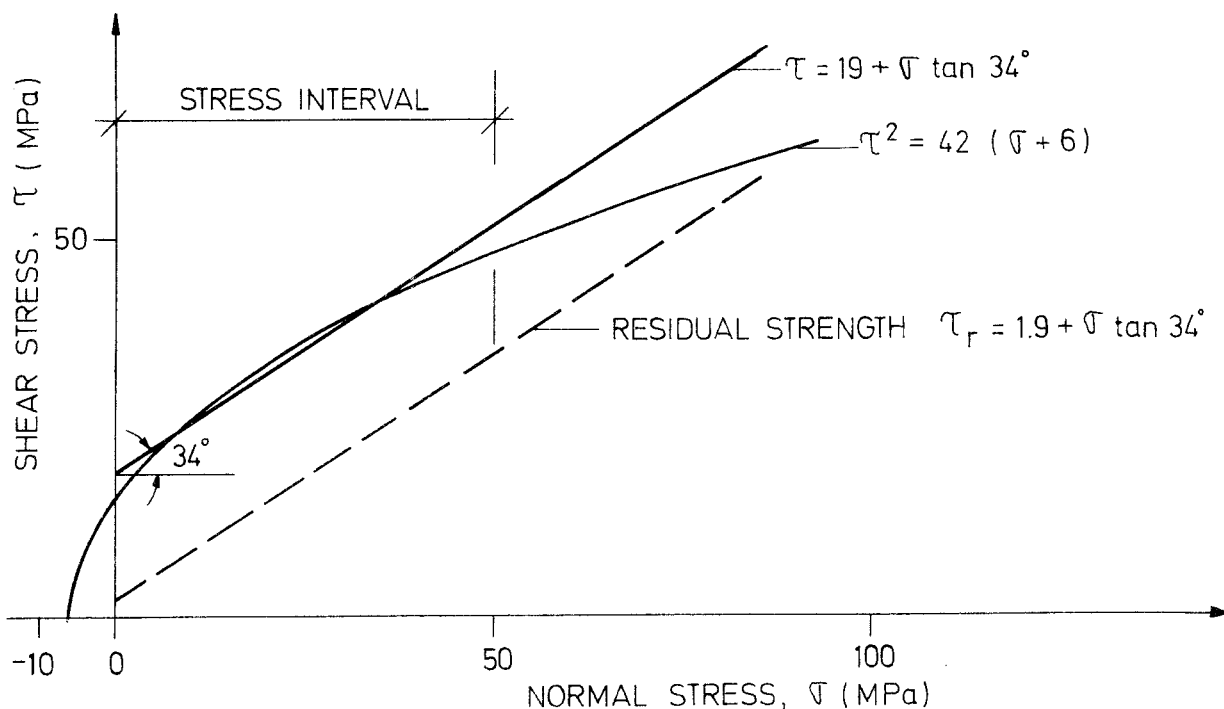This value is uncertain and very few results have been found in the literature.

FIGURE 4. FAILURE CRITERION FOR INTACT ROCK MATERIAL.

4.2 Deformational characteristics of rock specimens

The modulus of elasticity of a relatively small laboratory rock specimen may be greater than that of large block of rock subjected to full scale loading. Once again, this discrepancy is partly due to microfractures and deformation of joints in the large block, and partly due to the methods of testing and data reduction. The modulus of elasticity of granite will vary from 20 to 60 GPa (16).

Tests on Stripa granite by Swan and Stephansson (21) have demonstrated
that both elastic and strength anisotropy are negligible. Data for Poissons
ratio indicates that the value can be taken as 0.20 to 0.30. A few tests
of granite with different temperatures (23) shows that a reduction of
about 20% exists for the elasticity modulus and Poisson's ratio when the
temperature increase from 20 to 120°C. Tests of Stripa granite carried out
by Swan and Stephansson (21) have shown that Young's Modulus decreases
gradually with increasing temperature from 35 GPa at 20°C to 31 GPa at
100°C. However, Poisson's ratio seems to be constant, approximately 0.2,
in the temperature interval of 20 to 100°C.

This reduction is attributed to the differential thermal expansion and
moduli of elasticity of separate minerals within the rock which gives
rise to cracking. Results from the tests by Wawersik (17) indicate that
under the upper limit of stress there is no change in modulus for
different loading rates.

In Figure 5, a stress-strain curve for a granite is shown, as well as
the relationship used in the mathematical model for this study. The
relationship will give values on the safe side in the short term. In
the long term, the results will converge with the theoretical behavior
due to the fact that no values above the critical upper limit of stress
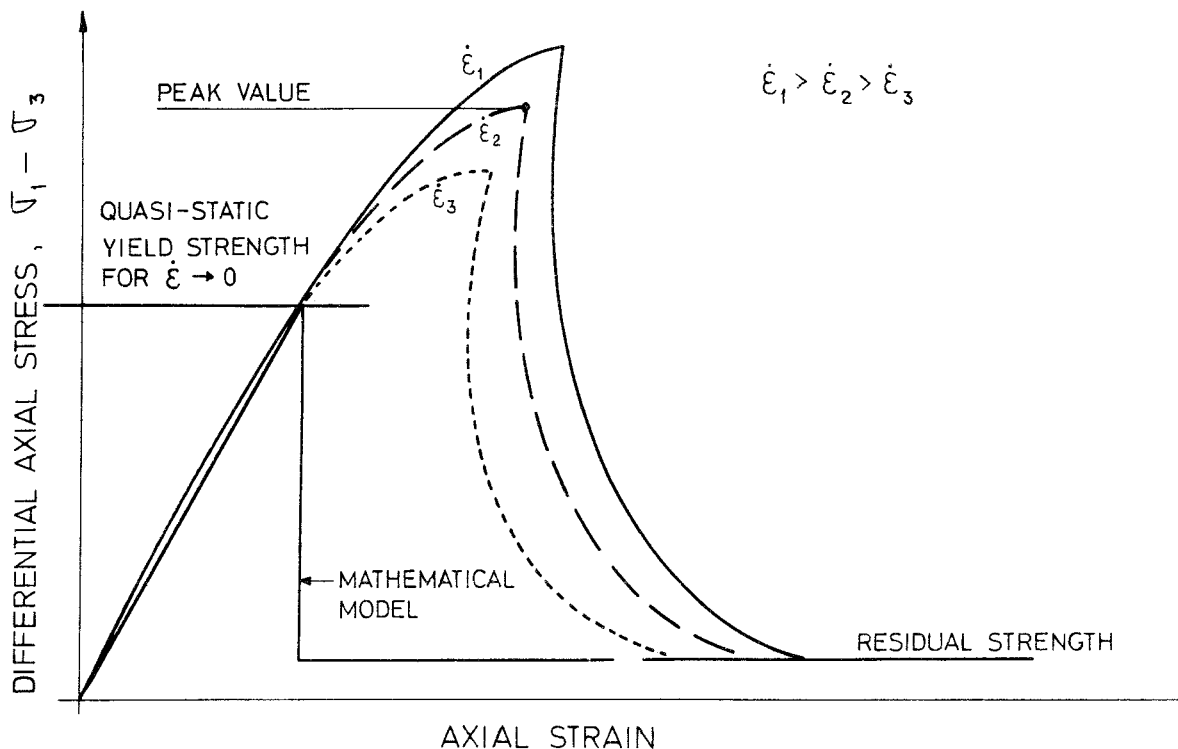are allowed.

FIGURE 5. PRINCIPAL STRESS-STRAIN CURVE FOR GRANITE

The deformation modulus of the rock mass can be much less than the
modulus of a small intact rock sample due to deformations of the joints,
and must be considered when the loading area is greater than the joint
spacing. An equivalent modulus of elasticity E_m of the rock mass can then
be calculated from the following equation:

$$E_m = \frac{1}{\frac{1}{E} + \frac{1}{p \cdot s}} \qquad (5)$$

where $\underline{E}$ is the modulus of the intact rock, $\underline{p}$ the stiffness of the joint,
and $\underline{s}$ the joint spacing. For example, with a stiffness value of 200 MPa/cm,
$\underline{E}$ of 50 000 MPa, and $\underline{s}$ of 180 cm, the modulus of the rock mass will be:

$$E_m = 21 \text{ GPa}$$

The joints have a dilatancy behavior due to the roughness of the joints.
This effect is observed as opening of a joint when shearing occurs,
mainly after the peak strength value has been reached. This means that
up to the shear strength value, the volume of the rock mass will decrease,
and chiefly depend on the lateral deformation of the intact rock material.
The Poisson's ratio ν_m of the rock mass can then be calculated such that
the lateral deformation will be the same for the rock mass, with its lower
elasticity modulus, and the intact rock, according to the following
equation:

$$\nu_m = \frac{\nu}{1 + \frac{E}{p \cdot s}} \qquad (6)$$

Where ν is the Poisson's ratio of the intact rock. For the sat of values
used above, it follows that $\nu_m = 0.10$.

After the peak value has been reached the joint width will increase
and neutralize the decrease of volume of the intact rock. For the
residual behavior of the rock mass, an increase of the volume can
be estimated due to the dilatancy of the joints. It is, however,
quite clear that volume changes and Poisson's ratio of a rock mass
are not fully understood.

4.3 Density

The range of density values in granite is not great. A representative value may be taken as 2700 kg/m^3 see Figure 10.

4.4 Failure criterion of the joint

The shear strength developed along discontinuities is of great importance to the local and time-dependent stability of an underground excavation. The surfaces of the discontinuities may be smooth, or rough, undulating or planar, and contain clay minerals. These details all affect the shear strengths available to maintain the competency of separate blocks or zones adjacent to excavations. However, for preliminary design studies, precedent data are of great use in conjunction with the assumptions made for the joint set frequency and orientation. A useful summary of joint shear strengths is provided by Barton (24). From a statistical consideration of these data, the following relationship was proposed:

$$\tau/\sigma_n = \tan \left| A \log_{10} (C_o/q_n) + 30^\circ \right|$$

where

τ = joint shear strength (MPa)

σ = joint normal stress (MPa)

A = constant, with:

A=20 : Rough, undulating joint surface

A=10 : Smooth, undulating joint surface

A=5 : Smooth, nearly planar joint surface

C_o = unconfined compressive strength of adjacent rock (MPa)

For the systematic jointing of the granite, A=10 is recommended since the joint can be smooth but undulating. Results from investigations of tension joint in granite carried out by Bjurström (25) indicate good agreement with Barton's formula if a value of A=10 is used.

If the rock joints are filled with secondary clay minerals or chlorite the available friction angles may be much lower. Within granite these lower values might be of the order of 20 degrees. It is also clear that the factors which influence the unconfined compressive strength also influence the shear strength of the joint.

In stress analysis with an elasto-plastic material it is better to use Mohr-Coulomb failure criterion (26); viz.:

$$\tau = C + \sigma_n \tan \phi \qquad (8)$$

This linear relationship will be a good approximation of the Barton equation (C_0 = 60 MPa, A = 10) for a stress interval $5 < \sigma_n < 25$ MPa when the cohesion c 1.0 MPa and the friction angle $\phi = 34°$, as illustrated in Figure 6.
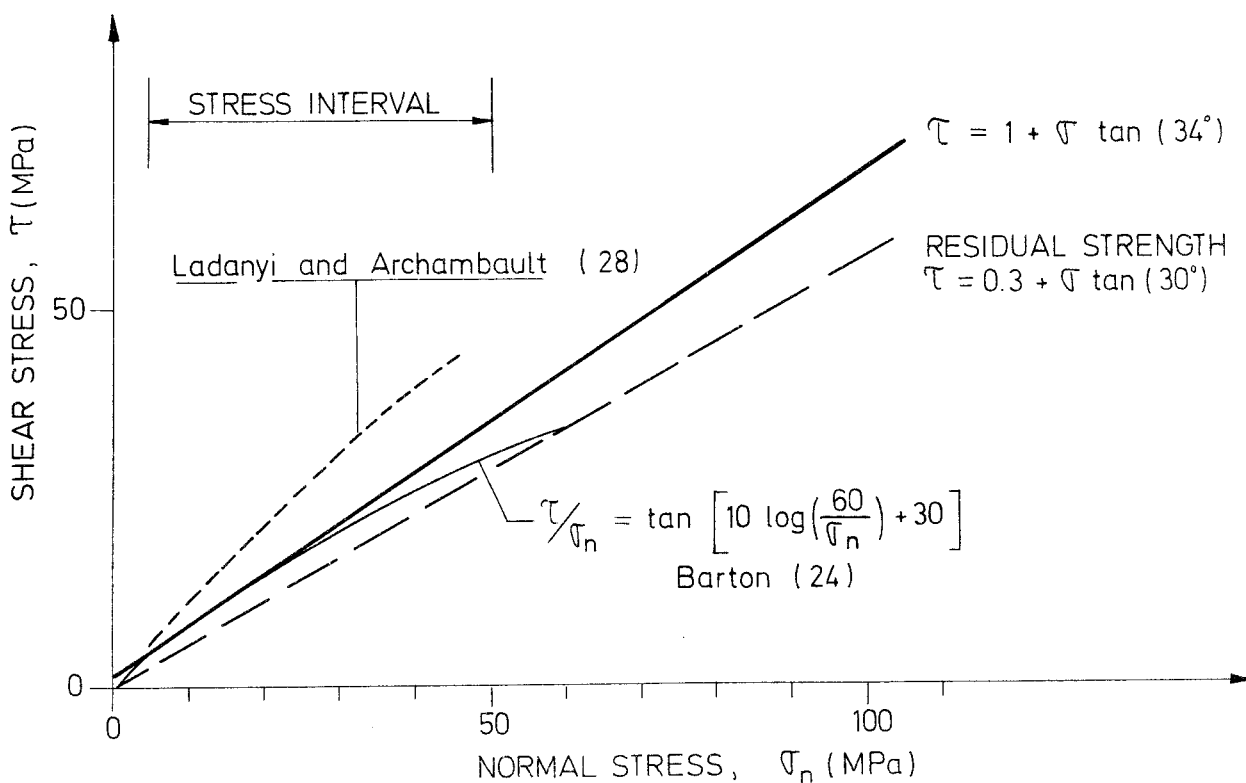

FIGURE 6. FAILURE CRITERION FOR A JOINT.

The equation of the peak shear strength according to Ladanyi and Archambault (29), with values of constants according to Goodman (28), will in the actual stress interval give a value higher than the calculation based on Barton's relationship. The proposed relationship with a Mohr-Coulomb failure criterion seems to be on the safe side.

The very low observed cohesion values reported by Hoek and Bray (27) can depend on the fact that much lower stresses were employed. Their results could be used as a demonstration of the residual strength since the rock mass failure of a slope is a result of a progressive failure. The residual strength can then be estimated to be:

$$\tau = 0.3 + \sigma_n \tan (30^{\circ}) \quad (MPa) \tag{9}$$

The residual strength of joints from the Stripa granite (17) may be described by a bilinear relationship and will give $C = 0$ and $\rho = 33^{\circ}$ for σ_n less than 4 MPa and $C = 0.7$ MPa and $\rho = 25^{\circ}$ for σ_n greater than 4 MPa.

Since investigations of the shear strength and stiffness of joints loaded over relatively long time periods have not been performed, it is very difficult to evaluate the creep behavior of the joint. As the creep behavior depends to some extent on the compressive strength of intact rock material, the effect of creep can be considered by use of the quasi-static yield stress for the unconfined compressive strength in Barton's formula.

4.5 Normal and shear stiffness of the joint

When analyzing groundwater flow as influenced by problems of a rock mechanics nature, the normal and shear stiffnesses of the joints must be determined. These properties vary according to the joint width and to the normal stress across the joints. A systematic investigation of joint stiffness existing for stresses at a depth of to 1 000 m has not been done. For considerably lower normal stresses and for unfilled and very thin joints, values of normal stiffness are of the order of 200 MPa/cm and for shear stiffness approximately 5-10 MPa/cm. Boutard and Groth (30), Witherspoon,et.al. (31) have reported a normal stiffness value of about 40 MPa/cm.

Through the use of a linear relationship between stress and stiffness, a joint can be completely closed even for a relatively low stress. Laboratory tests have shown that this behavior is not totally valid. Therefore in order to calculate the change in joint width and permeability as a function of the changes in stress, a non-linear relationship has to be used. The changes in joint width will then be smaller for higher stress levels.

Goodman (29) has proposed such a relationship:

$$\frac{\sigma - \sigma_0}{\sigma} = k \left(\frac{\Delta b}{V_{mc} - \Delta b} \right)^t \tag{10}$$

where
$\sigma - \sigma_0$ = change in stress

σ_0 = initial stress

Δb = change in joint width

V_{mc} = maximum possible closure

k, t = constants

Shehata (32) has proposed a semi-logarithmic relationship between changes in joint width and stress. Both these equations have the disadvantage that they are not defined when the initial stress is zero.

Joint compression is essentially unrecoverable. This fact has been observed by several investigators, including Goodman, (29), Londe (33), and Jovanna (34), from studies of water flow through fissured rock for different stress conditions, both loading and unloading. Gale (35) has found that there is a hysteresis effect, which however is recoverable to some extent.

A hyperbolic shape function between stress and deformation has been reported by Bjurström (25) with a closure of about 0.2 mm. Results from permeability and loading tests of a large granite sample for both tension joint and saw-cut joints have given values for the closure of about 0,15 - 0.3 mm (35,31). These closures seem to occur when the change of effective stress is about 10 MPa. In all of these tests, the initial stress was equal to zero.

For a rock mass at 500 m depth, the initial stress field is of an order of 10 MPa. For this stress level($\pm$5 MPa), a linear relationship between stress and deformation can be obtained. This is done with a very high value of the normal stiffness, around 1000 - 6000 MPa/cm, calculated from measurement reported by Witherspoon, et.al. (31).

The shear stiffness of a joint will also depend on the shear and normal stress levels, and the testing will show a strain-softening behaviour after the peak value is reached (25,29). Due to the dilatancy, the behaviour of a joint during both normal and shear stresses changes is very complicated, and not yet fully understood.

4.6   Initial stress field

Field test procedures for the determination of the in situ stress field vary from hydro-fracturing of well bores to measurements by strain relief techniques in drillholes in mine entries. Measurements by the latter technique, including the bore-hole deformation gauge and the "doorstopper", in mines that are reasonably adjacent to an identified rock mass site could provide valuable and reasonably accurate information. It is however quite evident that none of the methods used to measure in situ stresses are free from objections. Partly, there are practical difficulties in making the measurements, and partly, the results are to a large extent affected by residual stresses in the rock mass (36).

Based on measurements with a stiff loadcell carried out by Hast (37), the ratio between the horizontal and vertical stress K_o can be calculated, as illustrated in Figure 7. Based on the relationship between the sum of the principal horizontal stresses $\sigma_{h1} + \sigma_{h2}$ and the depth Z.

$$\sigma_{h1} + \sigma_{h2} = 20 + 0,095 \ Z \qquad \text{(MPa)} \qquad (11)$$

and the ratio

$$0.3 < \sigma_{h1}/\sigma_{h2} < 0.75 \qquad (12)$$

as proposed by Hast (37) the interval of K_o can be calculated as:

$$0.8 + \frac{170}{Z} < K_o < 2.7 + \frac{570}{Z} \qquad (13a)$$

Hoek (38) has suggested a similar relationship:

$$0,4 + \frac{100}{Z} < K_o < 0.8 + \frac{1000}{Z} \qquad (13b)$$

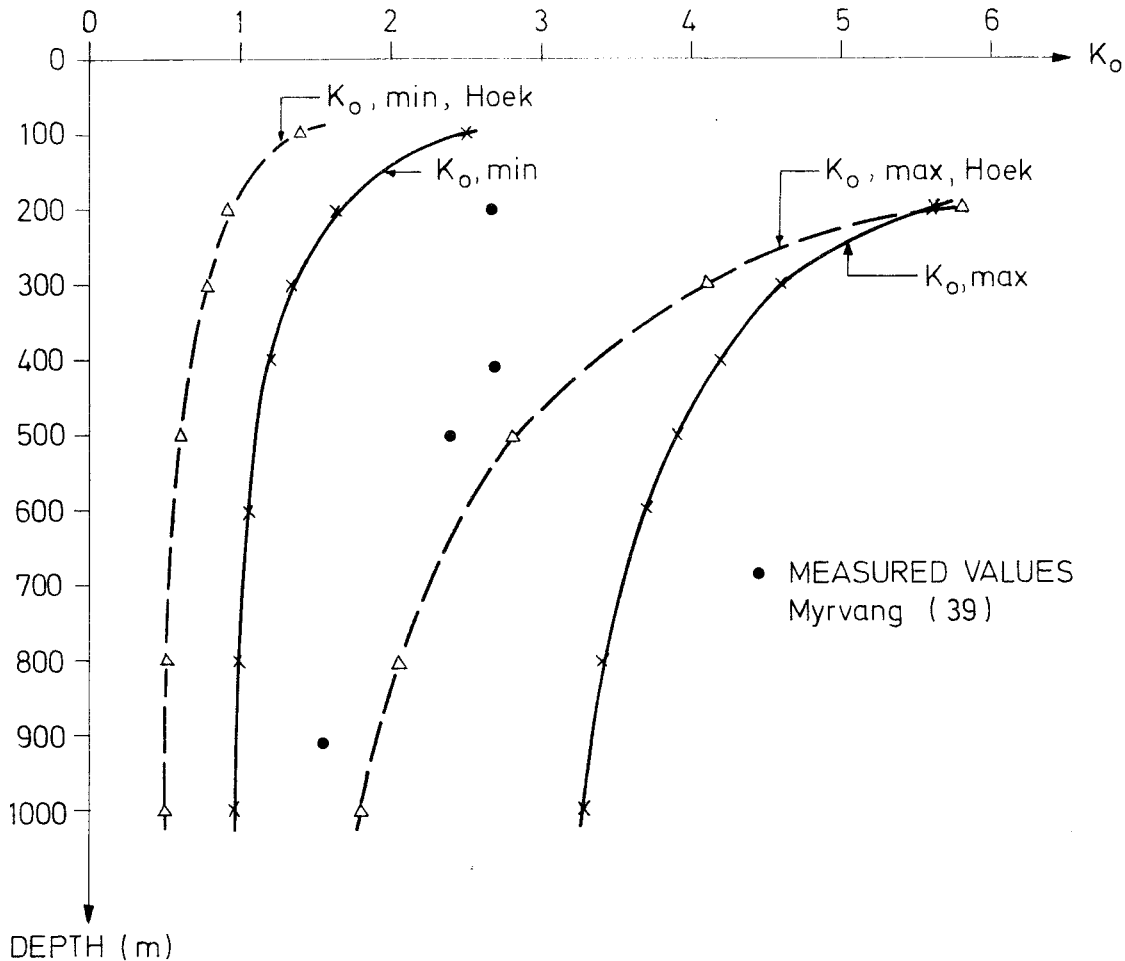The limit values for both relationships are shown in Figure 7.

FIGURE 7. RATIO BETWEEN HORIZONTAL AND VERTICAL STRESS AS A
FUNCTION OF DEPTH.

A good estimate of the K_o-value for Swedish precambrian bedrock seems to be the maximum value proposed by Hoek (34), as this line seems to be the average value of the measurements by Hast and is close to the measured values for Swedish precambrian rocks (39).

4.7   Thermal conductivity

The data presented in the litterature suggest that representative ranges of thermal conductivity may be taken as 4 to 9 millical/cm - sec - $^{\circ}$C (1.5 to 4 W/m - $^{\circ}$C).
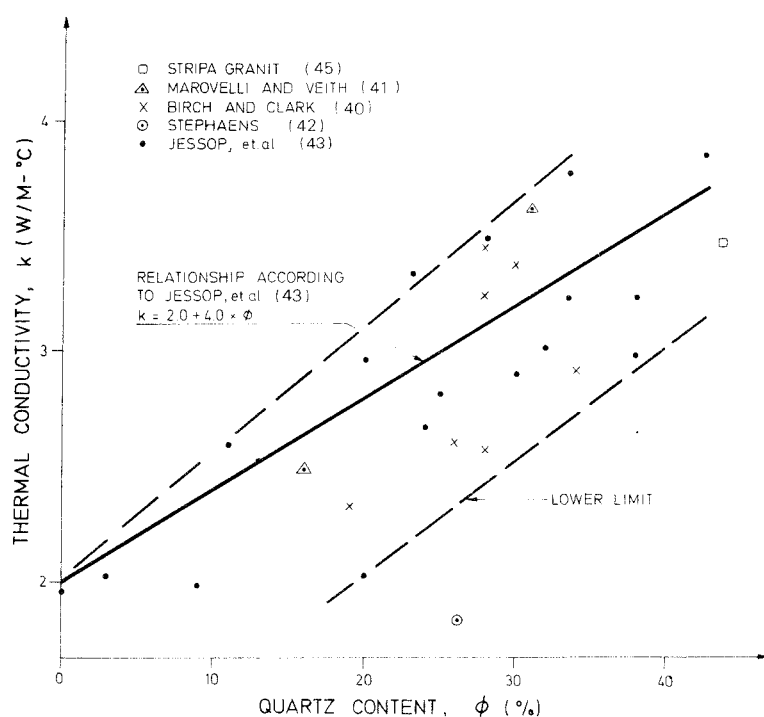

FIGURE 8. THERMAL CONDUCTIVITY AS A FUNCTION
OF QUARTZ CONTENT

A significant variable, which gives rise to the variance in the thermal conductivity of granitic rock, is the rock composition and crystal structure. A correlation between quartz content and thermal conductivity has been identified by Birch and Clark (40). The thermal conductivity of granite with different quartz contents at 50°C is reported by Birch and Clark (40), Marovelli and Veith (41) and Stephaens (42). As can be seen in Figure 8, the thermal conductivity increases with increasing quartz content. Results from over two hundred samples reported by Jessop, et.al. (43) have also been plotted on the same figure. Jessop, et.al. also give an empirical relationship between conductivity and quartz content:

$$K = 2.0 + 4.0 \, \emptyset \tag{14}$$

where K is the conductivity in W/m $^{\circ}$C and $\emptyset$ the quartz content. This curve seems to fit well with the measured values. These investigations also found that, similar to other rocks, the thermal conductivity decreased with increasing temperature. The results can be seen on Figure 9. Demitrier, et.al. (44) developed such an expression for predicting variations of thermal conductivity with temperature:

$$K = K_{50} - 0,2 \left[\frac{(T-30^{\circ})2.15}{T} - e^{-\frac{T}{300}} \right] \tag{15}$$

where K_{50} = thermal conductivity (Kcal/m, h, $^{\circ}$C) at 50°C

T = temperature in $^{\circ}$C

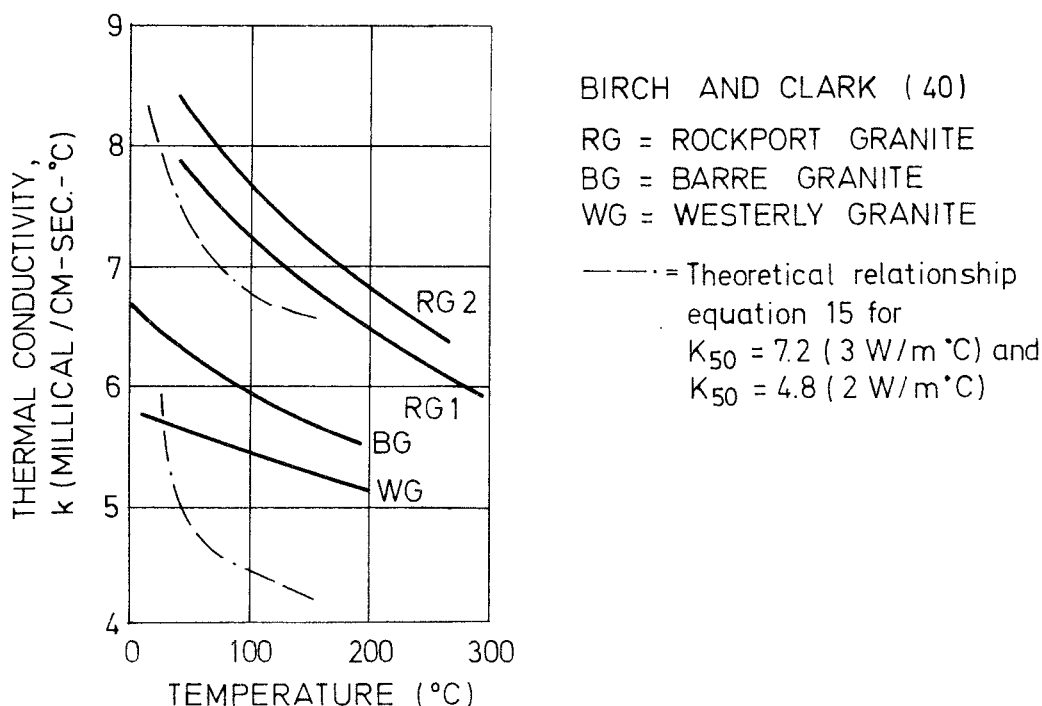The equation seems to give to high values of thermal conductivity for temperatures less than 40°C.

FIGURE 9. THERMAL CONDUCTIVITY VERSUS TEMPERATURE.

In addition to the measurements made on relatively homogeous granites, Assad (46) developed an equation for estimating the thermal conductivity of fluid bearing, porous rock as a function of the thermal conductivity of the fluid saturating the pores and the porosity; viz:

$$K/K_1 = (K_2/K_1)^m \qquad\qquad (16)$$

where K = thermal conductivity of fluid saturated rock

    K_1= thermal conductivity of rock solids

    K_2= thermal conductivity of the fluid

    m = an empirical exponent = C∅

        where C is a correlation factor (Ω 1.0)

           ∅ is fractional porosity

For $K_2/K_1 \cong 0,2$ and m = 10^{-4}, the thermal conductivity of the rock mass is nearly equal to the value of intact rock.

In Figure 10 results of measurements of thermal conductivity on granite samples from Swedish drillings have been summarized (47). The thermal conductivity have been plotted against the rock density.

FIGURE 10. THERMAL CONDUCTIVITY VERSUS DENSITY FOR SWEDISH GRANITE.

Some results reported by Beck, et.al.(48) Stille et.al. (71) indicate
that the conductivity in situ is less than that of a relatively small sample
since the effect of joints in the rock are neglected for the small sample.

Wether the conductivity measured in the laboratory on small rock samples is
representative for big rock masses is open to discussion (48,49).
Therefore, a conservative value of thermal conductivity should be chosen
for the purposes of numerical calculations.

The values of the specific heat reported by Lindroth and Krawza (50) and
Birch (51) give a range of 700-900 J/kg $^\circ$C. The specific heat increases
only slightly with increasing temperature. For the range of temperatures
from 0 to 100°C, a constant value could be used with little loss of accurancy
in thermal calculations. Specific heat tests on Stripa granite give a
value of 840 J/kg-$^\circ$C (45).

The thermal expansion coefficient of a rock mass is a function of the Youngs
Modulus, and expansion coefficient for each mineral in the rock matrix.

The values of the linear thermal expansion coefficient for granite, as
reported by (52), Richter et.al. (53) and Griffith (54), range from
3 to 12 X 10^{-6}/$^\circ$C.

Cooper and Simmons (55) have proposed a theoretical relationship based
on the volume fraction, modulus and expansion coefficient of each minerals
and compared the theoretical calculations with measured values of the
volumetric thermal expansion. The agreement was reasonably good. By
dividing the volumetric coefficients reported by (55) by a factor of 3,
a value of about 6 to 8 x 10^{-6}/6° is obtained for the thermal expansion
coefficient.

5.   GEOHYDROLOGY

Within this study, some theoretical aspects related to groundwater flow in fractured crystalline rocks have been reviewed. In addition, a brief literature review was undertaken for guidance in the selection of geohydrological parameters. Site specific field data for Sweden are only just becoming available. Thus, information from existing wells, construction activities, and studies in similar geological environments have been used to establish typical input parameters for the numerical models.

For the current study, flow of water in fractures within an essentially impermeable rock mass must be considered. The travel time and the quantity of water flow through the rock mass is the result of the interaction between different parameters:

- changes in effective stress will change the joint width;
- the permeability, or flow conductivity, of the joint is dependent on the effective joint aperature;
- the effective aperature and permeability of each joint and the spacing between adjacent joints give the amount and velocity of water flowing through a rock mass under a given hydrostatic gradient.

The principal coupling between the effective stress and the permeability of a rock mass is illustrated in Figure 11 .
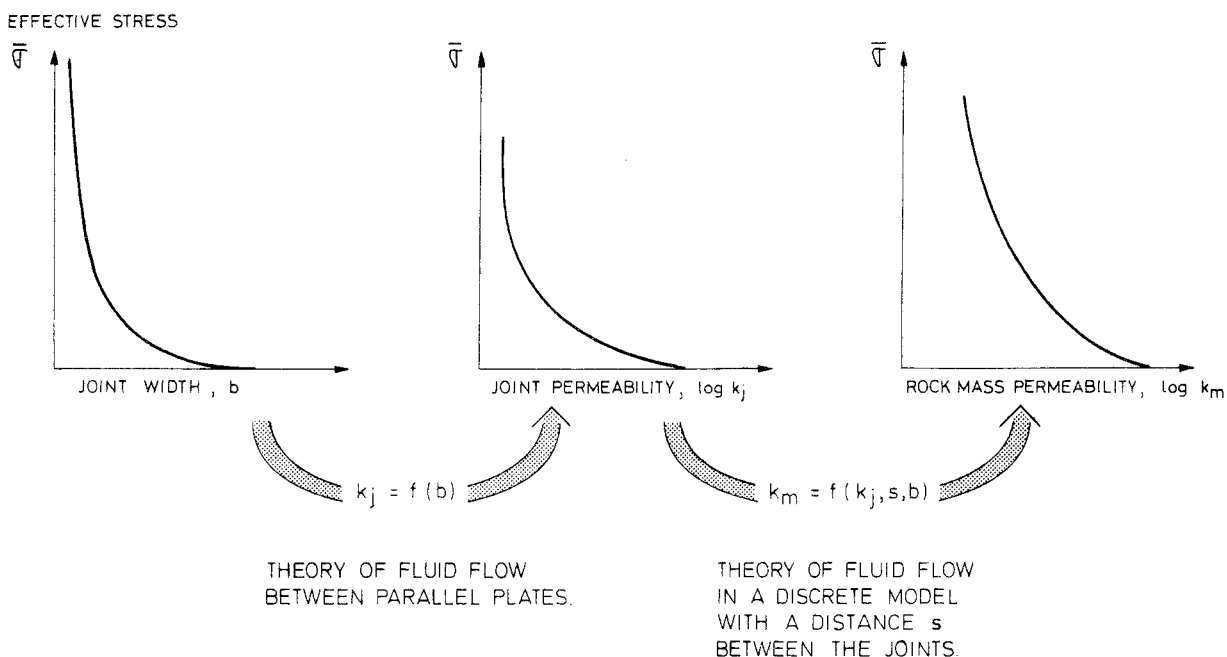
EFFECTIVE STRESS

FIGURE 11. PRINCIPAL RELATIONSHIP BETWEEN STRESS-JOINT
PERMEABILITY AND ROCK MASS PERMEABILITY.

5.1   Changes of joint aperature

The stress/deformation relationship for a joint has been discussed in chapter 4 of this report. It is quite clear that considerable additional investigation is required to clarify this relationship. No results have been found in the literature for the deformation of a joint at a depth of 500 m, where the initial stress field is of the order of 10 MPa. It is this stress level that Witherspoon,et.el.(31) found to be significant for closing a fissure.

Since previously mentioned relations between change of joint aperature and stress level are not well defined for an initial stress level of zero (29,32), the following equation has been assumed:

$$\bar{\sigma} = k_1 \left[\exp(k_2 \Delta b) - 1 \right] \tag{17}$$

where: $\bar{\sigma}$ = normal stress

Δb = change in joint aperature

k_1, k_2 = constants

The above relation employes an initial tangent stiffness of about 40 MPa/cm and a change in joint width of 0.15 mm at a stress level of 10 MPa. For this case, the constants k_1 and k_2 are equal to 0.14 and 29, respectively. As illustrated in Figure 12, the relationship seems to fit quite well with the values observed by Bjurström (25) and Witherspoon, et.al. (31).
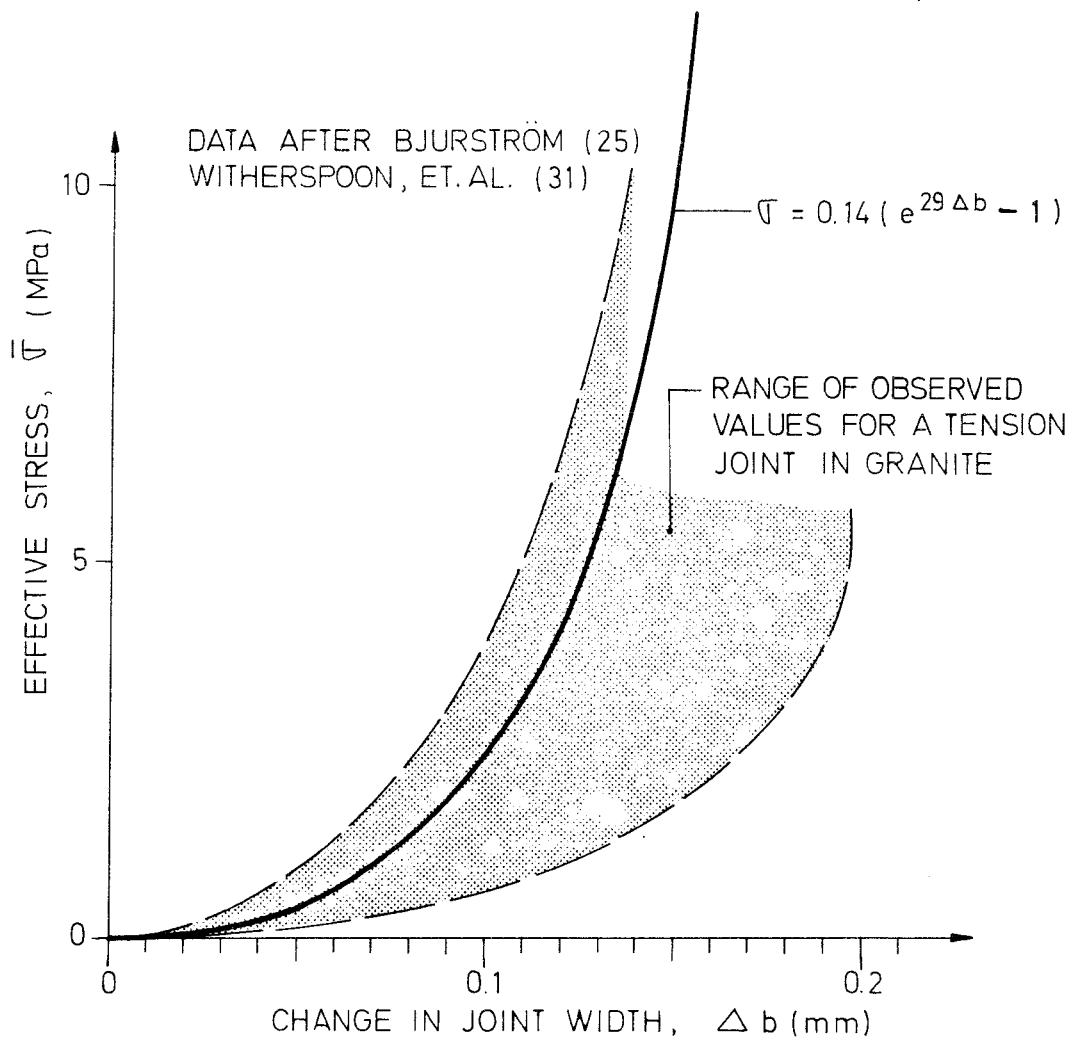
FIGURE 12. STRESS VERSUS CHANGE IN JOINT WIDTH

Very little data are available concerning the effect of shear deformation on the permeability of a joint. Both decreases and increases in permeability have been observed with increasing shear stress and shear deformation (24,56). This difference can depend upon the dilitancy which occurs mainly after the peak strength has been reached. It is obvious that this effect must be studied further before any definitive conclusions can be made. If the stress changes are recoverable, which is a conservative presumption, then equation (17) can be used for analyzing the flow within a joint as a function of the stress.

5.2. Flow in individual fractures

Because of the essentially planar nature of most fractures in rock, a "parallel plate" concept has frequently been used to describe the flow within the fracture (Snow 57). On the basis of laminar flow between smooth parallel plates, the discharge is given by:

$$q = ge^3i/12\nu = eK_ji \qquad (18)$$

where: q = discharge

g = graviational acceleration

e = effective fracture aperture

i = hydraulic gradient

ν = coefficient of kinematic viscosity

$K_j = ge^2/12\nu = f(e^2)$ = permeability of a fracture

By inspection of equation (18), it may be observed that the flow discharge is extremely sensitive to the effective fracture aperture. The permeability of the fracture is a function of the square of the aperture.

Emperical relationships have been formulated for rock fractures using a Reynolds number/friction factor approach. Sharp and Maini (56) demonstrate that this approach is applicable only to cases in which the flow is both parallel and irrotational. Because of the degree of surface roughness compared to the aperture of natural fractures, these conditions are not satisfied. They report that for a relationship of the form:

$$q \propto e^n$$

the following values for the exponent "n" have been deduced from experiments:

	Rough Fissure	Parallel Flow
Linear Laminar Flow	$n = 2$	$n = 3$
Non-linear Laminar Flow	$1.2 < n < 2$	–
Fully Turbulent Flow	$n = 1.2$	$n = 1.5$

However, this indicates that caution must be exercesed in the application of the simple parallell plate model. No final conclusions can be made without the benefit of additional test results.

Many other investigators have observed that even for "closed" joints at high stress levels, there is a measurable discharge of water. This of course means that the joint cannot be hydraulically closed even if the stress level is high.

In order to quantatively determine the effective aperture, one must back-calculate from observed flow rates and use a theoretical assumption for the joint permeability. Calculations performed in this manner by Witherspoon, et.al. (31), utilizing the theory of laminar flow between parallel plates, give an effective aperture of about 0.5 to 0.2 mm for very low stress levels. For stress levels of the order of 5 MPa, the effective aperture is calculated to be an order of magnitude less.

Since the value of the aperture as obtained by back calculation procedures is a mean value and the measurement of closure from stress loading is more a total value, no comparison can be made without knowing the geometry of the joint. Theoretically, a joint must be completely closed when the stress level is of the order of the compressive strength of the intact block.

Based on the theoretical assumptions of this discussion and a few measurements, the following relationship can be established between joint aperature and stress level:

$$\overline{\sigma} = 0.14 \; \left| \exp \; 29(0.25-b) \; - \; 1 \right| \qquad (19)$$

where the initial joint aperature is about 0.25 mm and the compressive strength is of the order of 200 MPa.

From theoretical considerations based on the roughness and undulation of a joint, the effective aperature must be 30-50 % of the measured maximum or initial joint width. The decrease of the effective aperature proceeds at a greater rate than the major changes of the maximum joint width for certain increases in stress level.

The effective aperature, e, can be estimated from following the equation which is a function of the measured change in joint width Δb:

$$e = 0.40 \cdot b_o \; \left(\frac{b_o \; - \; \Delta b}{b_o}\right)^2 \qquad (20)$$

where b_o is the initial joint width.

This relationship is based on an assumption that the effective aperature depends on the channel area and that the changes of area are of the order of the square of the ratio of the joint width.

With the expression given for the K_j in equation (18) and the assumption of effective aperature, the stress-dependent permeability can be calculated. Comparison between this calculated joint permeability and the major values reported by Witherspoon, et.al. (31) and Iwai (58) indicates that the theoretical curve has a similar shape to tha experimental data but gives a value of 2 to 3 times greater than the major value, as indicated in Figure 13 .
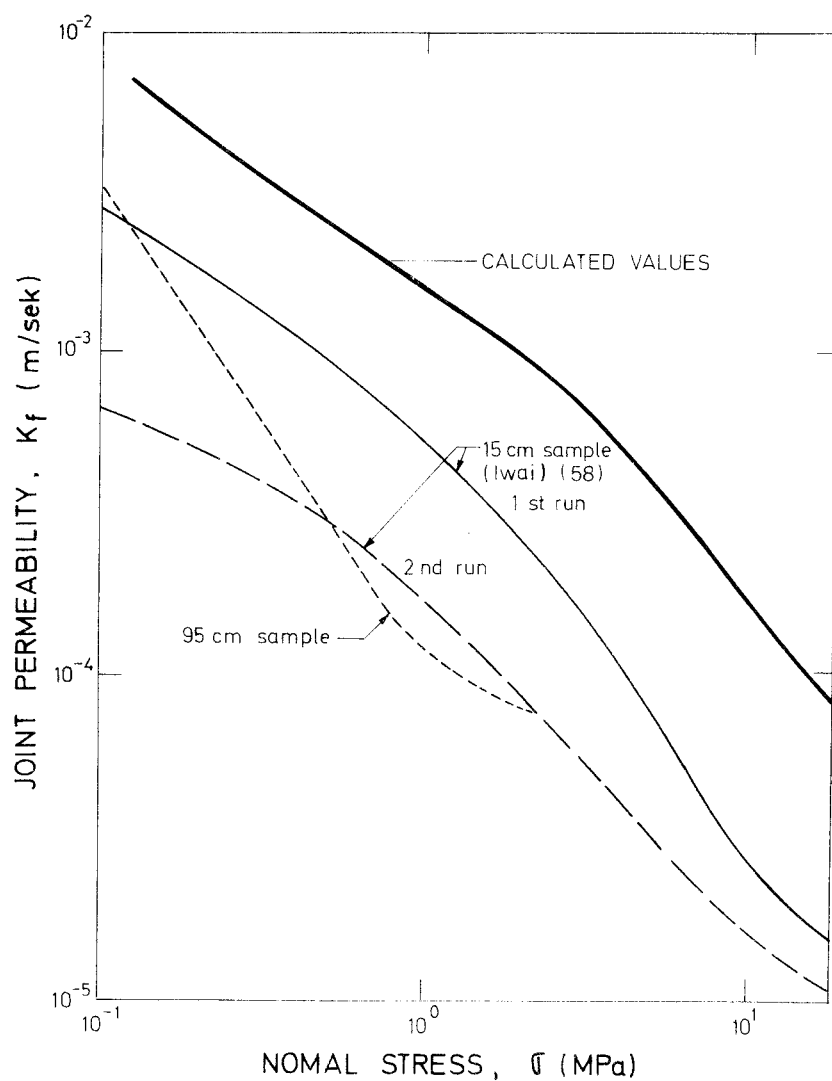
FIGURE 13. JOINT PERMEABILITY AS A FUNCTION
OF STRESS.

5.3   Continuum model

Classical methods of analysis of seepage flow through porous media with Darcy flow employ a continuum model. While the system consists of discrete grains and pores, the overall response is obtained by replacing the discrete elements by a continuum with equivalent conveyance properties. Provided that the size of the area of interest is many orders of magnitude greater than the size of the discrete elements, the continuum model is applicable. This is the case for most problems of seepage flow through soils. The application of this approach to fractures and jointed rocks requires further development as discussed below.

The parallel plate equivalent continuum model described here follows the development given by Snow (57). The continuum model has conveyance properties equivalent with series of parallel fractures. The permeability of the rock mass K_m can then be calculated by assuming a fracture spacing s and effective aperature e according to the equation:

$$K_m = (e/s)K_j \qquad\qquad (21)$$

where K_j is the permeability of the joint.

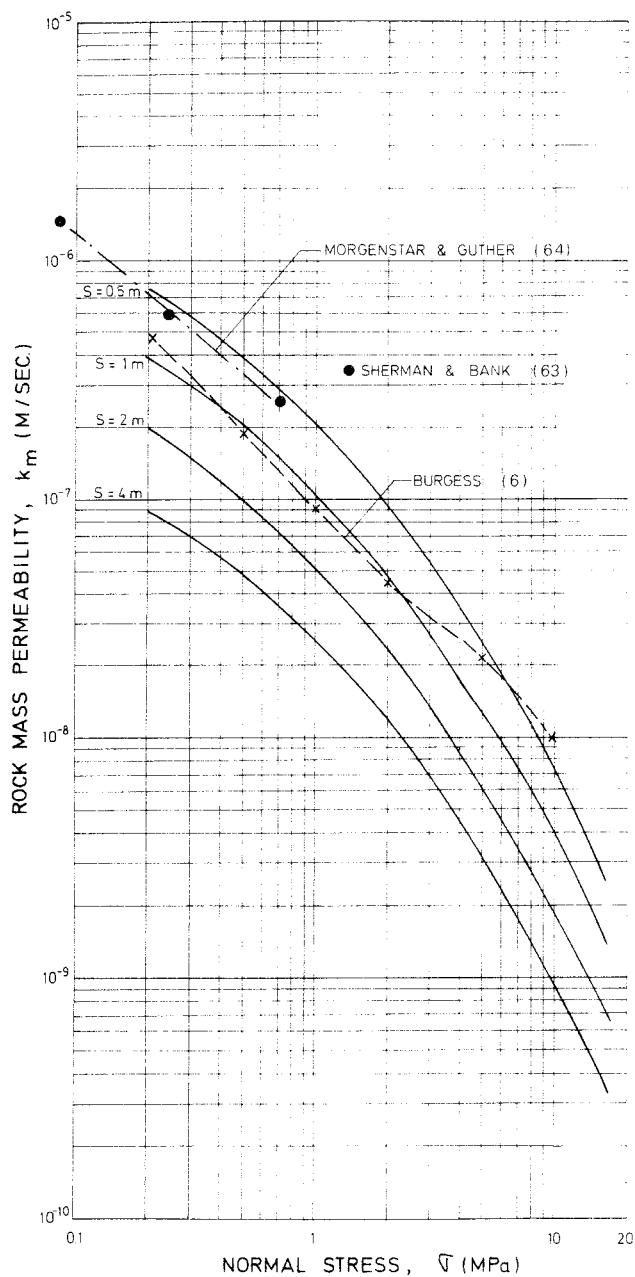Figure 14 presents some calculated results for different values of joint spacing.

FIGURE 14. ROCK MASS PERMEABILITY

5.4   Permeability

Rock mass permeability data available at the current time are limited to those
obtained from SGU drillhole K 1 at Kråkemåla (Oskarshamn) (62) and at Stripa
at 360 m depth, Stille, Lundström (72). The permeability test results for
packer tests at 2 m intervals from the ground surface to a depth of 494 m
at Kråkemåla are shown in Figure 15 and 16. The maximum permeability measured
was 3.93×10^{-7} m/s at a depth of 25 to 27 m. The limit of measurement for
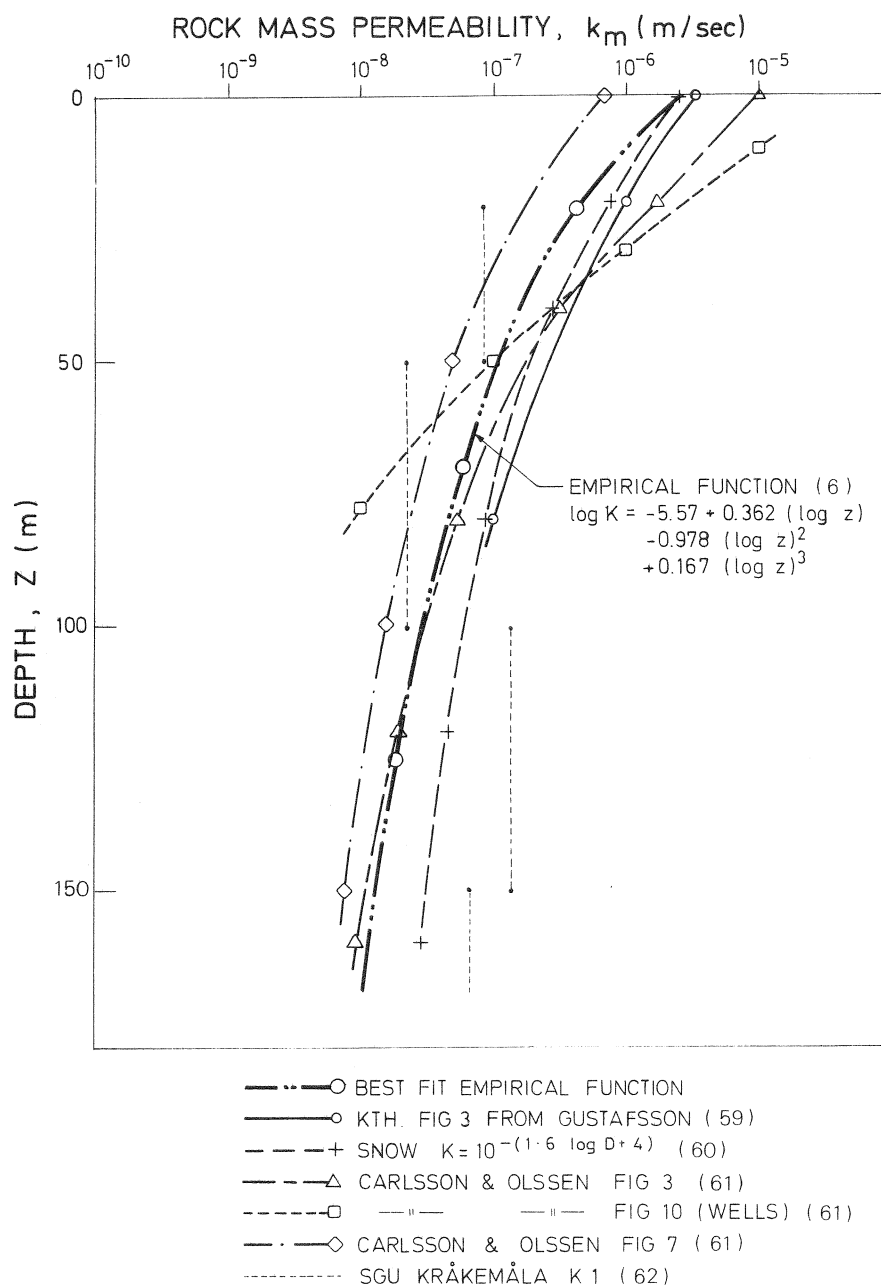the apparatus is 7.89×10^{-10} m/s at a test pressure of 60 m of water.
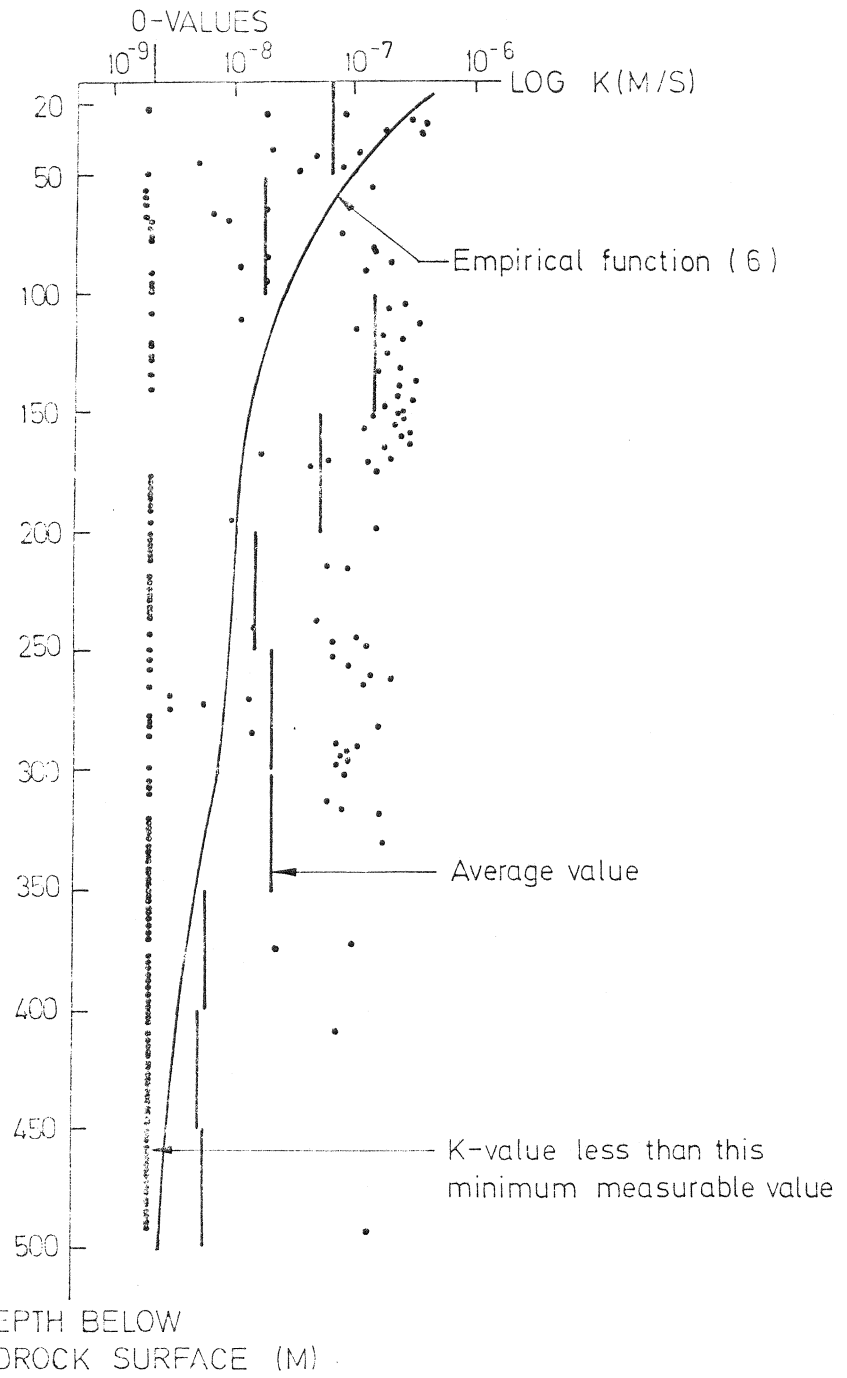
FIGURE 15. PERMEABILITY V DEPTH

FIGURE 16. PERMEABILITY TEST RESULTS
KRÅKEMÅLA DRILLHOLE K1

Values of permeability from measurements in Sweden are plotted as a function of depth in Figure 15. This information includes that obtained from wells in general, drillholes in the Forsmark area, and equivalent zone permeabilities from drillhole K 1. The rock mass permeability measured at Stripa (72) was $0.4 \cdot 10^{-10}$ m/s.

A mean curve for the variation of permeability with depth can be estimated and fit with a third degree polynomial (6). This procedure gives:

$$\log (K/K_O) = 5.57 + 0.362 \log (Z/Z_O) - 0.978 \log(Z/Z_O)^2$$
$$+ 0.167 \log(Z/Z_O)^3 \quad (22)$$

where K_O = unity permeability (m/s)

Z_O = unity depth (m)

It should be recognized that no physical significance can be attached to the form of the function given in equation (22). Most of the values are determined from vertical drillholes and thus preferentially measure horizontal permeability. This relationship can be transformed into an equation involving stress and permeability by use of:

$$Z = \sigma_n/\rho \qquad Z_O = \sigma_O/\rho \qquad (23)$$

where σ_n = normal stress (MPa)

σ_O = unity stress (MPa)

ρ = density

According to Figure 14, this emperical formulation seems to agree quite well with the theoretical equation for a joint spacing of about 1 meter. Further more, equation (22) is conservative for a vertical stress level greater than 5-10 MPa. If the joint is unrecoverable, very little change in the joint width will occur during unloading and the permeability will remain nearly constant at a low value. This in turn implies that a recoverable joint will be conservative for unloading. Measured values of permeability for different stress levels by Sherman and Bank (63), and the proposed linear relationship in a log-log diagram by Morgenstern and Guther (64), seem to fit quite well at low stress level, according to Figure 14 .

The joint porosity n is given by Snow (57):

$$n = e/s \qquad (24)$$

hence: $\qquad n = (3k)^{1/3} \; (2/s)^{2/3} \qquad (25)$

where use is made of equations (18) and (21).

All of the flow models used in this study are two-dimensional. By taking a cubical joint set aligned with the principal of the model, it follows that:

$$k_r = k_h + k_{v_1}$$

$$k_z = k_{v_1} + k_{v2}$$

$$n_r = n_h + n_{v1}$$

$$n_z = n_{v1} + n_{v2}$$

where the porosities are computed using the relationship in terms of permeabilities. k_r, n_r and k_z, n_z are the permeabilities and porosities in the radial and vertical directions, respectively. h is the horizontal joint set, and v1 and v2 are the two vertical joint sets.

As discussed previously, a fracture aperture is a function of the surrounding state of stress. In turn, the equivalent continuum permeability, being a function of the fracture aperture, should also decrease with depth due to increasing vertical stress.

Measurements of the in situ stresses in the Precambrian shield indicate a ratio of the horizontal stress to the vertical stress of 1.5 to 6 at depth of 200 m, and 1 to 3 at depth of 500 m. As a consequence, vertically oriented fractures might be expected to show less variation in aperture and spacing with depth than horizontal fractures. Thus, the equivalent continuum vertical permeability should exhibit a less marked decrease with depth than the horizontal permeability. These conclusions have been used in formulating the material properties used in the numerical simulation.

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FÖRTECKNING ÖVER KBS TEKNISKA RAPPORTER

20   Tektonisk analys av södra Sverige, Vättern - Norra Skåne
      Kennert Röshoff
      Erik Lagerlund
      Lunds Universitet och Högskolan Luleå september 1977

21   Earthquakes of Sweden 1891 - 1957, 1963 - 1972
      Ota Kulhánek
      Rutger Wahlström
      Uppsala Universitet september 1977

22   The influence of rock movement on the stress/strain
      situation in tunnels or bore holes with radioactive con-
      sisters embedded in a bentonite/quartz buffer mass
      Roland Pusch
      Högskolan i Luleå 1977-08-22

23   Water uptake in a bentonite buffer mass
      A model study
      Roland Pusch
      Högskolan i Luleå 1977-08-22

24   Beräkning av utlakning av vissa fissionsprodukter och akti-
      nider från en cylinder av franskt glas
      Göran Blomqvist
      AB Atomenergi 1977-07-27

25   Blekinge kustgnejs, Geologi och hydrogeologi
      Ingemar Larsson   KTH
      Tom Lundgren   SGI
      Ulf Wiklander   SGU
      Stockholm, augusti 1977

26   Bedömning av risken för fördröjt brott i titan
      Kjell Pettersson
      AB Atomenergi 1977-08-25

27   A short review of the formation, stability and cementing
      properties of natural zeolites
      Arvid Jacobsson
      Högskolan i Luleå 1977-10-03

28   Värmeledningsförsök på buffertsubstans av bentonit/pitesilt
      Sven Knutsson
      Högskolan i Luleå 1977-09-20

29   Deformationer i sprickigt berg
      Ove Stephansson
      Högskolan i Luleå 1977-09-28

30   Retardation of escaping nuclides from a final depository
      Ivars Neretnieks
      Kungliga Tekniska Högskolan Stockholm 1977-09-14

31   Bedömning av korrosionsbeständigheten hos material avsedda
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32   Long term mineralogical properties of bentonite/quartz
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     Preliminär rapport november 1977
     Slutrapport februari 1978
     Roland Pusch
     Arvid Jacobsson
     Högskolan i Luleå

33   Required physical and mechanical properties of buffer masses
     Roland Pusch
     Högskolan Luleå 1977-10-19

34   Tillverkning av bly-titan kapsel
     Folke Sandelin AB
     VBB
     ASEA-Kabel
     Institutet för metallforskning
     Stockholm november 1977

35   Project for the handling and storage of vitrified high-level
     waste
     Saint Gobain Techniques Nouvelles October, 1977

36   Sammansättning av grundvatten på större djup i granitisk
     berggrund
     Jan Rennerfelt
     Orrje & Co, Stockholm 1977-11-07

37   Hantering av buffertmaterial av bentonit och kvarts
     Hans Fagerström, VBB
     Björn Lundahl, Stabilator
     Stockholm oktober 1977

38   Utformning av bergrumsanläggningar
     Arne Finné, KBS
     Alf Engelbrektson, VBB
     Stockholm december 1977

39   Konstruktionsstudier, direktdeponering
     ASEA-ATOM
     VBB
     Västerås

40   Ekologisk transport och stråldoser från grundvattenburna
     radioaktiva ämnen
     Ronny Bergman
     Ulla Bergström
     Sverker Evans
     AB Atomenergi

41   Säkerhet och strålskydd inom kärnkraftområdet.
     Lagar, normer och bedömningsgrunder
     Christina Gyllander
     Siegfried F Johnson
     Stig Rolandson
     AB Atomenergi och ASEA-ATOM

42　Säkerhet vid hantering, lagring och transport av använt
　　kärnbränsle och förglasat högaktivt avfall
　　Ann Margret Ericsson
　　Kemakta november 1977

43　Transport av radioaktiva ämnen med grundvatten från
　　ett bergförvar
　　Bertil Grundfelt
　　Kemakta november 1977

44　Beständighet hos borsilikatglas
　　Tibor Lakatos
　　Glasteknisk Utveckling AB

45　Beräkning av temperaturer i ett envånings slutförvar i berg
　　för förglasat radioaktivt avfall Rapport 3
　　Roland Blomquist
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46　Temperaturberäkningar för använt bränsle
　　Taïvo Tarandi
　　VBB

47　Teoretiska studier av grundvattenrörelser
　　Preliminär rapport oktober 1977
　　Slutrapport februari 1978
　　Lars Y Nilsson
　　John Stokes
　　Roger Thunvik
　　Inst för kulturteknik KTH

48　The mechanical properties of the rocks in Stripa,
　　Kråkemåla, Finnsjön and Blekinge
　　Graham Swan
　　Högskolan i Luleå 1977-09-14

49　Bergspänningsmätningar i Stripa gruva
　　Hans Carlsson
　　Högskolan i Luleå 1977-08-29

50　Lakningsförsök med högaktivt franskt glas i Studsvik
　　Göran Blomqvist
　　AB Atomenergi november 1977

51　Seismotechtonic risk modelling for nuclear waste disposal
　　in the Swedish bedrock
　　F Ringdal
　　H Gjöystdal
　　E S Hysebye
　　Royal Norwegian Council for scientific and industrial
　　research

52　Calculations of nuclide migration in rock and porous media,
　　penetrated by water
　　H Häggblom
　　AB Atomenergi 1977-09-14

53   Mätning av diffusionshastighet för silver i lera-sand-bland-
      ning
      Bert Allard
      Heino Kipatsi
      Chalmers tekniska högskola 1977-10-15

54   Groundwater movements around a repository

      54:01   Geological and geotechnical conditions
                 Håkan Stille
                 Anthony Burgess
                 Ulf E Lindblom
                 Hagconsult AB september 1977

      54:02   Thermal analyses
                 Part 1 Conduction heat transfer
                 Part 2 Advective heat transfer
                 Joe L Ratigan
                 Hagconsult AB september 1977

      54:03   Regional groundwater flow analyses
                 Part 1 Initial conditions
                 Part 2 Long term residual conditions
                 Anthony Burgess
                 Hagconsult AB oktober 1977

      54:04   Rock mechanics analyses
                 Joe L Ratigan
                 Hagconsult AB september 1977

      54:05   Repository domain groundwater flow analyses
                 Part 1 Permeability perturbations
                 Part 2 Inflow to repository
                 Part 3 Thermally induced flow
                 Joe L Ratigan
                 Anthony S Burgess
                 Edward L Skiba
                 Robin Charlwood

      54:06   Final report
                 Ulf Lindblom et al
                 Hagconsult AB oktober 1977

55   Sorption av långlivade radionuklider i lera och berg
      Del 1 Bestämning av fördelningskoefficienter
      Del 2 Litteraturgenomgång
      Bert Allard
      Heino Kipatsi
      Jan Rydberg
      Chalmers tekniska högskola 1977-10-10

56   Radiolys av utfyllnadsmaterial
      Bert Allard
      Heino Kipatsi
      Jan Rydberg
      Chalmers tkniska högskola 1977-10-15