SEISMIC METHODS IN ENGINEERING GEOLOGY

by

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This thesis describes an investigation of the possibilities to measure ground-mass and especially joint parameters (joint direction and joint density) for engineering purposes by means of seismic waves. Therefore a literature study and a field investigation in the United Kingdom were done. The results show that seismic wave behaviour in one ground-mass can vary widely as result of an anisotropic ground-mass, where the anisotropic character is caused by orientated discontinuities, e.g. jointing, and that fan-shooting can be a very useful method to determine these anisotropic ground-mass parameters. In some cases it even is a necessary measuring method in order to determine the proper number of ground-mass layers.
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The stability of the ground mass is important in tunnel construction. References 19, 20, 24, 31, and 34 describe the evaluation from the study of seismic waves, degree of weathering and of the degree of jointing, which have great influence on the ground mass.

PART I

LITERATURE REVIEW
SECTION I
INTRODUCTION

The use of conventional surface seismic methods in engineering geology to assist in determining subsurface geological structures is well known. The different methods and their applications are widely described in the literature. That some of these methods can as well be used to estimate or even determine the elastic parameters and the quality of the subsurface ground is less well known.

Seismic methods are based upon propagation of an acoustic (seismic) wave through the ground. The behaviour of the wave in the ground-mass depends on the nature of the ground including all its irregularities like jointing, differences in mineral composition, porosity, etc. The influence of these factors on the seismic wave can be, to some degree, assessed and certain properties of the whole ground-mass determined.

Seismic techniques offer the opportunity to examine the properties of the whole ground-mass likely to be influenced by the construction of an engineering work. Conventional testing of samples in the laboratory or in the field can assess only the properties of a small part of the ground-mass.

In this literature review some papers will be referred which describe seismic methods to determine ground-mass parameters. The methods were mostly developed and used in connection with an actual site-investigation or existing engineering work. Because in most cases the application is not reviewed in the literature review, examples of engineering works for which seismic methods have been used are given below.

1. Dams & hydraulic works.

A sound knowledge of the elastic parameters of the ground-mass under a dam is of great importance in the construction and the proper working of dams. The degree of (especially open-) jointing and the directions of the jointing under and around the dam-site is also important. References 19, 21, and 23 describe methods to estimate the elastic parameters of the ground and the degree, direction and nature of jointing in connection with dams and hydraulic works.
2. Tunnelling and underground mining.

The stability of the ground-mass is important in tunnel construction. References 19, 20, 24, 31, and 34 describe the evaluation from the study of seismic waves, of the degree of weathering and of the degree of jointing, which have great influence on the ground-mass stability.

3. Nuclear power plants & large civil works.

The same ground-mass parameters are important as those for dams; Rodrigues (reference 29) describes the use of some seismic methods in the site-investigation for a nuclear power plant.

4. Amount of fragmentation resulting from explosives.

The power of an explosive charge influences the degree of fragmentation of the ground-mass and thus influences the resulting size of the ground-mass fragments. If the ground-mass fragments are to be used further, the usefulness of the material depends on the size of the fragments. On the other hand explosives are quite expensive, so that it is economical to use explosive charges as small as possible to get the necessary fragment size. McKenzie et al. (reference 22) describe the possibility of deducing the degree of jointing, and thus the fragment size, resulting from explosive charges, out of seismic velocities.

5. Excavation of ground-masses.

Weathering, jointing and ground-strength are the main parameters on which the method of excavation of a ground-mass depends. There are many papers which deal with the possibilities of establishing the excavation method from seismic studies (generally from the velocity of compressional waves). Most papers are from the manufacturers of the excavation machines (e.g. Caterpillar) and the application of the methods described is often only of use in combination with their machines.

A list as above gives the idea that only one or two particular ground-mass parameters are of importance in a particular engineering work. In reality all ground-mass parameters are important in predicting the behaviour of the ground-mass in a certain application, and thus is it possible to use, for example, the seismic methods developed for dam-building for the site-investigation on behalf of a tunnel.
SECTION II
THE APPLICATION OF THE SEISMIC METHODS

Seismic measurements in engineering-geology may be undertaken in three work ways, depending upon the raypath of the wave. These are:
- direct(1) waves - on the surface of the ground
- refracted waves - on the surface of the ground
- reflected waves - in boreholes

2.1 DIRECT WAVES
1. On the surface of the ground

The shock source and one or more geophones (often up to 12 or 24) are located on the surface of the ground to be investigated, so that the layer directly under the source and receivers is examined. Distances between source and geophones can range from 1 to 100 m. A weight-drop device or a small explosive charge may be used as the source of the shock.

2. Measurements in boreholes
a. Along a borehole
The source is located at the surface near the top of the borehole and a probe containing one or more geophones, sometimes arranged in special configurations, is let down along the borehole.

b. Cross-hole
Two boreholes are required. The source, which may provide the shock by explosive or other

(1) For a description of the refraction/reflection laws, the reader is referred to the textbooks on seismic methods.
means activated by an electrical signal, is placed in one borehole. The receiving unit is let down in the second borehole, so that the acoustic properties of the ground are measured between the two boreholes.

2.2 REFRACTED WAVES

1. On the surface of the ground

When the ground is divided into various layers, then not only the top layer is of interest, but also the deeper layers. Information about these deeper layers can be obtained by a study of the refracted waves. The interpretation of the refracted waves is more difficult than that of direct waves due to the fact that also the layers above a particular refractor will have an influence on the refracted signal.

2. Measurements in boreholes

Source and geophones separated by a fixed distance in one borehole.

The acoustic properties of the ground immediately adjacent to the borehole, between source and geophones, are measured. The ground immediately adjacent to the borehole will have an other velocity due to the bore process itself and/or to the bore hole liquids. This causes the measuring of refracted waves.

2.3 REFLECTED WAVES

In engineering-geology these are only used in marine surveys to obtain information about layers below the sea floor.

Although, it must be noted, that in future reflected wave methods will be more used in land surveys because the apparatus become cheaper and more handsome.

The literature review deals with direct and refracted waves only. Investigations which are based on reflected waves use mostly apparatus and methods developed for deep seismic exploration. The problems which are initiated with these me-
Methods are very different from those of the direct and refracted wave methods, so that they have not been incorporated in this review.
SECTION III
THEORETICAL CONCEPTS

3.1 ELASTIC THEORY - WAVE PROPAGATION

If an acoustic(1) wave propagates through the ground, the stresses on an infinite small block of ground are as shown below.

On the front face the stresses are:

\[
\sigma_{xx} + \frac{\partial \sigma_{xx}}{\partial x} \, dx, \quad \sigma_{xy} + \frac{\partial \sigma_{xy}}{\partial x} \, dx, \quad \sigma_{xz} + \frac{\partial \sigma_{xz}}{\partial x} \, dx
\]

Since these are opposite to those operating on the rear face, the net (unbalanced) forces are:

\[
\frac{\partial \sigma_{xx}}{\partial x} \, dx \, dy \, dz, \quad \frac{\partial \sigma_{xy}}{\partial x} \, dx \, dy \, dz, \quad \frac{\partial \sigma_{xz}}{\partial x} \, dx \, dy \, dz
\]

Total force in the direction of the X-axis per unit volume:

\[
\frac{\partial \sigma_{xx}}{\partial x} + \frac{\partial \sigma_{xy}}{\partial y} + \frac{\partial \sigma_{xz}}{\partial z} = \int \frac{\partial^2 u}{\partial t^2} \, dt^2
\]

(1) For a more detailed description of elastic wave propagation, the reader is referred to the textbooks on seismic methods.
\( x, y, z = \) directions along the axis
\( \sigma = \) disturbance
\( t = \) time
\( \rho = \) density

with for a homogeneous isotropic medium:

\[ \sigma_{ii} = \lambda \Delta + 2 \nu \epsilon_{ii} \]  
(Law of Hooke)

\[ \sigma_{ij} = \nu \epsilon_{ij} \quad i \neq j \]

\( \Delta = \) the change in volume

This results in the general wave-equation:

\[ \frac{1}{v^2} \frac{\partial^2 \psi}{\partial t^2} = \nabla^2 \psi \quad (3.2) \]

\( \psi \) is the disturbance travelling through the medium and can be a volume disturbance: \( \Delta \) with \( v^2 = (\lambda + 2\nu)/\rho \)


or \( \psi \) can be a rotational disturbance: \( \theta_i \) \( (i= x, y, z) \)

with \( v^2 = \nu / \rho \)

\( \psi \) is a function of place and time and can be a harmonic wave.

A harmonic plane wave solution is:

\[ \psi = A \cos \frac{2\pi}{\lambda} (x - vt) \quad (3.3) \]

\( A = \) amplitude of the wave
\( \lambda = \) wavelength
\( t = \) time
\( x = \) place along \( x \)-axis
\( V = \) phase velocity

The Lame constants \( \lambda \) and \( \nu \) describe the elastic properties of the medium.
3.2 GROUND PARAMETERS WHICH INFLUENCE SEISMIC
WAVE PROPAGATION

Ground is built up out of different materials, normally minerals or aggregates of minerals, arranged in bodies of a particular shape, which for the purpose of this thesis are described as 'grains'.
The 'grains' are arranged and orientated in layers or irregular bodies as the result of the origin and tectonic history of the ground.
Between the 'grains' and also between particles in the 'grains' there are often voids filled with liquids, (in shallow ground normally water) or with gas. Also as a result of its tectonic history the ground is often jointed, cleaved, faulted, etc.
Any part of the ground is subject to lithostatic pressure, causing an effective stress, due to the weight of the ground-mass above and sometimes as well to stresses resulting from past and present tectonic and geomorphological forces.
The lithostatic pressure is generally expected to be isotropic, although this is not always true.
The other effects cause a more or less severe anisotropy in the seismic behaviour of the ground.
The different features and their effects on the elastic properties of the ground and thus on the behaviour of the acoustic waves are listed below.

3.2.1 Mineral particles

The mineral particles inside a 'grain' often have an anisotropic elastic structure. Also the shape of the particles is normally not spherical.
When the mineral particles and/or the grains have been orientated due to geological processes, severe anisotropy can exist. If this anisotropy is related to a layered structure, properties in the direction along the layers will be different to those perpendicular to the layering.

(2) In the literature the lithostatic pressure is often designated as hydrostatic pressure, but the hydrostatic pressure is defined as an isotropic pressure, so that it is better to define the ground-mass pressure, which has not to be isotropic, as the lithostatic pressure.
3.2.2 Porosity and degree of water-saturation

Because liquids or gasses can not transmit shear strains, the voids and the filling material have a major influence on the properties of acoustic waves.

Tests reported by various authors (references 12, 14, and 35) show that:

1. **velocity**
   a. the shearwave velocity decreases with increasing degree of saturation,
   b. the compression wave velocity decreases with an increasing saturation up to 95% saturation and then shows a sharp increase from 95% to 100% saturation,

2. **attenuation**
   a. The shearwave attenuation decreases with an increase in the degree of saturation,
   b. the compression-wave attenuation increases with an increase of saturation up to 95% and decreases from 95 to 100% saturation.

The attenuation relations (reference 35) were obtained by laboratory tests on resonating bars of Massilon sandstone with frequencies between 500 and 1700 Hz.

3.2.3 Jointing

Due to previous or present stress configurations most ground-masses are jointed or cleaved. Joints differ from porosity voids in both shape and dimensions.

Voids have mainly a circular shape and have approximately throughout a certain type of ground the same dimensions. Joints are mostly planar and normally there is a large difference in planar dimensions between the various groups of joints found in one type of ground. Joints can be divided in two types with a different influence on acoustic waves:

1. Closed joints; There are joints (mechanical discontinuities) in which both sides of the joint surfaces are in more- or-less continuous contact. (Joint surfaces are often coated with a different material than the ground. This allows the closed joints to be seen.)
2. Open joints; These are joints in which the contact between opposing joint surfaces are discontinuous. The open spaces between contact points are generally partially or wholly filled with water, groundfragments, and perhaps clay.

Joints with dimensions comparable to or smaller than the wavelength of the acoustic wave will have, apart from the form of the joints, an influence on the acoustic wave behaviour which is comparable to the influence of porosity voids.

Joints with dimensions larger than the wavelength of the acoustic wave will act as a different planar medium and will have an influence which can be described by the reflection/refraction laws.

The influence a closed joint has on an acoustic wave will depend for a large part on the pressure with which the joint surfaces are pressed together. This pressure may be effective or tectonic stresses.

3.2.4 Effective stress

The grains at a certain depth will be pressed together with a certain effective stress as the result of the weight of the ground above. The higher this stress the better and larger the grain contacts and the smaller the openness of the joints. This means that the shear- and compression-wave velocities increase with increasing effective stress and that the loss of energy due to grain-boundary friction decreases (references 25, 33, and 35).

3.2.5 Tectonic (directional-) stresses

Tectonic stresses have the same effect as the effective stresses, but only in a particular direction. Tests and measurements of acoustic wave velocities have been undertaken (particularly in coal-mining) to estimate the direction of developing and in-situ stresses, which are of importance with respect to excavation stability (reference 31).

3.2.6 Weathering

Weathering causes a widening of the joints and decomposition of the ground material, the latter leading to an increase in porosity.
Both of these processes decrease the velocity and increase the loss of energy of the acoustic wave.

3.2.7 Waterflow

Groundwaterflow has a directional influence on the acoustic waves, but this is likely to be negligible in comparison with the other anisotropic features described above. The literature does not describe any evidence of waterflow influence on seismic measurements to shallow depths.
3.3 NON ELASTIC GROUND MODELS

The wave equation based on an elastic model holds only for a pure elastic homogeneous medium. As described in the foregoing section, natural ground does not fit this theoretical model.

As a wave passes through the ground the energy is continually converted from kinetic into elastic potential energy and reverse. During this process some of the energy is converted into heat. Since part of the heat conduction away there is a loss of energy.

The cause of the transformation of energy into heat has long been thought of to be grain-boundary friction. Recent investigations have shown that this process is not important in at least deeper ground layers.

K. Winkler et. al. have shown (reference 35) that it is likely that fluid-flow energy losses under higher confining pressure are more important than grain-boundary friction. Laboratory tests under low confining pressure have not been done, so the reasons for transforming energy into heat are, for shallow ground-masses, still uncertain. However it is likely that grain-boundary friction still plays an important role and that other factors such as fracturing, piezo-electricity, thermo-electricity, etc. will also attenuate and change the velocity of the acoustic wave.

Although the energy losses in shallow ground-masses are not quite understood, some authors have tried to establish a model for wave propagation in shallow ground-masses which allows for an attenuation of the acoustic wave.

In the next section a visco-elastic model, described by Jean Marc Roussel (reference 30), will be reviewed.

3.3.1 Visco-elastic theory - wave propagation

A visco-elastic model can be described by a combination of a spring and a dashpot.

\[
\text{spring (E)} \quad \text{dashpot (\eta)} \quad \text{mass: m}
\]
This model operates in accordance with the differential equation:

\[
d\frac{x}{dt^2} + \frac{\eta}{dt} + \frac{E}{m} x = 0
\]  

(3.4)

\( m \) = mass  
\( \eta \) = coefficient of viscosity  
\( E \) = elastic

Moving wave in a visco-elastic system
If \( e_1 \) is the deformation and \( \frac{de_1}{dt} \) is the deformation velocity then the following three equations are valid:

\[
n_1 = \alpha e_1 + k_1 \frac{de_1}{dt}
\]

\[
n_2 = \beta e_1 + k_2 \frac{de_1}{dt}
\]

\[
n_3 = \gamma e_1 + k_3 \frac{de_1}{dt}
\]

(3.5)

\( n_1, n_2, n_3 \) = force  
\( k_1, k_2, k_3 \) = coefficients of viscosity  
\( \alpha, \beta, \gamma \) = elastic

If the ground-masses are isotropic then \( \beta = \gamma \) and \( k_2 = k_3 \).  
If \( e_2 \) and \( e_3 \) are the deformation in the orthogonal directions, with \( \frac{de_2}{dt} \) and \( \frac{de_3}{dt} \) as the deformation velocities, then:

\[
n_1 = \beta e_2 + k_2 \frac{de_2}{dt}
\]

\[
n_2 = \alpha e_2 + k_1 \frac{de_2}{dt}
\]

\[
n_3 = \beta e_3 + k_2 \frac{de_3}{dt}
\]

\[
n_1 = \beta e_3 + k_2 \frac{de_3}{dt}
\]

\[
n_2 = \beta e_3 + k_2 \frac{de_3}{dt}
\]

\[
n_3 = \alpha e_3 + k_1 \frac{de_3}{dt}
\]

(3.6) (3.7)
Combining the systems (3.5), (3.6) and (3.7) give:

\[
n_1 = \alpha e_1 + \beta (e_2 + e_3) + k_1 \frac{de_1}{dt} + k_2 \left( \frac{de_2}{dt} + \frac{de_3}{dt} \right)
\]

(3.8)

and two other analogue equations.

If:

\[
\alpha = \lambda + 2\gamma \\
\beta = \gamma \\
Q = \sum_i e_i
\]

the results are:

\[
n_1 = \lambda \theta + 2\gamma e_1 + \lambda' \frac{d\theta}{dt} + 2\gamma' \frac{de_1}{dt}
\]

(3.9)

and

\[
t_{ij} = \lambda \delta_{ij} + 2\gamma e_{ij} + \lambda' \frac{d\theta}{dt} \delta_{ij} + 2\gamma' \frac{de_{ij}}{dt}
\]

(3.10)

\( \delta_{ij} \) = Kronecker symbol

If a wave is travelling through a visco-elastic medium with \( u \) the displacement in the x-direction and \( v \) the displacement orthogonal on the x-axis: \( y \), the deformation becomes:

\[
e_{xx} = \frac{\lambda}{\gamma} u \\
e_{xy} = \frac{1}{2} \left( \frac{\lambda}{\gamma} u + \frac{\lambda}{\gamma} v \right)
\]

(3.11)

\[
e_{yy} = \frac{\lambda}{\gamma} v
\]

and (3.10) becomes:

\[
t_{xx} = \left( \lambda + 2\gamma \right) e_{xx} + \lambda e_{yy} + \left( \lambda' + 2\gamma' \right) \frac{\lambda e_{xx}}{\gamma} + \lambda' \frac{\lambda e_{yy}}{\gamma}
\]

(3.12)

If on a square \( S \) with a mass of \( \rho S dx \) a force \( F \) is working:

\[
dF = (t_{xx}\chi + t_x) S = \frac{\delta t_{xx}}{\delta x} S dx
\]

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The inertial force is:

\[ \rho S \frac{\partial^2 u}{\partial t^2} \]

and thus

\[ \frac{\partial t}{\partial xx} S \frac{\partial x}{\partial x} = \rho S \frac{\partial^2 u}{\partial t^2} dx \]

from (3.12)

\[ \frac{\partial t}{\partial xx} = (\lambda + 2\mu) \frac{\partial e}{\partial xx} + (\lambda' + 2\mu') \frac{\partial}{\partial x} \left( \frac{\partial e}{\partial t} \right) \]

because \( e_{yy} \) and \( e_{xy} \) do not introduce forces along the x-axis.

\[ e_{xx} = \frac{\partial u}{\partial x} \]

\[ \frac{\partial t}{\partial xx} = (\lambda + 2\mu) \frac{\partial^2 u}{\partial x^2} + (\lambda' + 2\mu') \frac{\partial^3 u}{\partial x^2 \partial t} \]

(3.11)

and finally:

\[ (\lambda + 2\mu) \frac{\partial^2 u}{\partial t^2} + (\lambda' + 2\mu') \frac{\partial^3 u}{\partial x^2 \partial t} - \int \frac{\partial^2 u}{\partial t^2} = 0 \]

(3.13)

This equation is found by a number of authors as: Lamb, Kolsky, Knopoff, Sinitzyn, but with different forms for

\[ \frac{\partial^3 u}{\partial x^2 \partial t} \]

From the dimensions it appears that

\[ [\lambda' + 2\mu'] \propto \rho \eta \]

and thus (3.13) becomes:

\[ (\lambda + 2\mu) \frac{\partial^2 u}{\partial x^2} + \int \eta \frac{\partial^3 u}{\partial x^2 \partial t} - \int \frac{\partial^2 u}{\partial t^2} = 0 \]

(3.14)
To get a solution from (3.14) Roussel proposed that
\[ u = u_0 e^{-ax} e^{i\omega (t-x/V)} \]  \hspace{1cm} (3.15)
is likely to describe the amplitude of a moving wave in which \( a \) is the attenuation coefficient and \( V \) the wave velocity.

Equation (3.14) can be written in the form:
\[ \frac{\partial^2 u}{\partial x^2} + \frac{1}{\lambda + 2\mu} \frac{\partial^3 u}{\partial x^2 \partial t} - \frac{2}{\lambda + 2\mu} \frac{\partial u}{\partial t^2} = 0 \]  \hspace{1cm} (3.16)
and if
\[ A = \frac{\rho \eta}{\lambda + 2\mu} \quad B = \frac{\rho}{\lambda + 2\mu} \]
and from (3.15)
\[ \frac{\partial^2 u}{\partial x^2} = (-a - \frac{i\omega^2}{V}) u_0 e^{-ax} e^{i\omega (t-x/V)} \]
\[ \frac{\partial^3 u}{\partial x^2 \partial t} = (-a - \frac{i\omega^2}{V}) i\omega u_0 e^{-ax} e^{i\omega (t-x/V)} \]
\[ \frac{\partial^2 u}{\partial t^2} = (i\omega)^2 u_0 e^{-ax} e^{i\omega (t-x/V)} \]
and equation (3.16) becomes:
\[ \left(-a - \frac{i\omega^2}{V}\right) + A i\omega \left(-a - \frac{i\omega^2}{V}\right) - B (i\omega)^2 = 0 \]
The real part is than:
\[ a^2 - \frac{\omega^2}{V^2} - 2 \frac{A \omega^2}{V} x + B \omega^2 = 0 \]  \hspace{1cm} (3.17)
and the imaginary part:

\[ \frac{2a \omega}{V} + A \frac{a^2 \omega}{V^2} = 0 \]  \hspace{1cm} (3.18)

from (3.17) and (3.18) can be found:

\[ \eta = \frac{2a \sqrt{3} \omega^2}{(\omega^2 + a^2 \sqrt{3} \omega^2)^2} \]

and

\[ \lambda + 2\mu = \frac{\rho V^2 \omega^2 - a^2 \omega^2}{(\omega^2 + a^2 \omega^2)^2} \]

From the elasticity coefficient obtained by static measurements:

\[ \lambda + 2\mu = \frac{1 - \nu_{\text{stat}}}{(1 + \nu_{\text{stat}})(1 - 2\nu_{\text{stat}})} = E_{\text{stat}} f(\nu_{\text{stat}}) \]

in which \( \nu \) means the Poisson modulus from a static test and \( E \) the Young's modulus from a static test. If \( V \) is independent of the frequency, which is true if the frequency range is not too wide (200 - 600 Hz), then

\[ \rho V^2 = E_{\text{seis}} \frac{1 - \nu_{\text{seis}}}{(1 + \nu_{\text{seis}})(1 - 2\nu_{\text{seis}})} = E_{\text{seis}} f(\nu_{\text{seis}}) \]

this is true for an elastic model, but will be used for a visco-elastic model. The quotient between the static and seismic \( E \) moduli becomes:

\[ \frac{E_{\text{stat}} f(\nu_{\text{stat}})}{E_{\text{seis}} f(\nu_{\text{seis}})} = \frac{\omega^2 - a^2 \omega^2}{\omega^2 (\omega^2 + a^2 \omega^2)^2} \]

Although the visco-elastic model is likely to describe shallow ground better than the elastic model, most authors describe the behaviour of a seismic wave in a ground-mass with an elastic model.
SECTION IV
COMPARISON OF SEISMIC AND MECHANICAL GROUND-MASS PARAMETERS

4.1 COMPARISON OF ACOUSTIC FIELD AND LABORATORY MEASUREMENTS

Seismic field measurements differ from (ultrasonic) laboratory measurements through:

1. volume influence,
2. differences in the frequencies of the used waves,
3. ..., between ground in-situ and sample.

4.1.1 Volume influenced by the acoustic wave

An acoustic wave propagating through the ground is expected to be influenced by a certain volume of ground. This volume \( V \) is related to the wavelength \( \lambda \) of the acoustic wave and of course to the distance \( l \) between source and receiving point.

According to Lykoshin et. al. (reference 21):

\[
V \approx \pi (0.25 \lambda)^2 l \tag{4.1}
\]

Engineering-geology seismic surveys use a hammer or weight-drop source or a small explosive charge, with a distance between source and geophones from 5 to 100m. The principal frequencies generated by the energy sources given above are between 10 and 600 Hz. If, for example, the distance from source to geophone was 10m, the principal frequency 150 hz and the phase velocity 2000 m/s, following equation (4.1) the volume \( V \) influenced will be about 350 m\(^3\).

In seismic laboratory measurements an ultrasonic test device is used. The frequencies are between 1,000 and 100,000 Hz and with sample lengths of 10 to 100 cm, the measured volume becomes, for a sample length of 20 cm with a test frequency of 10,000 Hz and a phase velocity of 2000 m/s, about 0.002 m\(^3\).
4.1.2 Significance of wave frequencies

Frequencies of waves in seismic field tests are from 10 to 600 Hz and frequencies generated by ultrasonic laboratory test equipment are from 1,000 to 100,000 Hz.

It is known from wave theory that when a wave encounters a feature with different elastic constants whose radius of curvature is comparable to or smaller than the wavelength the wave will be diffracted rather than reflected and refracted.

The result is that when such a feature has dimensions comparable to or smaller than the wavelength the wave will pass the feature without significant reflection and/or refraction.

Because of the difference between the frequencies used in seismic field tests and in ultrasonic laboratory tests a feature in the ground has an influence on a field wave different from the influence the same feature will have on a laboratory wave.

This is proved by investigations done by Lykoshin et al. (reference 21), who measured the compressional velocities of acoustic waves with ultrasonic and seismic frequencies at different angles to the anisotropic structures. Figure 1 gives as an example the summary velocity indicatrices for two limestone bodies.

![Indicatrices](image)

(After Lykoshin)

I = ultrasonic compression wave velocity
II = seismic
velocity in km/s

Figure 1: Seismic and ultrasonic velocity against joint orientation
The first of these bodies (sample a) was composed of poorly but regularly jointed rock and showed relatively constant velocities for both seismic and ultrasonic frequencies independent of orientation (fig. 1,a). In the second body (sample b) consisting of highly jointed limestone with lithogenic and large tectonic fissures, ultrasonic velocities varied with orientation but seismic velocities remained almost constant (fig. 1,b). The difference in shape of the velocity/orientation curves in figure 1,b is quite obvious.

4.1.3 Differences between ground and sample

Laboratory test samples seldom include significant discontinuities and/or are often disturbed by their excavation from the ground. Also it is very difficult to obtain the same test conditions in the laboratory as they were in the field, with regard to such factors as tectonic stresses, waterflow, etc.
4.2 ELASTIC MODULI

Different expressions are used in the literature for the same elastic moduli, so that it is necessary to define the different elastic moduli.

1. the static E modulus $E_{\text{stat}} = \text{Young's modulus as defined by Hooke's Law:}$

$$E = \frac{\sigma_{xx}}{\varepsilon_{xx}}$$

$\sigma_{xx}, \sigma_{yy}, \sigma_{zz}$ are constant, $\varepsilon_{yy}$ and $\varepsilon_{zz}$ are constant.

The strain $\varepsilon$ is linearly related to the stress $\sigma$, and the medium is expected to behave elastically.

2. the deformation E modulus $E_{\text{def}} = E_{\text{stat}}$ but obtained by deformation of the ground, which is not necessarily elastic.

$$E_{\text{def}} = \frac{\sigma_{xx}}{\varepsilon_{xx}}$$

$\varepsilon_{yy}$ and $\varepsilon_{zz}$ are zero.

3. the dynamic E modulus $E_{\text{dyn}} = E_{\text{stat}}$ but obtained by recycled loading tests.

$E_{\text{stat}}$, $E_{\text{def}}$ and $E_{\text{dyn}}$ are obtained from in-situ plate-bearing, flat jack, or radial jacking tests or from laboratory compression tests.

4. the seismic E modulus $E_{\text{seis}} = \text{The so-called "dynamic" elastic modulus calculated out of seismic waves.}$

4.2.1 Differences between $E_{\text{stat}}$ and $E_{\text{seis}}$

The static modulus $E$, calculated out of a plate-bearing or out of laboratory compression tests, is based upon the linear deformation under load:

$$E = \frac{\sigma}{\varepsilon}$$

$\sigma = \text{stress}$

$\varepsilon = \text{strain}$

Because a ground-mass is not a pure elastic medium there are some major differences between $E_{\text{stat}}$ and $E_{\text{seis}}$.

---

These differences and the reasons for them have been formulated by Arnost Dvorak (reference 13). Contributory factors are:

1. **Time effect**
The time through which seismic stresses act is up to 0.01 s, with relaxation immediately following. During a static test the stresses can act for up to several hours and continuous deformation occurs. If the rock-mass behaves like a Kelvin-Voight body, then the deformation under stress is time-dependent (see section 3.3).

2. **Intensity of stress**
Static tests apply stresses up to 100 - 1000 MN/m². Seismic measurements apply stresses up to 0.1 - 1 MN/m². The deformation under seismic stresses is an 2 or 3 order of magnitude lower than the deformation under static stresses. If the stress-strain behaviour of the material is not completely linear, the differences in stress ranges will cause a difference in the E moduli values.

3. **Thermic effect**
Because the seismic stresses act for a small time, the developed heat is not compensated; the process has an adiabatic character. The long time period over which static stresses act allows the heat to flow away, which gives this process an isothermic character. It is obvious that this difference will have, at least some, influence on the energy of acoustic waves.

4. **Water content**
The static E modulus for dry ground is generally greater than the static E modulus for moist ground. For the seismic E modulus the reverse has been observed even in ground under a pressure of up to 40 MN/m².

5. **Joints and their filling**
As the void ratio of the ground increases both the static and seismic E moduli tend to diminish. The difference between the static and seismic moduli is influenced more by open joints and their plastic filling, because elastic waves spread over such joints without decrease of velocity. Deformation from static stresses are substantially greater even if the initial first cycle deformation is decreased by repeating cycles of loading.
6. Volume influence

As already described in section 4.1.3, samples are mostly different from the ground out of which they have been excavated. These samples are of small volume in comparison with the volume of ground influencing acoustic waves and also do not contain the joints, fractures and other discontinuities, which influence values of the seismic E modulus.
SECTION V
EMPIRICAL RELATIONSHIPS

From measuring the velocities and attenuation of acoustic waves it is possible to obtain an (apparent-) elastic tensor, but because interpretation of this tensor is difficult and uncertain, attempts have been made to find empirical relations between specific ground-mass parameters and parameters of the acoustic compression and/or shearwaves.

5.1 VELOCITY OF ACOUSTIC WAVES

The received signal is expected to have travelled along the fastest raypath, this does not need to be the shortest raypath. If a wave encounters an abrupt change in elastic and/or density parameters the wave will be reflected, refracted and/or diffracted.

5.1.1 Ground quality determined by velocities of acoustic waves

Rippability charts.

In engineering-geology one of the important ground parameters is the force necessary to break the ground by excavation machines. This force is dependent upon the tensile strength of the ground-mass. Tensile strength of a ground-mass is related to the degree of jointing and the degree of weathering. The acoustic wave arrival-time also depends on these ground-mass features. If the investigation is done on the surface of a layered ground-mass (see figure 2), the effective stresses increase with depth. If the ground-mass is also weathered the degree of weathering normally decreases with depth. Both effects cause an increase of acoustic velocity with depth. Only in a regular ground-mass structure can this velocity-depth relationship be calculated with sufficient certainty. In most ground-masses the raypath of the acoustic wave is uncertain and thus the velocity calculated from arrival times should be considered as an apparent velocity.
When the ground material and structure are known the apparent velocity gives a rough idea of the ground-mass quality. These relations are expressed in the so-called rippability charts, normally based on compressional wave velocities, because these are easy to measure.

It is apparent that the velocity-quality relations are not unique, from the fact that different authors obtain from different investigations different charts for the same type of ground. Examples of two charts are figure 3 and 4.
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**LÉGENDE:**
- Terrassable aux engins à lame (décapapeuse, houeuse, chargeuse, pelleuse)
- Défendable (limites valables pour défonceuse dont portée par un tracteur de puissance > 230 cv dont l'effort maxi de traction est supérieur à 35000 Kg)
- Marginal (toujours défendable, toujours non défendable)
- Dislocation à l'explosif nécessaire

Figure 3: Example of a rippability chart

One of the main drawbacks of the charts is that the ground material, the geological structure and the power/weight ratio of the available excavation machines must be known beforehand (at least roughly) to obtain good results.
### 41-B (524 H.P.) WITH SINGLE & MULTI SHANK RIPPER

**Ripping Capability As Related To Seismic Wave Velocities**

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<tr>
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<th>2.1</th>
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**VELOCITY**

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**Ripping In This Seismic Range Is Practical Only If Material Has Favorable Ripping Characteristics:**

**Favorable Ripping Characteristics:**

- Fractures, faults & lines of cleavage;
- Brittleness;
- Stratification or lamination;
- Weathering and decomposition of cementing material;
- Grain size, and moisture permeation.

Figure 4: Example of a rippability chart

5.1.2 Ground quality out of field and laboratory velocities

Estimating of ground quality is also done by relating the laboratory (ultrasonic-) velocity to the seismic velocity measured in the field. Deere et al. (reference 11) have shown that:

\[
\left( \frac{V(\text{field})}{V(\text{lab})} \right)^2 \times 100 \% \approx \text{R.Q.D.}
\]

\(V(\text{field})\) = seismic compression velocity
\(V(\text{lab})\) = laboratory (ultrasonic-) compression velocity
R.Q.D.(1)= Rock Quality Designation

Because the R.Q.D. is related to the degree of jointing, the equation can be used to estimate the rippability.

5.1.3 Groundwatertable

Another important factor in engineering-geology is the depth of the groundwatertable, particularly in connection with rippability-charts, because the degree of water saturation influences the wave velocity (see section 3.2.2). When, in refraction seismic surveys, the time difference between the first two opposite maxima is measured, (figure 5) it is likely that an abrupt change of this time difference is due to changes in water saturation degree (reference 5). Although this effect is not based on velocity measurements, but rather on frequencies, it is named here because it is mainly used in connection with rippability charts.

5.1.4 Depth of open joints

In rippability-charts no differentiation is made of the different features which influence the ground-mass velocity. It would be an improvement to look at the received signal in detail to see if any evidence of a particular feature can be detected.

(1) R.Q.D. is a rock quality designation used in the description of rock cores and is defined as the percentage of recovered intact rock with lengths more than 0.10 m, compared to the total core length drilled.
The large values of \( t \) correlate with a water saturated kaolinite layer.

Example of an seismic investigation in Kerroch, South Bretagne, France.

Figure 5: Determining groundwaterdepth

Time-distance relations look often as if there are two layers; a low velocity top layer and a faster second layer, which can not be examined by the increase of ground pressure, decrease of weathering or by the geological structure. Merkler et al. (reference 23) stated that the joints will become (more) closed on a certain depth, so that an apparent two layer structure is generated (figure 6).
A low velocity ($V_1$) top layer with open joints and a second faster ($V_2$) layer without open joints.

Figure 6: Influence of open joints on seismic velocity

The depths of the open joints become now, according to the refraction laws (see figure 7):

In situation I:

$$h = \frac{d}{2} \left( \frac{t}{t_{(\text{min})}} - 1 \right)$$

In situation II:

$$h = \frac{d}{2} \sqrt{\frac{t}{t_{(\text{min})}} - 1}$$

$h$ = joint depth
$d$ = distance between source and geophone
$t$ = measured traveltime
$t_{(\text{min})}$ = minimal travel time measured in the 'area'; thus the travel time for ground with the best quality
Figure 7: Seismic raypaths through ground-mass with open joints.

This is satisfactory in a first approximation, but when the joint density is low or the joint depth is large, the up- and downgoing wave will travel for most part through intact ground and thus will travel with the same or nearly the same velocity as in the second (without open joints) layer. From wave theory, confirmed by tests on concrete blocks, (reference 34) it is proved that the received signal will then consist of a series of hyperbolae (figure 8) and that the depth of the open joints becomes:

\[ T = \frac{\sqrt{a^2 + H^2}}{V_2} + \frac{3a}{2V_2} \]

- \( a \) = the joint separation
- \( H \) = depth of the joints
- \( V_2 \) = the ground velocity including all the ground features, except open joints

5.1.5 Statistical evaluation of velocities

Most of the results described above have a very restricted applicability, because the degree of jointing and the openness of the joints can vary widely over relatively small areas, and even within one seismic line. It becomes then particularly difficult to differentiate between effects caused by jointing and by other ground-mass properties. For this reason attempts have been made to modify the velocity functions by statistical methods. The idea is that if, in a particular area, the seismic velocity has a particular variation due to variation in the degree of jointing and/or in the openness of the joints, both these variations should have a relation with each other and with the ground quality.
5.1.5.1 Kluftigkeitsfactor (broken-mass factor)

One of the quality factors from a statistical approach is the so-called "kluftigkeitsfactor" (K). This kluftigkeitsfactor is defined by different authors in a slightly different way.

After Keller:

\[
K = \frac{V(\text{min})}{V(\text{max}) - V(\text{eff})} \times \frac{V(\text{eff})}{V(\text{max}) - V(\text{min})}
\]

Figure 8: Velocity determination in ground-mass with open joints.
After Iliev:

\[
\frac{V(\text{max}) - V(\text{eff})}{V(\text{max})}
\]

\( V(\text{eff}) = \text{measured velocity} \)
\( V(\text{max}) = \text{highest velocity measured of the same ground} \)
\( V(\text{min}) = \text{lowest, lowest, lowest, lowest, lowest, lowest} \)

Merkler (reference 23) has suggested that both functions could be combined to (2):

\[
\frac{V(\text{max}) - V(\text{eff})}{V(\text{max}) - V(\text{min})}
\]

The problem of variation in the degree of jointing and in the openness of the joints occurs also when comparing laboratory results with field measurements. Apart from the volume aspect and the difference in the test wave frequencies, as described in section 4.1, the laboratory sample has seldom the same relative jointing degree and openness, because samples are mostly taken from intact ground-masses or from bore-cores. In both cases the sample will contain relatively less joints than the ground-mass, and thus the sample is of a better quality and has a higher acoustic velocity than the ground out of which it was excavated.

5.1.5.2 Heterogeneity-Scale factor

Statistical relations between laboratory tests and field tests are mostly called heteroginity or scale effect relations (reference 21).

The index of heterogeneity:

\[
V = \frac{\left( \frac{V_{\text{mod}} - V_{i \text{ mod}}}{V_{\text{mod}} - V_{o \text{ mod}}} \right) (\log L_{0 i} f_i^2 - \log L_{i o} f_o^2)}{\log V_{i \text{ mod}} - \log V_{o \text{ mod}}}
\]

(2) note the simularity between this equation and that for Relative Density of sands.
V = the ground volume controlling the effective ranges of oscillation at the velocity $V_{i \text{mod}}$ with frequency $f$, measured at base $L$

$V_{i \text{mod}}$ = modal most probable values of velocities at different scales of studies

0 : $V(i \text{mod}) = V(i \text{mod minimal})$

∞ : $V(i \text{mod}) = V(i \text{mod maximal})$

1 : current values of $V$, $f$ and $L$

Openness of joints is highly influenced by the ground pressure; under a higher pressure the joints will close, what gives a higher velocity.

It is easier to do tests under different ground pressures than to do a series of tests on samples with different joint degrees (reference 17).

An example of a correlation function between acoustic velocity and pressure is:

$$\sigma_{\text{max}} - \sigma = \frac{V_{p \text{max}}^2 - V_p^2}{\sigma_{\text{max}}} = \frac{V_{p \text{max}}^2}{V_p^2}$$

$\sigma_{\text{max}}$ = the compression strength

$V_p$ = compressional wave velocity

Golodkowskaya (reference 17) claims that this equation holds for all petrographic types of rocks. Figure 9 shows the compressive strength and the compressional wave velocity against the investigated volume of ground:

$$W = 1 \lambda^2$$

$\lambda$ = wavelength
tholeiitic basalt bodies in undisturbed zones

porphyritic basalt bodies with low tectonic jointing

porphyritic basalt bodies with high tectonic jointing

(After Golodkovskaya)

Figure 9: Correlation between acoustic velocity and compression strength.
5.1.6 Joint densities and directions out of velocity analyses

Although there are a great number of publications which deal with solid rock properties and seismic waves, the number of publications which refer to joint directions and especially joint densities is quite small. The main article on this subject is: "Estimation of crack parameters from observations of P-wave velocity anisotropy" by Crampin et. al. (reference 9). They have, based on previous articles by Garbin and Knopoff (references 15 and 16), developed a function for the estimation of low concentrations of thin, penny-shaped orientated cracks, whose diameter is small compared to the seismic wavelength and where the overall cracked volume is large compared to the wavelength.

Garbin and Knopoff found that the variations of the P-wave velocities due to dry cracks were:

\[
\left( \frac{1}{\lambda_1 + 2\mu_1} \right) = \left( \frac{1}{\lambda + 2\mu} \right) \left( 1 + \frac{8}{3V_0} \sum_{i=1}^{N} \left( a_i^3 \frac{\mu \sin^2 \theta_i \cos^2 \theta_i}{3\lambda + 4\mu} \right) \right)
\]

and due to liquid-filled cracks:

\[
\left( \frac{1}{\lambda_1 + 2\mu_1} \right) = \left( \frac{1}{\lambda + 2\mu} \right) \left( 1 + \frac{64}{3V_0} \sum_{i=1}^{N} \left( a_i^3 \frac{\mu \sin^2 \theta_i \cos^2 \theta_i}{3\lambda + 4\mu} \right) \right)
\]

\(\lambda, \mu\) = the apparent Lame constants
\(\lambda, \mu\) = the Lame constants of the ground
\(a_i\) = the radius of crack \(i\)
\(N\) = the amount of joints/meter
\(\Theta_i\) = the angle between crack \(i\) and the raypath
\(V_0\) = volume of the ground including joints

For randomly orientated cracks, the formulas (5.1) and (5.2) become according to Garbin and Knopoff:
\[
\begin{align*}
\frac{1}{\lambda_1 + 2\mu_1} &= \frac{1}{\lambda + 2\mu} \left( 1 + \frac{8}{3} \frac{\lambda}{\nu} \frac{\mu}{\nu} \right) \\
&+ \\n\frac{1}{\lambda_1 + 2\mu_1} &= \frac{1}{\lambda + 2\mu} \left[ (\frac{1}{3}\lambda + \frac{1}{5}\mu) \frac{1}{\nu} + \frac{\lambda^2}{2\mu(\lambda + \mu)} \right] \\
&+ \\n\frac{1}{\lambda_1 + 2\mu_1} &= \frac{1}{\lambda + 2\mu} \left( 1 + \frac{128}{45} \frac{\lambda}{\nu} \frac{\mu}{\nu} \right) \\
&+ \\
\end{align*}
\]

\[V_p = \sqrt{\frac{\lambda + 2\mu}{\mu}} \]  

\(V_p = \text{P-wave velocity}\)  
\(\mu = \text{ground-mass density}\)

Crampin a.o. have combined (5.1), (5.2) and (5.5) with 
\(\lambda = \mu\) for one joint system of equally orientated joints:

\[
\frac{1}{\nu_{PD}} = \frac{1}{\nu_{PD}} \left( 1 + \frac{8}{3} \frac{\lambda}{\nu} \left( \frac{1 + 2\cos^2 \theta}{\sin^2 \theta \cos^2 \theta} \right) \right) \]  

\[
\frac{1}{\nu_{PS}} = \frac{1}{\nu_{PS}} \left( 1 + \frac{64}{3} \frac{\lambda}{\nu} \left( \frac{1}{\sin^2 \theta \cos^2 \theta} \right) \right) \]  

with \(\xi = \frac{\lambda}{\nu}\) Crampin defines:

\[
R_D = \frac{1}{1 + \frac{8}{3} \xi \left( \frac{1}{\sin^2 \theta \cos^2 \theta} \right) \left( \frac{1 + 2\cos^2 \theta}{\sin^2 \theta \cos^2 \theta} \right)} \]  

\[
R_S = \frac{1}{1 + \frac{64}{3} \xi \left( \frac{1}{7} \sin^2 \theta \cos^2 \theta \right)} \]  

- 38 -
which gives:

\[
\frac{1}{V_{pD}} = \frac{R_{D}^{1/2}}{V_{po}} = \frac{1}{V_{pS}} = \frac{R_{S}^{1/2}}{V_{po}}
\]

with \( p \) as the degree of saturation; \( p \cdot \epsilon \) is the saturated joint density and \((1-p) \cdot \epsilon\) is the dry joint density.

For two joint systems the velocity variation function becomes now:

\[
V_{p} = \frac{V_{po}}{\sqrt{R_{D1} \cdot R_{D2}} + \sqrt{R_{S1} \cdot R_{S2}}} (5.10)
\]

Bamford and Nunn (reference 3) performed small-scale refraction experiments with a weight-drop source over shallow Carboniferous limestone in the Hutton roof locality of northwest England.

Crampin et al. have used function (5.10) to estimate the joint directions and densities using the velocities from these measurements. Figure 10 shows the original data, the first five-term Fourier series approximation and the estimation of function (5.10) with the original data.
Observed $P$-wave velocity versus azimuth and modeling at HPF. Crosses = scattergram data (after paper 3); dashed lines = variation of the first five-term Fourier series approximation; solid line = variation of biplanar cracks with parameters in Table 2 for $V_p = 6.0$ km/sec. The inset shows the distribution of the residuals about the biplanar fit (solid line), where the residuals are in km/sec for direct comparison with the figure.

(After Grampin et. al.)

Data of table 2 are: crack densities: $\xi_1 = 0.26$
$\xi_2 = 0.17$

Azimuth of crack normals: $\Theta_1 = -51.5^\circ$
$\Theta_2 = 40.7^\circ$

Percentage of saturation: $p = 48.1\%$

Fourier series
rms: $= 3.072 \text{ (km/s)}^2$

Figure 10: Velocity anisotropy due to crack orientation
5.2 WAVEFORM

The received waveform will differ from the original signal due to two effects:

1. Multiple reflections (figure 11)

- Signal passing through a sequence of thin reflectors

3. Seismograms may be substantially independent of the sequence being examined. The mutual coherence of the individual waves may be assessed. The overall coherence of the individual waves may be assessed. The overall coherence of the individual waves may be assessed.

Figure 11: Signal modification through multiple reflections

Due to changes in elasticity short-path multiples will occur. This example is of long-time reflection signal, but in short-time engineering-geology
surveys the joints can act as thin reflectors. The stronger arrivals will have the same sign as the primary waveforms since successive impedance contrasts lowers the signal frequency as time increases.

2. The signal initiated by the source will consist of a series of waves with different energy and frequency. Because of the dependence of the wave behaviour on the frequency, waves with a higher frequency will attenuate faster than waves with a lower frequency. This causes the frequency and amplitude of the received signal to decrease with distance.

The amplitude of an extending spherical acoustic wave will decrease with distance and there will be an additional decrease due to losses of energy as described in section 3. The amplitude will also decrease due to reflection, refraction and diffraction. This energy is not lost, but will due to changes in direction, not arrive at the geophones. Most publications do not differentiate between the losses due to absorption and the losses due to reflection, etc.

The absorption of energy is expected by most authors to depend on frequency. The same conclusion may be drawn from the study of a visco-elastic model. (section 3.3.1) Attenuation can be distance dependent:

\[ e^{-a x} \]

or time dependent:

\[ e^{-\gamma t} \]

\[ t = \text{traveltime} \]
\[ x = \text{distance} \]
\[ a = \text{spacial attenuation} \]
\[ \gamma = \text{temporal attenuation} \]

with \( a = \frac{\gamma}{c} \) and \( a = \frac{\omega}{2 c Q} \)
\( \omega = \text{angular frequency} \)
\( c = \text{phase velocity} \)
\( \frac{1}{Q} = \text{specific attenuation factor} \)

The published data give several relationships between absorption mechanism, attenuation and frequency. Conclusions as to these relationships are usually based on broad extrapolations from seismic field and laboratory data derived over individually limited ranges of frequency.
Qualitative relations derived from the literature are:

1. Q values for ground are usually an order of magnitude below those for many other materials, and for a mineral aggregate Q can often be ten times lower than the Q for the single crystal or grain.

\[ Q \text{ (calcite)} = 1900 \text{ (Peselnick and Zietz, 1959)} \]
\[ Q \text{ (limestone)} = 200 \]

2. Laboratory experiments on samples taken from homogeneous ground give \( 1/Q \) independent of frequency.

3. Seismograms also show \( 1/Q \) to be substantially independent of frequency at frequencies below 1 Hz, but with a depth sensitivity and thus a pressure sensitivity (as described in section 3.2.4). For frequencies from \( 20 - 100 \) Hz \( 1/Q \) increase with increasing frequency.

4. In liquids \( 1/Q \) is proportional to frequency.

These qualitative relations are mostly based upon laboratory and/or seismic results from deep and thus more homogeneous ground.

The above conclusions are partly based upon conclusions published by Attewell (reference 1).

Attewell shows as well some quantitative relations between attenuation and frequency (figure 12). A least squares analysis from the data out of figure 12 gives:

\[ y = 5.835 \times 10^{-3} \times f^{1.005} \text{ dB/s} \]
\[ a = 5.068 \times 10^{-7} \times f^{0.911} \text{ dB/cm} \]

for \( 10^{-3} < f < 1 \) and

\[ a = 1.012 \times 10^{-5} \times f^{0.911} \text{ dB/cm} \]

for \( 1 < f < 10^8 \)

The conclusions that may be drawn from the discussion of the wave form relations are:

1. For frequencies between \( 10^{-3} \) and 1 Hz (Rayleigh-waves), the attenuation is directly proportional to frequency; the constant of proportionality is \( 5.10^{-7} \text{(s.dB/cm)} \)

2. For frequencies between 1 and \( 10^8 \) Hz (compression waves), the attenuation is approximately direct proportional to frequency; the constant of proportionality is \( 1.10^{-5} \text{(s.dB/cm)} \)
Attenuation against frequency

Sedimentary rocks

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>P-waves</th>
<th>R-waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>$10^{-1}$</td>
<td>$10^{-2}$ dB/cm</td>
<td>$10^{-3}$ dB/s</td>
</tr>
<tr>
<td>$10^{-2}$</td>
<td>$2.1 \times 10^{-6}$ s dB/cm</td>
<td></td>
</tr>
<tr>
<td>$10^{-3}$</td>
<td>$4.7 \times 10^{-5}$</td>
<td></td>
</tr>
</tbody>
</table>

(After Attewell)

least-squares fit

95% confidence limits

Figure 12: Attenuation against frequency

3. for frequencies between $10^{-3}$ and $10^7$ Hz (Rayleigh and compression waves), the attenuation is directly proportional to frequency; the constant of proportionality is $2.1 \times 10^{-6}$ (s dB/cm)

4. and for frequencies between $10^{-3}$ and $10^7$ Hz (Rayleigh, compression, shear, and Love waves), the internal friction is independent of frequency with a mean value of $4.7 \times 10^{-5}$

Although these conclusions were formulated already in 1966 by Attewell there is, except for the last one, in the literature no evidence that these conclusions are wrong. The conclusion about the specific attenuation factor (= the internal friction) is not likely to be completely true (see section 3.3 and reference 35).
5.2.1 Influences of joints on acoustic wave amplitude

If reflection, refraction and diffraction are taken into account the signal received will be the result of a series of interfering waves. Theoretical solutions are not given in the literature, but some attempts have been made to find empirical relations.

When a harmonic compressional wave encounters an elastic discontinuity, a reflected compression and shearwave and a refracted compression and shearwave are generated. The energy relations are described by Knott (1899). The distribution of the energy over the waves depends on the angle of incidence of the original wave. Generally speaking an increase of the angle of incidence decreases the energy of the refracted waves and increases the energy of the reflected waves. If the elastic discontinuity consists of a series of orientated joints it is clear that the amplitude of the received signal will be related to the joint density and also to the angle between the raypath of the acoustic wave and the joints.

Field measurements of this effect are reported only from bore-hole logging. In figure 13 the correlation between the decrease of velocity and the increase of the attenuation with the highly fractured zones, is quite clear.

A more general conclusion by King (reference 20) was that shear-wave-amplitudes are more affected by shallow-dipping fractures, whereas compression-wave-amplitudes are reduced by fractures steeply dipping in relation to the axis of the borehole.

5.2.2 Rise-time relations

The attenuation factor can be defined in terms of the fractional loss of maximum stored energy per cycle:

\[ \frac{1}{Q} = \frac{\Delta J}{2\pi J(\text{max})} \]

\( J(\text{max}) = \) maximum stored energy
\( \Delta J = \) fractional loss of energy

For a dry ground \( Q \) is nearly independent of the frequency over a long range.
compressional velocity (km/s)

relative amplitude
first arrival

highly fractured and altered zones

acoustic log velocity

Distance from collar (m)

Figure 13: Relation between compression velocity, amplitude and fracturing

The attenuation coefficient is:

\[ a = \frac{\pi f}{Q V} \]

\( a \) = attenuation coefficient
\( f \) = frequency
\( V \) = wave velocity

\( Q \) is constant, so that the higher frequency components of a pulse are spatially attenuated more rapidly than the lower frequency components. This leads to a decrease of sharpness of the pulse and to a broadening of the pulse. Determination of the pulse broadening is done by measuring the rise-time (see figure 14).

McKenzie et al. (reference 22) did a series of cross-hole ultrasonic and seismic measurements (figure 15).
5.3 ELASTIC MODULI

Although the elastic moduli calculated out of the compression and shearwave velocities are not the real elastic moduli of the ground-mass (for the reasons described in section 4.2.1) many authors use these elastic moduli and have tried to compare them with the static or deformation moduli. A number of values of the seismic $E$ and of the static $E$ moduli are to be found in the literature, mostly based upon tests in one area. Figure 16 compares these values. As these figures clearly show, there is no reliable correlation between the seismic and the static or deformation $E$ moduli.

However, it should be noted that scale factors play an important part in the determination of the static and deformation moduli and at least some of the disposition of the points in figure 16 may come from this factor.

Stacey (reference 33) proposed that for preliminary design purposes the following relations could be used:

\[
\text{static } E = \frac{1}{4} \text{ seismic } E
\]
\[
\text{deformation } E = \frac{1}{8} \text{ seismic } E
\]
5.3.1 Relations between joint density and elastic moduli

T. Kazimierz et.al. (reference 19) have done seismic investigations on a site which was proposed as the foundation for a gravity dam. Besides the seismic tests a series of static compressional tests were done. Their method is based on a comparison of the static/deformation/seismic E moduli with a coefficient of fissuration, which is defined as:

$$c_f = \frac{1}{S} \sum_{i=1}^{n} a_i b_i + \sum_{i=1}^{n} a_i b_i$$

- $c_f$ = coefficient of fissuration
- $a_i$ = length of fissure
- $b_i$ = width
- $S$ = reference area

In figure 17 are shown the results from the tests plotted against the coefficient of fissuration for a series of meas-
As figure 17 shows the relation between different elastic moduli and the coefficient of fissuration is very vague, if indeed there is any relation at all.

If the coefficient of fissuration does not reflect the anisotropy of the joints and the elastic tensor direction, 

Seismic measurements cannot be used to estimate the influence on the measured elastic moduli values. It is clear that the relations can

turn out to be good.

5.3.2 "Fissure influence"

Figure 16: Static and deformation E against seismic E modulus.

As thin reflectors, which show multiple reflections (see section 5.3). The multiple reflections have the same 

sign as the primary wave, because successive impedance con-

trasts are in opposite directions, but have a small delay 
time in comparison with the primary signal. This lowers the frequency with an increase of the amount of joints.
Figure 17: Elastic moduli against fissuration coefficient

In Figure 17 are shown the results from the tests plotted against the coefficient of fissuration for a series of meas-

- 50 -
As figure 17 shows the relation between the different elastic moduli and the coefficient of fissuration is very vague, if indeed there is any relation at all. The coefficient of fissuration does not reckon with the angle between the joints and the elastic test direction. Because this angle has considerable influence on the measured elastic moduli values, it is clear that the relations can not be good.

5.3.2 "Petite sismique"

B. Schneider has developed the so-called method "petite sismique". (reference 32). The method is based upon the frequency \( f \) of a shearwave:

\[
f = \frac{V_S}{\lambda_S}
\]

\( V_S \) = shearwave velocity  
\( \lambda_S \) = shear wavelength

The shearwave frequency seems to have a linear correlation with the static E modulus. Figure 18 shows for different sites the mean value of the static E modulus against the mean frequency of the shearwave.

Schneider explained that fractures serve as filters selectively attenuating high frequency components of the propagating signal, this gives a relation between the static E modulus and the shearwave frequency.

In the author's opinion it is also possible that the joints act as thin reflectors, which cause multiple reflections (see section 5.2). The multiple reflections have the same sign as the primary wave, because successive impedance contrasts are in opposite directions, but have a small delay time in comparison with the primary signal. This lowers the frequency with an increase of the amount of joints.
Figure 18: Static E modulus against shearwave frequency

(after B. Schneider)
SECTION VI
CONCLUSIONS

It is astonishing that nearly none of the articles reviewed are concerned with anisotropy. In particular, the anisotropy of jointing, which is likely to be the most pronounced ground-mass feature, is seldom examined. This is particular strange for it is known that the direction, character and density of joints have an important influence on both laboratory tests and field measurements.

For further investigation it seems necessary that the anisotropy, especially of jointing, should be taken into account. This can be done in two ways:

1. The anisotropy is measured separately,
2. The measurements are done at random, whereby the number of measurements must be large enough to allow statistical evaluating of the data.

The articles which deal with evaluating the anisotropic behaviour of the ground-mass, are the articles published by Backus, Bamford, Crampin and Garbin and the primary article which deals with statistical evaluating of the data is the article by A. Golodkovskaya (reference 17). Both series of articles describe results which are, in comparison with those, not listed above, very good.

Also the method "petite sismique" has proved to be successful, but it is strange that after the measurements done by Schneider (reference 32) there have been hardly any attempts by other authors to use this method. One of the reasons for this may be that shearwaves are difficult to generate and difficult to measure.
PART II

JOINT DENSITIES AND DIRECTIONS DEDUCED FROM SEISMIC WAVES
The purpose of this investigation was to develop a fast method for measuring joint properties, and if possible, ground-mass properties, through (refraction-) seismic methods.

The fieldwork and part of the elaboration was done in cooperation with the Department of Geology of the University of Leeds, United Kingdom. In the neighborhood of Leeds there was the opportunity to investigate different types of ground-masses in a relatively small area.

The fieldwork was done in quarries which were selected on:

1. differences in rock type in the different quarries,
2. simple tectonic and sedimentary structures,
3. the rocks had to be preferably unweathered,
4. clearly defined joint-directions,
5. the investigated rocks had to have one or more free sides so that the geology (in fact only bedding-slope and tectonic features like shear-zones) could be investigated.

The disadvantage of doing the investigations in quarries was the impossibility, because of falling stones, to make a joint-scaling of the cliff-faces. This has been replaced by making stereo-photographs on which a relative joint density has been measured.
The seismic measurements were done with a 12-channel Nimbus enhancement seismograph with built-in metal-paper recorder from the University of Leeds. The geophones were arranged within 5-20 m long straight traverses. The impact point was placed 1 or 1.5 m (in the claypit 0.5 m) from the first geophone. In most cases it was necessary to have two impacts for a proper signal. After a good signal was recorded the geophone line was rotated through about 22.5°, while the impact point was kept on the same place. This was repeated at least 8 times, so that a "seismic fan" was recorded (figure 19).

A metal tube of about 2 m high in which a weight could drop was used as a seismic source. This configuration kept the impact energy from different drops broadly the same and gave a strong good P-wave signal.
The measurements were mostly done on equalized horizontal benches, which made a topographical correction unnecessary. As far as possible the investigations were done on rocks with bedding parallel to the bench-surface, so that slope-corrections were also unnecessary. To be certain that the measured velocities were from one layer, the investigations in most quarries were done on top of a 2-3 m thick layer, underlain by a softer layer (mostly shale); of a lower seismic velocity. This assure that arrivals came from the thick layer and were not refracted from deeper layers. Sometimes man-made overburden, consisting of rock fragments and clay, with a thickness between 0.2 and 0.5 m, was found on top of the bench. Samples were taken from the measured layer for laboratory tests and stereo photographs were taken from the quarry cliffs for an estimation of the relative joint densities.
SECTION IX
QUARRIES

9.1 NATIONAL COAL BOARD OPEN PIT MINE

The NCB-mine is a coal mine south-east of Leeds in the
neighbourhood of Newsamgreen, Dunstan Hills and Avenue Wood
(431 500 N; 436 500 E, Nat. Grid).

(figure 20, geological profile: figure 21, and photo: figure 22)

Figure 20: Seismic fan NCB-mine
### bench surface

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>fresh, thinly laminated, grey/black, moderately weak, shale/coal, extremely closely spaced joints. Disturbed by ripping, blasting and partly fill.</td>
</tr>
<tr>
<td>0.5</td>
<td>sample</td>
</tr>
<tr>
<td></td>
<td>fresh, medium bedded, grey, well graded, very strong, siltstone, medium blocky joints</td>
</tr>
<tr>
<td>2.2</td>
<td>fresh, thinly laminated, grey/black, moderately weak shale/coal, extremely closely spaced joints</td>
</tr>
</tbody>
</table>

**Figure 21: Geological profile NCB-mine**

The NCB-mine works the Carboniferous Coal Measures, which consist here out of an interlayering of shale/coal layers and siltstone layers. The main joint direction is 112° and a second joint system has the direction of 205°, both vertical. No form of weathering was visible and no water could be seen although it had rained in the days before the measurements.

The seismic measurements were done on top of a 1.7 m thick siltstone layer, underlain by a shale/coal layer of 0.5 m. (see figure 21). The lines 112° to 180° were seismically surveyed in both normal and reversed directions in order to calculate the bedding slope. In the mine the bedding was, as far as could be seen (near) horizontal.
The Black Hill quarry is situated 500 m north-east of the A660 road between Leeds and Otley (442 300 N; 427 000 E, Nat. Grid). (figure 23, geological profile: figure 24, and photo: figure 25)

In this quarry is excavated Carboniferous Millstone Grit. The rocks consist out of 1 to 3 m thick layers of weathered, poorly graded, coarse to very coarse grained sandstone beds, interlayered with shale/siltstone beds. The main jointing is 000° and a second jointing has the direction of 075°, both vertical.

There was standing water on the bench where the measurements were done and also there were seepages directly above the shale layer, which indicate that the rocks were saturated.

The measurements were taken on top of a 2.5 m thick sandstone layer, underlain by a 1.6 m thick shale layer, under which were the sandstone 1s found again (figure 24).
There was no or nearly no man-disturbed layer on top of the sandstone layer. Because the layers were horizontal in all directions over a long distance, reversed shooting was not done.
moderately weathered, thick to very thick bedded, grey/yellow, medium to very coarse (gravelly) grained, poorly graded, moderately strong, sandstone, large to very large blocky joints

completely weathered, thinly laminated, yellow/grey/brown, (very-) weak, shale, extremely closely spaced joints

Figure 24: Geological profile Black Hill quarry

9.3 GREEHOW HILL QUARRY

This is a deserted quarry north of Leeds near Grimwith (463 800 N; 411 020 E, Nat. Grid). (figure 26 and photo: figure 27)

This quarry has been made to excavate the Carboniferous calcium-carbonate limestones.

The rocks consist of slightly weathered limestone with fluorite and galenite in some joints and shearzones.

The bedding-dip direction is $160^\circ$ with a dip of $25^\circ$. There are two joint systems with orientations 015/85 (main system) and 292/75.

The quarry bottom, on which the measurements were done, was covered with clay overburden. Water stood on the clay overburden, so it is likely that the rocks and joints were saturated with water.

In this quarry it was not possible to examine the measured rocks under the bench, but the rocks and joint properties are so regular, that this is not thought to be a problem (figure 27).
The description of the rocks above the bench is:

slightly weathered, thick bedded, light to dark grey, fine to medium grained, well graded, strong, limestone (wackestone), medium to large blocky joints.

It is assumed that similar strata are present under the bench for such rocks are to be found throughout the general area.
9.4 MAGNESIUM LIMESTONE QUARRY

The Magnesium Limestone quarry is situated beside the Old London road, South of Stutton, near Tadcaster along the A64 Leeds-York (440 600 N; 447 250 E, Nat. Grid). (figure 28, geological profile: figure 29, and photo: figure 30)

Perman Magnesium Limestone is excavated in the quarry. The rocks consist of slightly weathered, sometimes very porous (with visible solution voids), closely fractured, soft Magnesium Limestone. There are two main joint systems with directions 153° and 063°, both about vertical. No water was visible.
As can be seen on the photo, this quarry does not satisfy the requirements described in section 7. The photo shows clearly that just below the test site there are sedimentary and tectonic structures. This test site was chosen to test the working method in a more real situation.
The claypit is situated near Elny, north of York. (466 500 N; 451 230 E, Nat. Grid). (figure 31, geological profile: figure 32, and photo: 33)

The clay is a closely fissured firm glacial clay with occasionally more silty layers. The fissures have a length up to 0.2 - 0.5 m, and it is likely that they are orientated randomly.

On the surface no evidence of water was seen. From the seismic measurements appears that the groundwater table was at a depth of 1.3 m, and that the bedrock was at a depth of 14 m.

The only irregularity in the homogeneity of the clay-mass was a 0.6 m thick horizontal band with organic silt layers at 3 m depth (see figure 32).

Two different seismic measurements have been made:

1. fan-shooting,
0 m
surface

0.6
residual soil
moderately weathered

1.3
slightly weathered, thin to medium bedded, yellow/light brown, fine grained, weak to moderately strong, very porous, magnesium limestone, closely to medium spaced joints

5.4

Figure 29: Geological profile Magnesium Limestone quarry

2. a long distance line (60 m), normal and reversed, to find the bedrock depth.
9.5 CLAYPIT

The claypit is situated near Elby, north of York. (466 502 N, 45 230 E, Wat. Grid). (figure 31, geological profile: figure 32, and photo: 33)

The clay is a claysilted firm glacial clay with occasionally more silty layers. The fissures have a length up to 0.2 - 0.5 m, and it is likely that they are oriented randomly. On the surface no evidence of water was seen. From the seismic measurements appears that the groundwateertable was at a depth of 1.3 m, and that the bedrock was at a depth of 14 m.

The only irregularity in the homogeneity of the clay-mass was a 0.6 m thick horizontal band with organic silt layers at 3 m depth (see figure 33).

Two different seismic measurements have been made:

1. fan-shooting.

Figure 30: Magnesium Limestone quarry
Figure 31: Situation outline claypit
10.1 THEORY

As described in the literature review, Crampin et al. (reference 9) have developed a method to estimate joint directions, joint densities and the degree of saturation from the velocity anisotropy of a seismic fan. The formula Crampin has used is based upon the theoretical formulas described by Garbin & Knopoff (references 15, 16). They developed these formulas to explain the velocity anisotropy in upper mantle velocities. They assume an isotropic rock with an orientated series of thin, penny-shaped cracks, whose diameter is small compared to the seismic wavelength and where the overall cracked volume is large compared to the wavelength. They have developed formulae for dry and for saturated cracks. Crampin stated that for a combination of two or more series of orientated cracks, the harmonic mean of each of the dif-

---

Figure 32: Geological profile claypit
For a rock with two orientated series of cracks, where each can be partly saturated the formula becomes:

\[
V_p = \frac{V_{p0}}{(1 - p) + \frac{P}{\sqrt{R_{D1} \cdot R_{D2}}}} + \frac{P}{\sqrt{R_{S1} \cdot R_{S2}}}
\]  

(10.1)

Figure 33: Claypit

different velocity anisotropies gives the total velocity anisotropy.
with:

\[ R_{Di} = \frac{1}{1 + \frac{8}{3} \xi_i \left( \frac{8}{7} \sin^2 \theta_i \cos^2 \theta_i + \frac{(1 + 2 \cos^2 \theta_i)^2}{4} \right)} \]

\[ R_{Si} = \frac{1}{1 + \frac{64}{3} \xi_i \left( \frac{1}{7} \sin^2 \theta_i \cos^2 \theta_i \right)} \]

\[ \xi_i = \text{joint density} \]

\[ \theta_i = \text{angle between joint normal and raypath} \]

\[ p = \text{degree of saturation} \]

\[ V_p = \text{measured velocity} \]

\[ V_{po} = \text{intact rock velocity} \]

It is as well possible to evaluate a velocity anisotropy function for more than one series of joints and/or for a combination of dry and saturated joints directly from the velocity anisotropy functions by Garbin and Knopoff.

The Garbin and Knopoff function, is for dry cracks:

\[ \left( \frac{1}{\lambda + 2\mu} \right) = \left( \frac{1}{\lambda + 2\mu} \right) \left( \frac{1}{\lambda + 2\mu} \right) \left( 1 + \frac{8}{3V_o} \sum_{i=1}^{N} \frac{8y \sin^2 \theta_i \cos^2 \theta_i (\lambda + 2y \cos^2 \theta_i)^2}{3 \lambda + 4y \mu (\lambda + \mu)} \right) \]  
(10.2)

for saturated cracks:

\[ \left( \frac{1}{\lambda + 2\mu} \right) = \left( \frac{1}{\lambda + 2\mu} \right) \left( 1 + \frac{64}{3V_o} \sum_{i=1}^{N} \frac{\mu \sin^2 \theta_i \cos^2 \theta_i}{3 \lambda + 4 \mu} \right) \]  
(10.3)

\[ \lambda, \mu = \text{the apparent Lame constants} \]

\[ a_i = \text{the Lame constants of the ground} \]

\[ N = \text{the radius of crack} \]

\[ \theta_i = \text{the amount of cracks per m} \]

\[ \theta_i = \text{the angle between the normal of crack} \]

\[ \text{and the raypath of the seismic wave} \]

If

\[ DR_i = \frac{8y \sin^2 \theta_i \cos^2 \theta_i}{3 \lambda + 4 \mu} + \frac{(\lambda + 2y \cos^2 \theta_i)^2}{2y(\lambda + \mu)} \]  
(10.4)
and

\[ S_i = \frac{8 \mu \sin^2 \theta_i \cos^2 \theta_i}{3 \lambda + 4 \mu} \]  \hspace{1cm} (10.5)

If crack \( i \) is saturated: \( Q_i = 1 \) and if dry: \( Q_i = 0 \)

The function for a combination of dry and saturated cracks becomes:

\[ \frac{1}{v_p^2} = \frac{1}{v_{po}^2} \left( 1 + \frac{8}{3} \sum_{i=1}^{N} a_i^2 (DR_i (1 - Q_i) + S_i Q_i) \right) \]  \hspace{1cm} (10.6)

and for two different series of cracks:

\[ \frac{1}{v_p^2} = \frac{1}{v_{po}^2} \left( 1 + \frac{8}{3} \left( a_1^2 DR_1 \sum_{i=1}^{N} (1 - Q_i) + a_1^2 S_1 \sum_{i=1}^{N} Q_i \right) + \right. \]
\[ \left. \left( a_2^2 DR_2 \sum_{i=1}^{N} (1 - Q_i) + a_2^2 S_2 \sum_{i=1}^{N} Q_i \right) \right) \]  \hspace{1cm} (10.7)

- \( N_1, N_2 \) = the amount of cracks/meter of the different crack series
- \( a_1, a_2 \) = the radius of the cracks of the different crack series

because:

\[ \sum_{i=1}^{N} (1 - Q_i) = (1 - p)N \] \hspace{1cm} and \hspace{1cm} \[ \sum_{i=1}^{N} Q_i = pN \]
(10.7) becomes:

\[
\frac{1}{V_p^2} = \frac{1}{V_0^2} \left( 1 + \frac{8}{3} \left( \frac{a_1^3 DR_1 (1-p)N_1}{a_1^3 S_1 pN_1} + \frac{a_2^3 DR_2 (1-p)N_2}{a_2^3 S_2 pN_2} \right) \right)
\]

(10.8)

and if \( \lambda = \psi \) and \( \xi = \frac{Na^3}{V_0} \) (\( \xi \) = crack density):

\[
\frac{1}{V_p^2} = \frac{1}{V_0^2} \left( 1 + \frac{8}{3} (1-p) \xi_1 \left( \frac{8}{7} \sin^2 \psi \cos^2 \psi + \frac{1}{4} (1 + 2 \cos^2 \psi)^2 \right) \right) + \frac{8}{3} \xi_2 \left( \frac{8}{7} \sin^2 \psi \cos^2 \psi + \frac{8}{3} (1-p) \xi_2 \left( \frac{8}{7} \sin^2 \psi \cos^2 \psi \right) \right)
\]

(10.9)

Formulae (10.1) and (10.9) are used to estimate the joint parameters out of the fan-velocities from the different quarries by means of a Marquardt algorithm (IMSL computer library, reference 18).

If the dip of the layers is not parallel to the bench, the slope adds to the velocities a term:

\( A \cos \phi + B \sin \phi \) (\( \phi \) is the fan-angle)

This gives for a normal and reversed seismic line:

\[
\frac{1}{V(\text{nor})} = \frac{1}{V(\text{cor})} + A \cos \phi + B \sin \phi
\]

(10.10)

\[
\frac{1}{V(\text{rev})} = \frac{1}{V(\text{cor})} + A \cos(\phi + 180^\circ) + B \sin(\phi + 180^\circ)
\]
Out of which A and B can be calculated if the normal and reversed velocities are known. If A and B are known then the measured fan-velocities can be corrected to become the real refractor velocities.
10.2 VELOCITY ANISOTROPY IN THE QUARRIES

10.2.1 NCB-mine

The normal and reversed velocities indicate that the refractor plane is slightly dipping (appendix A.1). Applying formula (10.10) gives:

\[ A = -2.82 \times 10^{-2} \text{ s/m} \]
\[ B = -3.08 \times 10^{-2} \text{ s/m} \]

Because the direction of maximum dip equals:

\[ \arctan \left( \frac{B}{A} \right) \]

the dip direction of the refractor plane is 228°. The corrected velocities are listed in table 1.

Table 1.

<table>
<thead>
<tr>
<th>refractor depth below shot point</th>
<th>refractor depth below shot point</th>
</tr>
</thead>
<tbody>
<tr>
<td>degrees</td>
<td>V(nor)</td>
</tr>
<tr>
<td>---------</td>
<td>-------</td>
</tr>
<tr>
<td>112</td>
<td>1.69</td>
</tr>
<tr>
<td>135</td>
<td>1.50</td>
</tr>
<tr>
<td>157</td>
<td>1.37</td>
</tr>
<tr>
<td>180</td>
<td>1.06</td>
</tr>
<tr>
<td>202</td>
<td>1.42</td>
</tr>
<tr>
<td>225</td>
<td>1.03</td>
</tr>
<tr>
<td>247</td>
<td>1.23</td>
</tr>
<tr>
<td>270</td>
<td>1.47</td>
</tr>
</tbody>
</table>

[10.10]
The depths of the refractor are calculated out of the intercept times of appendix A.1 with the following formula:

\[ d = \frac{T_i V_1 V_2}{2 \sqrt{V_1^2 - V_2^2}} \]  

(10.11)

\[ T_i = \text{intercept time} \]
\[ V_1 = \text{first layer velocity} \]
\[ V_2 = \text{second },\text{ }, \text{second} \]
\[ d = \text{thickness of first layer} \]

and are listed in table 1.

The depth results do not agree with the dip-direction of 228° calculated out of the velocities and have a large scattering. This is caused by a not completely flat refractor. The mean of the depth values is 0.56 m, which does correlate with the boundary shale/coal to siltstone (figure 21).

The lines with fan-angle 180° and 225° do not show any difference between \( V_1 \) and \( V_2 \) (appendix A.1), and thus the calculation of the depths for this line is impossible.

Estimations of the parameters from formula (10.1) and from formula (10.9) are listed in table 2. In figure 34 the relations between velocity and fan-angle for the two formulas with the estimated parameters and the corrected refractor velocities out of table 1 are drawn.

<table>
<thead>
<tr>
<th>parameter</th>
<th>estimation with formula:</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \theta_1 )</td>
<td>090°</td>
</tr>
<tr>
<td>( \theta_2 )</td>
<td>202°</td>
</tr>
<tr>
<td>( \xi_1 )</td>
<td>0.04</td>
</tr>
<tr>
<td>( \xi_2 )</td>
<td>0.66</td>
</tr>
<tr>
<td>( p )</td>
<td>65%</td>
</tr>
<tr>
<td>( V_{po} )</td>
<td>1.88 km/s</td>
</tr>
<tr>
<td>correlation coefficient</td>
<td>0.61</td>
</tr>
</tbody>
</table>
Figure 34: NCB-mine velocities against fan-angle

10.2.1.1 Discussion NCB-mine results

In figure 34 it is obvious that there are two maxima: one on 112° and one on 2020, which coincide quite well with the joint directions observed in the field and with the estimated $\theta_1$, $\theta_2$ (the normals of the joint planes).

The overall fit is poor, what implies that at least one of the other parameters is unreliable and that other solutions could be possible.

To check that these poor results are not caused by the algorithm, the estimations were also done by a simplex algorithm [reference 27] and by a Broyden-Powell algorithm (in use at the University of Rotterdam). The results of these
estimations were the same as those with the Marquardt algorithm.
10.2.2 **Black Hill quarry**

The bedding of the layers in the Black Hill quarry is horizontal (section 9.2).

From the time-distance graphs (appendix A.2) it is obvious that, except the line with fan-angle $330^\circ$, the graphs look like a two layer refraction situation. The refractor depths calculated with formula (10.11) are listed in table 3.

<table>
<thead>
<tr>
<th>fan-angle</th>
<th>$V_1$</th>
<th>$V_2$</th>
<th>refractor depth calculated according formula (10.11)</th>
</tr>
</thead>
<tbody>
<tr>
<td>degrees</td>
<td>km/s</td>
<td>m</td>
<td></td>
</tr>
<tr>
<td>330</td>
<td>1.34</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>352</td>
<td>0.70</td>
<td>2.25</td>
<td>1.14</td>
</tr>
<tr>
<td>015</td>
<td>0.49</td>
<td>2.40</td>
<td>1.08</td>
</tr>
<tr>
<td>037</td>
<td>0.43</td>
<td>1.62</td>
<td>0.54</td>
</tr>
<tr>
<td>050</td>
<td>0.26</td>
<td>1.84</td>
<td>0.50</td>
</tr>
<tr>
<td>070</td>
<td>0.27</td>
<td>3.65</td>
<td>0.77</td>
</tr>
<tr>
<td>092</td>
<td>0.23</td>
<td>2.63</td>
<td>0.50</td>
</tr>
<tr>
<td>115</td>
<td>0.48</td>
<td>1.73</td>
<td>0.43</td>
</tr>
<tr>
<td>137</td>
<td>0.67</td>
<td>1.53</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Because the geological profile (figure 24) shows no evidence for a refractor at a depth of about 0.65 m, it is likely that the (apparent-) refraction is caused by the fact that the joints become (more-) closed or that the joints become filled (with clay, water, etc.) at a certain depth. In both cases the velocity of the acoustic wave increases, so that on a certain distance from the source the wave, which has travelled via a deeper level, arrives before the direct wave.

Also the energy reduction of the seismic wave on a higher level is much larger than on a deeper level, due to the fact that the absorption coefficient decreases with increasing depth and due to the fact that after crossing an open joint the transmitted wave contains only a fraction of the original wave energy.

The result is that, although the direct wave is the first arriving wave, the wave which has travelled via a deeper
level is often measured as first arrival, because the direct wave does not give a detectible signal (see figure 35).

Figure 35: Influence of open joints on seismic raypath and arriving times

Figure 35 shows that the V1-velocities in the graphs of the lines 352° to 137° (appendix A.2) and in table 3 are likely to be apparent velocities and that only the V1-velocity from line 330° is a real V1-velocity.

If figure 35 shows indeed the real situation in the Black Hill quarry than the intercept-times (T1) become a function of:

1. the depth of the open joints,
2. the spacing of the open joints,
3. the distance between the source and the first open joint.
According to Stephansson et al. (reference 34):

\[ T_1 = \frac{\sqrt{B^2 + H^2} + H}{V_1} - \frac{B + 1/2A}{V_2} \]  

(10.12)

- **T1** = intercept-time
- **A** = joint spacing
- **B** = distance between the source and the first joint
- **H** = joint depth
- **V1** = velocity of the material between the joints
- **V2** = velocity under the tip of the joints

If it is assumed that both joint systems observed in the field contain open joints, the two systems can be defined as:

1. system 1 with \( \theta_1 \) the direction of the normal of the joints, spacing \( A_1 \), and source-first geophone distance \( B_1 \);
2. system 2 with \( \theta_2 \) the direction of the normal of the joints, spacing \( A_2 \), and source-first geophone distance \( B_2 \).

The mean spacing of the joints in the direction \( \theta \) becomes:

\[ A = \frac{1}{A_1} \cos(\theta - \theta_1)! + \frac{1}{A_2} \cos(\theta - \theta_2)! \]  

(10.13)

Formulae (10.12) and (10.13) give estimated values as listed in table 4. The distance between source-first geophone (B) is defined as illustrated in figure 36. Because the arrival-times in the direction \( 330^\circ \) do not seem to be disturbed by open joints, while in the direction \( 070^\circ \) the open joint effect seems to be maximum, it is likely that only one joint system causes the apparent refraction.

The joint spacing \( A \) in the direction \( \theta \) becomes, for one joint system:

\[ A = \frac{A_1}{1\cos(\theta - \theta_1)!} \]  

(10.13)

Formulae (10.12) and (10.14) with a \( V_1 \)-velocity of 1.34 km/s give the estimated values which are listed in table 4. The correlation coefficient for the one-joint system is about the same as the correlation coefficient for the two-joint system, although the number of degrees of freedom (= number of measurements - number of parameters) for the two-
<table>
<thead>
<tr>
<th>parameter</th>
<th>estimated value</th>
</tr>
</thead>
<tbody>
<tr>
<td>for 2 joint systems</td>
<td>for 1 joint system</td>
</tr>
<tr>
<td>$\theta_1$</td>
<td>$084^\circ$</td>
</tr>
<tr>
<td>$\theta_2$</td>
<td>$352^\circ$</td>
</tr>
<tr>
<td>A1</td>
<td>4.4 m</td>
</tr>
<tr>
<td>A2</td>
<td>4.5 m</td>
</tr>
<tr>
<td>B1</td>
<td>3.5 m</td>
</tr>
<tr>
<td>B2</td>
<td>1.2 m</td>
</tr>
<tr>
<td>depth</td>
<td>5.4 m</td>
</tr>
<tr>
<td>velocity</td>
<td>2.1 km/s</td>
</tr>
<tr>
<td>correlation!</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Joint system is lower than for the one-joint system. This confirms that one joint system with joint-normal direction $078^\circ$ is likely to be sufficient.

Also the distance source-first open joint (B) in the one-joint system is small enough to explain the later arrival-times at small distances source-geophone in the directions $050^\circ$ and $092^\circ$ (see appendix A.2). This is in contrary to the distance source-first open joint in the two-joint system.

Although the estimated joint depth for the one-joint system is more than the depth to the bottom of the shale layer (see figure 24), it is likely that the V2-velocities are the velocities of the top of the sandstone layer directly under the shale layer, which lies at a depth of about 4.1 m.

The velocity of the shale layer can now roughly be calculated if it is assumed that:

1. the minimum sandstone velocity at the surface is 1.34 km/s perpendicular to the surface,
2. the maximum sandstone velocity just above the shale layer is 3.65 km/s perpendicular to the surface,
3. the increase of the velocity is linear with depth:

$$V = h \left( \frac{3.65 - 1.34}{2.5} \right) + 1.34$$
The traveltime through the sandstone perpendicular to the surface now becomes:

\[ T(\text{sandstone}) = \int_{0}^{2.5} \frac{dh}{V} \]

and the average velocity of the sandstone becomes:

\[ V(\text{sandstone}) = \frac{2.5}{T(\text{sandstone})} \]

and because:

\[ \frac{H(\text{estimated})}{V(\text{estimated})} = \frac{H(\text{sandstone})}{V(\text{sandstone})} + \frac{H(\text{shale})}{V(\text{shale})} \]

\[ V(\text{shale}) \approx 1 \text{ km/s}, \text{ what seems to be reasonable for a completely weathered shale on a depth of 2.5 m, and thus under an effective stress of at least: 52 kN/m}^2. \]
The V2-velocities are velocities which seem not to be influenced by open joints, but as is obvious from the V2-velocity variation, there has to be a dominant discontinuity direction. Which is likely to be a closed joint system and thus should vary according to the functions given in section 10.1. Estimates of the parameters from formula (10.1) and from formula (10.9) are listed in table 5 and in figure 37 the relations between velocity and fan-angle for the two formulas with the estimated parameters and the V2-velocities out of table 3 are plotted.

Table 5.

<table>
<thead>
<tr>
<th>parameter</th>
<th>estimation with formula:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(10.1)</td>
</tr>
<tr>
<td></td>
<td>(10.9)</td>
</tr>
<tr>
<td>θ₁</td>
<td>088°</td>
</tr>
<tr>
<td>θ₂</td>
<td>347°</td>
</tr>
<tr>
<td>ε₁</td>
<td>0.35</td>
</tr>
<tr>
<td>ε₂</td>
<td>1.00</td>
</tr>
<tr>
<td>p</td>
<td>67 %</td>
</tr>
<tr>
<td>Vₚ₀</td>
<td>3.96 km/s</td>
</tr>
<tr>
<td>correlation coefficient</td>
<td>0.84</td>
</tr>
<tr>
<td>correlation coefficient</td>
<td>0.84</td>
</tr>
</tbody>
</table>

10.2.2.1 Discussion Black Hill quarry results

From the open- and closed-joint analyses it can be concluded that one system consists mainly of open joints and that the other system consists mainly of closed joints. Both systems are about perpendicular to each other. The joint systems do fit exactly with the joint systems observed, whereby the system which is parallel to the quarry-cliff (joint-normal 180°) is the open system and the system perpendicular to the quarry-cliff (joint-normal 350°) is the closed system. That one system is open is likely caused by expansion of the rock masses in the direction of the quarry-cliff. The joint spacing of 4.9 m for the open joints (see table 4, one joint system) seems to be reliable.
This value can not be compared with field observations because measuring of joint openness can be done on the excavation cliff-sides only which does not give information about the continuation of the openness through the entire rock-mass.

The estimated closed-joint densities of table 5 must be considered as nonsense, because a joint density ($\beta$) value of 1 means that the joint volume equals the rock-mass volume. This means as well that the estimated saturation degree and the estimated P-wave velocity are unreliable.
10.2.3 Greehow Hill quarry

From the time-distance graphs (appendix A.3) it is clear that there are three layers:

1. layer 1, likely to be the clay overburden with a velocity of about 0.68 km/s,
2. layer 2, likely to be a limestone layer with a velocity of about 1.71 km/s,
3. layer 3, a second limestone layer with a large velocity variation between 2.37 and 4.00 km/s.

The depths of the various layers calculated out of the velocities and the intercept times from appendix A.3 are listed in table 6.

<table>
<thead>
<tr>
<th>fan-angle</th>
<th>thickness of 1st layer</th>
<th>thickness of 2nd layer</th>
<th>thickness of 1st+2nd layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>degrees</td>
<td>m</td>
<td>m</td>
<td>m</td>
</tr>
<tr>
<td>217</td>
<td>0.87</td>
<td>0.65</td>
<td>1.53</td>
</tr>
<tr>
<td>240</td>
<td>0.46</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>262</td>
<td>-</td>
<td>-</td>
<td>2.31</td>
</tr>
<tr>
<td>285</td>
<td>0.33</td>
<td>1.31</td>
<td>1.65</td>
</tr>
<tr>
<td>307</td>
<td>0.48</td>
<td>1.00</td>
<td>1.49</td>
</tr>
<tr>
<td>330</td>
<td>0.43</td>
<td>0.99</td>
<td>1.43</td>
</tr>
<tr>
<td>352</td>
<td>-</td>
<td>-</td>
<td>1.37</td>
</tr>
<tr>
<td>015</td>
<td>-</td>
<td>-</td>
<td>1.44</td>
</tr>
<tr>
<td>mean</td>
<td>0.52</td>
<td>0.99</td>
<td>1.60</td>
</tr>
</tbody>
</table>

In the directions 240°, 262°, 352° and 015° there is no evidence of a second refractor, although the refractor depths for these directions are about equal to the depth of the second refractor in the other directions. This proves that also in these directions the V_3-velocities are likely to be from the deeper limestone layer.
Because the bedding of the layers in the Geehow Hill quarry is not horizontal (see section 9.3), the velocities are corrected for the bedding-slope analogue to what has been described for the NCB-mine velocities. Applying:

\[
\frac{1}{V_{\text{nor}}} = \frac{1}{V_{\text{cor}}} + A \cos \varphi + B \sin \varphi
\]

\[
\frac{1}{V_{\text{rev}}} = \frac{1}{V_{\text{cor}}} + A \cos(\varphi + 180^\circ) + B \sin(\varphi + 180^\circ)
\]

gives:

\[
A = -0.0569 \text{ s/m}
\]

\[
B = 0.0196 \text{ s/m}
\]

This gives an angle of maximum dip of:

\[
\arctan \frac{B}{A} = -19^\circ
\]

which is in agreement with the in the field measured dip-direction of the bedding. The real refractor velocities are listed in table 7.

<table>
<thead>
<tr>
<th>fan-angle</th>
<th>( V_1 )</th>
<th>( V_2 )</th>
<th>( V_3^{\text{nor}} )</th>
<th>( V_3^{\text{cor}} )</th>
<th>( V_3^{\text{rev}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>degrees</td>
<td>km/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>217</td>
<td>1</td>
<td>(0.68)</td>
<td>1.71</td>
<td>2.37</td>
<td>2.81</td>
</tr>
<tr>
<td>240</td>
<td>1</td>
<td>1.01</td>
<td>1.48</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>262</td>
<td>1</td>
<td>1.08</td>
<td>-</td>
<td>3.23</td>
<td>3.11</td>
</tr>
<tr>
<td>285</td>
<td>1</td>
<td>0.67</td>
<td>1.66</td>
<td>3.08</td>
<td>2.79</td>
</tr>
<tr>
<td>307</td>
<td>(0.68)</td>
<td>1.78</td>
<td>3.70</td>
<td>3.12</td>
<td>2.70</td>
</tr>
<tr>
<td>330</td>
<td>(0.68)</td>
<td>1.63</td>
<td>2.96</td>
<td>2.52</td>
<td>2.19</td>
</tr>
<tr>
<td>352</td>
<td>0.65</td>
<td>-</td>
<td>2.37</td>
<td>2.08</td>
<td>1.85</td>
</tr>
<tr>
<td>375</td>
<td>0.68</td>
<td>-</td>
<td>4.00</td>
<td>3.33</td>
<td>2.86</td>
</tr>
</tbody>
</table>

Estimations of the parameters from formula (10.1) and from formula (10.9) are listed in table 8 and figure 38 shows the relations between velocity and fan-angle for the two formulas with the estimated parameters, and together with the \( V_3^{\text{cor}} \)-velocities out of table 7.
Table 8.

<table>
<thead>
<tr>
<th>parameter</th>
<th>estimation with formula:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(10.1)</td>
</tr>
<tr>
<td>$\theta_1$</td>
<td>320°</td>
</tr>
<tr>
<td>$\theta_2$</td>
<td>194°</td>
</tr>
<tr>
<td>$\xi_1$</td>
<td>0.44</td>
</tr>
<tr>
<td>$\xi_2$</td>
<td>1.00</td>
</tr>
<tr>
<td>p</td>
<td>94 %</td>
</tr>
<tr>
<td>$V_{p0}$</td>
<td>3.76 km/s</td>
</tr>
<tr>
<td>correlation</td>
<td></td>
</tr>
<tr>
<td>coefficient</td>
<td>0.32</td>
</tr>
</tbody>
</table>

10.2.3.1 Discussion Greehow Hill quarry results

As in the NCB-mine and in Black Hill quarry the correlation between the measured joint directions and the velocity maxima is very good.
The estimated joint normals also agree with the field directions, but as in the foregoing sections, the estimated joint densities are likely to be unreliable and thus the saturation degree and the P-wave velocity in intact rock are as well unreliable.
10.2.4 Magnesium Limestone quarry

It is clear from the time-distance graphs that there are three layers:

1. layer 1, likely to be the residual soil with a velocity of 0.33 km/s,

2. layer 2, with a mean velocity of about 0.49 km/s and a velocity variation between 0.42 km/s and 0.72 km/s,
3. Layer 3, with a mean velocity of about 1 km/s and a velocity variation between 0.69 km/s and 1.58 km/s.

Because it is likely that the layers are dipping, (see appendix A.4 and figure 30), formula (10.10) is applied to calculate the corrected layer velocities. These velocities are listed in table 9.

Table 9.

<table>
<thead>
<tr>
<th>fan-angle</th>
<th>V_1</th>
<th>V_2nor</th>
<th>V_2cor</th>
<th>V_2rev</th>
<th>V_3nor</th>
<th>V_3cor</th>
<th>V_3rev</th>
</tr>
</thead>
<tbody>
<tr>
<td>degrees</td>
<td>km/s</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>355</td>
<td>0.33</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.86</td>
<td>1.58</td>
<td>1.37</td>
</tr>
<tr>
<td>018</td>
<td>0.31</td>
<td>0.54</td>
<td>0.52</td>
<td>0.50</td>
<td>1.09</td>
<td>1.01</td>
<td>0.94</td>
</tr>
<tr>
<td>041</td>
<td>0.33</td>
<td>0.44</td>
<td>0.43</td>
<td>0.43</td>
<td>0.76</td>
<td>0.74</td>
<td>0.72</td>
</tr>
<tr>
<td>063</td>
<td>0.34</td>
<td>0.53</td>
<td>0.53</td>
<td>0.53</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>085</td>
<td>0.35</td>
<td>0.41</td>
<td>0.42</td>
<td>0.42</td>
<td>0.96</td>
<td>1.00</td>
<td>1.04</td>
</tr>
<tr>
<td>108</td>
<td>0.34</td>
<td>0.47</td>
<td>0.49</td>
<td>0.50</td>
<td>0.66</td>
<td>0.69</td>
<td>0.73</td>
</tr>
<tr>
<td>130</td>
<td>0.34</td>
<td>0.52</td>
<td>0.55</td>
<td>0.58</td>
<td>0.88</td>
<td>0.96</td>
<td>1.06</td>
</tr>
<tr>
<td>153</td>
<td>0.26</td>
<td>0.67</td>
<td>0.72</td>
<td>0.78</td>
<td>0.93</td>
<td>1.03</td>
<td>1.15</td>
</tr>
</tbody>
</table>

Calculations of the depths of the various layers are listed in table 10. The thickness of the first layer does agree with the thickness of the residual soil, as proposed above, and the thickness of the second layer is likely to be the moderately weathered zone. The depth of the third layer is, except for the depth in the direction 041°, more or less constant with a mean value of 2.3 m.

Estimations of the parameters from formula (10.1) and formula (10.9) with the second and third layer velocities are listed in table 11. Figure 39 shows the relations between velocity and fan-angle for the two formulas and are plotted the V_2cor and V_3cor velocities out of table 9.
Table 10.

<table>
<thead>
<tr>
<th>fan-angle</th>
<th>thickness of</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st layer</td>
</tr>
<tr>
<td>degrees</td>
<td>m</td>
</tr>
<tr>
<td>355</td>
<td>-</td>
</tr>
<tr>
<td>018</td>
<td>0.5</td>
</tr>
<tr>
<td>041</td>
<td>0.4</td>
</tr>
<tr>
<td>063</td>
<td>0.7</td>
</tr>
<tr>
<td>085</td>
<td>0.2</td>
</tr>
<tr>
<td>108</td>
<td>0.5</td>
</tr>
<tr>
<td>130</td>
<td>0.6</td>
</tr>
<tr>
<td>153</td>
<td>0.9</td>
</tr>
</tbody>
</table>

10.2.5 Claypit

As already mentioned in section 9.5 two different seismic measurements were done in the claypit.

10.2.5.1 Fan-shooting

From the time-distance graphs for the first arrival P-wave (see appendix A.5) it is clear that there is one regular refraactor at a depth of about 1.3 m. The velocities (see figure 40) are between 1.5 and 1.65 km/s with a mean velocity of 1.60 km/s over all profiles. These velocities are typical for a water saturated clay, so that it is most likely that the refractor is the groundwater table. Above this level there was a gradual decrease of velocity up to the surface.

The velocity variation is about equal to the uncertainty interval, so that it is likely that the velocity variation is caused by measuring scattering.
### Table 11.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>2nd layer</th>
<th>3rd layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>estimation with formula:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\delta_1$</td>
<td>$180^\circ$</td>
<td>$190^\circ$</td>
</tr>
<tr>
<td>$\delta_2$</td>
<td>$071^\circ$</td>
<td>$069^\circ$</td>
</tr>
<tr>
<td>$\varepsilon_1$</td>
<td>0.76</td>
<td>1.00</td>
</tr>
<tr>
<td>$\varepsilon_2$</td>
<td>0.03</td>
<td>0.00</td>
</tr>
<tr>
<td>$\rho$ (%)</td>
<td>41</td>
<td>70</td>
</tr>
<tr>
<td>$v_{po}$ (km/s)</td>
<td>0.85</td>
<td>0.79</td>
</tr>
<tr>
<td>correlation coefficient</td>
<td>0.85</td>
<td>0.87</td>
</tr>
<tr>
<td>p (%)</td>
<td>41</td>
<td>70</td>
</tr>
<tr>
<td>$v_{po}$ (km/s)</td>
<td>0.85</td>
<td>0.79</td>
</tr>
<tr>
<td>correlation coefficient</td>
<td>0.85</td>
<td>0.87</td>
</tr>
</tbody>
</table>

#### 10.2.5.2 Long-distance line

The long-distance line shows two refractors; one at a depth of 1.3 m (the groundwater table) and one at a depth of 14 m with a velocity of 3.13 km/s.

This last refractor is likely to be the bedrock just below the excavation depth of the claypit.
2.0 velocity (km/s)

1.0

0.0

fan-angle (degrees)

+ V_{2\text{cor}} \text{ resp. } V_{3\text{cor}} \text{ out of table 9 with uncertainty interval}

--- velocity variation according to formula (10.1) with parameters out of table 11

-- velocity variation according to formula (10.9) with parameters out of table 11

in field observed joint directions

Figure 39: Magnesium Limestone quarry velocities against fan-angle

10.3 CONCLUSIONS-VELOCITY ANISOTROPY

Out of the velocity anisotropy, as described in the sections above, can be drawn the following general conclusions:

1. The differences in velocity in a particular ground-mass, due to direction only, can be as large as a factor two. This means that the me-
1. The velocity seems to be very sensitive to the joint-density, so that the velocity anisotropy can be used to estimate the joint directions.

2. To deduce the joint-density out of the velocities is, in general, not possible, because open joints with significant openness do not transmit the seismic wave. The wave will then be diffracted at the ends of the open joints so that the raypaths of the seismic waves in different directions are no longer in one plane. As described in the literature review the seismic velocity is dependent on the effective stress, this implies that the seismic velocity increases with depth. This means that the raypath in a direction with a higher open joint density will be at a deeper level than in directions with a lower open joint density or without open joints.

The above factors make that the intact-ground velocity in the formulas (10.1) and (10.9) depends on the direction.

Figure 40: Claypit velocities against fan-angle

P-wave velocity with uncertainty interval
4. The difficulties, described in the points above, will cause the estimated water-saturation degree and the estimated intact rock-mass velocity to be also unreliable.

5. The number of refractors, which of course depends on the number of different velocities, can vary with direction, because the velocities of different refractors can be equal in one direction and can be unequal in another direction.

6. It is not possible to decide which of the formulae (10.1) or (10.9) best describes a jointed ground-mass because of the difficulties listed above.
SECTION XI
GROUND-MASS PARAMETERS OUT OF ENERGY RELATIONS OF P-WAVES

11.1 THEORY

Decrease of energy of P-waves passing through ground-masses is caused by four mechanisms:

1. spherical divergence
2. absorption
   a. in the ground-mass itself
   b. in the filling material in the joints
3. partitioning of energy at an interface (boundary rock-joint infill)
4. extension of seismic raypath through open joints

11.1.1 Spherical divergence

Wavefronts diverge spherically from a shotpoint; thus if the total energy does not change, the energy per unit area will decrease directly proportional with distance. At the same time there is a decrease of the total energy due to absorption. If $E(A) =$ energy density on a distance $r$ from the shotpoint then the energy $E(tA)$, which flows through area $A$ in a unit time (see figure 41) becomes,

$$E(tA) = E(A) \times V \times A$$

and analogue:

$$E(tB) = E(B) \times V \times B$$

$$V = \text{wave velocity}$$
The total energy $E(t_B)$ is less than $E(t_A)$ due to absorption. This absorption is expected to be exponential with distance:

$$E_r = E_0 e^{-ar} \quad (11.2)$$

$E_r = $ energy at distance $r$
$E_0 = $, , , a unit distance from shot point
$a = $ absorption coefficient
$r = $ distance to shotpoint

The absorption coefficient is roughly equal to $c/\lambda$ and with $V = f \times \lambda$ gives:

$$a = \frac{f \cdot c}{V} \quad (11.3)$$

$f = $ frequency
$\lambda = $ wavelength
$c = $ constant for a certain type of rock
$V = $ wave velocity
in which $f$ is a function of $r$:

$$a = \frac{f(r)\cdot c}{V}$$

($c/V$ is now a rock constant)

(11.2) + (11.3) give:

$$E_r = E_0 e^{-f(r)\cdot c\cdot r/V}$$

and the quotient between the energy on distance $r$ and $r+dr$:

$$\frac{E(r+dr) - (f(r+dr)(r+dr)-f(r)\cdot r)\cdot c/V}{E(r)} = e$$

Formula (11.1) describe the difference in energy between $E(tA)$ and $E(tB)$:

$$\frac{E(tB) - (f(r+dr)(r+dr)-f(r)\cdot r)\cdot c/V}{E(tA) - (f(r+dr)(r+dr)-f(r)\cdot r)\cdot c/V} = e$$

area $A = r\cdot da\cdot r\cdot db$
area $B = (r+dr)\cdot da\cdot (r+dr)\cdot db$

$$\frac{E(r+dr) - (f(r+dr)(r+dr)-f(r)\cdot r)\cdot c/V}{E(r) - (r+dr)^2} = e$$

which give after integration:

$$E(r) = C_0 \frac{1 - f(r)\cdot r\cdot c/V}{r^2}$$

(11.4)

11.1.2 Partitioning of energy at an interface

If a harmonic P-wave (which a seismic P-wave is expected to be) crosses a change of elastic parameters, the wave will split in a reflected P- and S-wave and in a refracted P- and S-wave (see figure 42). The relationships (1) between these waves are given in terms of amplitudes by Knott (1899)
If in a first approximation \( a(1) = 0^\circ \) (angle of incidence = 90\(^\circ\)) then:

\[
\frac{A_2}{A_1} = \frac{Z_1 - Z_2}{Z_1 + Z_2}
\]

in which: \( Z = \rho \times V \)

\( \rho = \) density of medium \\
\( V = \) velocity of medium

and the so-called 'transmission coefficient (T)' becomes:

\[
T = \frac{E_2}{E_1} = \frac{4 \times Z_1 \times Z_2}{(Z_1 + Z_2)^2}
\]

Figure 42: Partitioning of energy at interface

---

(1) The reader is referred to the literature for a description of these relationships.
11.1.3 Open joints

Calculating formula (11.5) for an open joint with dimensions larger than the wavelength of the seismic wave, the transmission coefficient becomes:

\[ T = \frac{4.2500 \cdot 5000 \cdot 1.360}{(2500 \cdot 5000 + 1.360)} = 1.1 \times 10^{-4} \]

\( \rho(\text{rock}) = 2500 \text{ kg/m}^3 \)
\( \rho(\text{air}) = 1 \text{ kg/m}^3 \)
\( V(\text{rock}) = 5000 \text{ m/s} \)
\( V(\text{air}) = 360 \text{ m/s} \)

for the energy-transmission rock-air; for air-rock the transmission coefficient is the same, so that the total transmission coefficient for one open joint becomes: 1.21 \times 10^{-8}.

If \( J \) is the amount of joints per meter than:

\[ E(2) = E(0) \cdot T \]

By example: for \( J = 10 \) joints/m, which is a quite common amount of joints per meter, the energy coming out of one meter of jointed rock becomes:

\[ E = E(0) \cdot (1.21 \times 10^{-8}) = E(0) \times 10^{-80} \]

This illustrates that energy transport through open joints can not be important, because the energy arriving at the geophones will be unmeasurable, and thus will never be recognised as first arrival.

This means that energy arriving at the geophones comes through solid rock only. But therefore it is obvious that the raypath will become longer dependent on the amount of open joints.

11.1.4 Spherically extending wave propagation in a jointed ground-mass

If \( \delta r \) is defined as the extra raypath due to one open joint than \( r_n \delta r \) is the extra raypath over a distance \( r \) due to \( n \) open joint/meter.

If \( T \) is the transmission coefficient for the energy passing a closed or filled joint than \( T_n \delta r \) is the transmission coefficient for \( n \) closed joints/meter over a distance \( r \).

The energy relationship in a jointed ground-mass becomes:

\[ \frac{E(R_2)}{E(R_1)} = \left( \frac{r_1}{r_2} \right)^2 \left( \frac{\alpha(r_2-r_1)}{n_c(r_2-r_1)} \right) e^{-\alpha(r_2-r_1)} \cdot T_n \]  

\[ \text{(11.6)} \]
In which:

\[ R_i = r_i + r_i n_c \frac{\Delta r}{r} \]

In theory formula (11.6) should describe a spherically extending wave in a jointed ground-mass. In practice the following three problems occur:

1. the difference between an open and a closed joint depends on the frequency of the seismic signal in comparison with the openness of the joint (see section 3.2.3),
2. the waves measured in the quarries are not spherical extending, but refracted waves,
3. the uncertainty in the amplitude measurements is so large that estimation of the parameters from formula (11.6) becomes very difficult and costs enormous amounts of computing-time, without the certainty that the results are reliable.

11.1.5 Attenuation of refracted waves

For use in the investigated quarries formula (11.4) had to be modified for refracted waves. A refracted wave travels just below the refractor plane (see figure 43). If it is assumed that the refractor plane is flat, than the energy density (I) reduces, due to area expansion, with:

\[ D.I_1.r_1.d \varphi = D.I_2.r_2.d \varphi \Rightarrow \frac{I_1}{I_2} = \frac{r_2}{r_1} \]

thus the intensity decreases inversely with distance. The energy which, is refracted into the lower velocity layer above, depends on the ground-mass parameters of the two layers (density, seismic velocity, jointing-degree and jointing-orientation, etc.) and the contact between the two layers. These parameters are unknown in the investigated quarries, but if it is assumed that these energy losses are dependent on distance in the same way as is expected for the absorption in the ground-mass itself, then the energy relation becomes relatively simple:

\[ \frac{I_1}{I_2} = \frac{r_2^{r_2-r_1}}{r_1} \]

(11.8)
Formulas (11.8) and (11.9) give:

\[
\frac{1}{2} \rho V \omega^2 A^2 = \frac{r_2 - a(r_2 - r_1)}{r_1} \tag{11.10}
\]

The square root of this function:

\[
\frac{\omega_1 A_1}{\omega_2 A_2} = \left( \frac{r_2}{r_1} \right)^{1/2} \frac{1}{e} - 1/2a(r_2 - r_1) \tag{11.11}
\]

is, except for the \((r_2/r_1)^{1/2}\) term, equal to the proposed solution of Roussel (reference 30) for a moving wave in a visco-elastic model (\(\omega\) is assumed to be constant) so that it may be possible to use the absorption factor from formula (11.11) in the correction factor for the calculation of the static \(E\) modulus out of the seismic \(E\) modulus.

According to Roussel:

\[
\lambda + 2\mu = \int V^2 \omega^2 \left( \frac{\omega^2 - a^2 V^2}{\omega^2 + a^2 V^2} \right) \tag{11.12}
\]

- 103 -
with:

\[ E(\text{seis}) \cdot f(\sqrt{\text{seis}}) = \rho \, V^2 \]
\[ E(\text{stat}) \cdot f(\sqrt{\text{stat}}) = \lambda + 2\nu \]

and

\[ f(\sqrt{\text{seis}}) = \frac{1 - \sqrt{\text{seis}}}{(1 + \sqrt{\text{seis}})(1 - 2\sqrt{\text{seis}})} \]
\[ f(\sqrt{\text{stat}}) = \frac{1 - \sqrt{\text{stat}}}{(1 + \sqrt{\text{stat}})(1 - 2\sqrt{\text{stat}})} \]

If is assumed that \( \sqrt{\text{seis}} \approx \sqrt{\text{stat}} \) than:

\[ \frac{E(\text{stat})}{E(\text{seis})} = \frac{\omega^2 - a^2\nu^2}{\omega^2 + a^2\nu^2} \]

(11.13)

\( \rho = \text{density} \)
\( V = \text{seismic wave velocity} \)
\( \lambda, \nu = \text{the Lame constants} \)
\( \nu = \text{poisson modulus} \)
11.2 SEISMIC PARAMETERS

For each quarry has been examined in detail the correlation between the following seismic wave parameters and the degree of jointing.

1. Amplitude
   a. The amplitude should decrease with an increase of the amount of joints due to three effects:
      i. partitioning of energy at the joint surfaces,
      ii. multiple reflections due to the joints (see section 5.2),
      iii. joint infill; generally the joint infill will have a structure less compact than the intact ground-mass; a less compact structure will cause a higher absorption factor than the intact ground-mass.
   b. The amplitude should decrease with an increase of the angle of incidence between the seismic raypath and the joint normal. Generally it can be stated that the transmission of energy through a joint become negligible when the angle of incidence is more than about 30°.

2. Frequency
   the frequency should decrease with an increase of the amount of open joints, due to the effect of multiple reflections at the joint surfaces (see section 5.2).

3. The correction factor (K)
   This correlates the static E modulus with the seismic E modulus and results from the assumption that the ground-mass behaves like a visco-elastic model:
   \[
   \frac{E(\text{seis})}{E(\text{stat})} = K = \omega^2 \frac{\left( \omega^2 - a^2v^2 \right)}{\left( \omega^2 + a^2v^2 \right)^2}
   \]
   (11.14)

4. The static E modulus
   This is calculated on the basis of a visco-elastic model as proposed by Roussel whereby the factor
\( f(\nu_{\text{stat}}) \) is set on an arbitrary value of 1; and thus formula (11.12) becomes:

\[
E(\text{stat}) = \int \frac{\omega^2 v^2}{(\omega^2 + a^2 v^2)^2} \, (11.15)
\]

11.3 MEASURING AND CALCULATING OF SEISMIC WAVE PARAMETERS

11.3.1 Measuring arrival-time, amplitude and frequency

Because recording of the received signals in digital form on magnetic tape was not possible (a digital recorder was not available), the signals were recorded on metal paper only by means of the built-in recorder (see figure 44).

![Figure 44: Example metal paper recording](image-url)
The amplitude was measured on this paper in mm and corrected for the gain-setting of the amplifier. The trace size control was for all channels on its maximum value. To keep errors as small as possible no filters were used and thus a correction was not necessary. Also the frequencies of the signals were far above the natural frequency of the geophones, so that the response curve of the geophones was likely to be flat and a correction therefore redundant.

In principle it is enough to measure the arrival-time of the begin and of the first peak of the signal to determine the frequency, but the arrival-time of the maximum peak is difficult to measure with enough accuracy on metal paper. It was found to be better to calculate the frequency out of the mean of:

\[
\frac{1}{4(T_2 - T_1)} \quad \text{and} \quad \frac{1}{4(T_3 - T_2)}
\]

in which the times were measured as in figure 45 is illustrated.

![Figure 45: Measuring seismic wave parameters](image)

Only the first half of the received signal was used because the second half was, over small distances, often disturbed by the surface waves.
Because the time and especially the amplitude had to be measured on the paper there was a severe reduction of accuracy in comparison with digital recording and direct processing of the data, not the least because it was in this setup impossible to differentiate between refracted and reflected signals. Reflected signals can cause systematic errors if they arrive within $0.5 \times \frac{1}{f}$ second after the first arrival of the refracted signal. Because actual measuring of the reflection arrivals was impossible, it is assumed that when the first arrival pulse of the refracted wave was more or less regular, it was not disturbed by reflections.

11.3.2 Calculation of the absorption factor

The absorption factor (a) is calculated by means of the least squares of:

$$\frac{1}{2} \ln ((1/2 \rho V R) \omega A y) = C + a R$$

(11.16)

in which:
- $y$ is a factor to equalize the dimensions in $(\text{kg} \cdot \text{m}^2/\text{s}^3)^{1/2}$ and has a value 1
- $A$ is the amplitude
- $\rho$ is the density of the ground-mass; determined from the samples (see section 12),
- $V$ is the velocity as calculated in section 10,

For $\rho$ and $V$ have been used values which are likely to be true for a certain depth below the surface. The velocity and density at the surface will be lower than these, but are expected to be related to the deeper values. Yet these have been used because the surface values are not known with enough certainty.
- $R$ is the distance along the refractor plane; this distance is calculated as the distance between the point A and the geophone (see figure 46).
- $\omega$ is the angular frequency $= 2\pi f$; $f = \text{frequency}$ as calculated in the section above.
- $C$ is a value which indicates the original energy introduced into the ground-mass, and serves as a control on the accuracy of the calculation of formula (11.13), because the energy introduced into the ground-mass is for the different directions in a particular quarry more or less constant (the number of weight-drops was kept constant for a whole fan).
11.3.3 Line frequency \((\bar{f})\)

The frequencies from the different geophones along one line scattered too much to use them for more than the qualitative determination that the frequency slightly decreases with distance. Therefore a mean frequency for each line (the line frequency, \(\bar{f}\)) is calculated, which is the arithmetic mean of the frequencies from the geophones along a line.

\[ E \left( \text{seis} \right) \frac{\omega^2}{E \left( \text{stat} \right)} = K = \frac{\omega^2 - a^2v^2}{\omega^2 + a^2v^2} \] 

\( \omega = 2\pi f \)

A = absorption factor

V = velocity
11.3.5 The static $E$ modulus

$$E\text{(stat)} = f \left( \frac{\omega^2 - a^2v^2}{\omega^2 + a^2v^2} \right)$$  \hspace{1cm} (11.15)

whereby is assumed that $f(\nu\text{stat}) = 1$, as proposed in section 11.2.
11.4 DISCUSSION OF ATTENUATION

11.4.1 Claypit

The claypit is studied first because the ground-mass in the claypit is expected to be the most regular one, so that the behaviour of formula (11.16) could be studied. In appendix B.1 the absorption, C, f, K and static E modulus calculated as described above are listed, together with the correlation coefficient for the fit of formula (11.16) on the data. The listing shows that:

1. the absorption factor and C-factor (from formula (11.16)) are independent of the fan-angle and are about constant, as is expected for the claypit. The high correlation coefficient confirms that formula (11.16) reasonably describes the seismic signal.

2. the frequencies tend to be lower in the direction of the excavation cliff (direction 180°). The reason for this is presumably that in the direction parallel to the excavation face the fissures are larger due to expansion of the ground-mass in the direction of the excavation face due to stress release. This larger fissures will cause a decrease of frequency.

3. The correction factor (K) and the static E modulus are both about constant.

11.4.2 NCB-mine

The different seismic parameters are listed in appendix B.2, and are plotted in figure 47. The following effects are remarkable:

1. The absorption factor is about constant for the normal shot directions, but the reversed shots give a different absorption factor.

2. The line frequencies (f) show two maxima: at 112° and at 225°. The first maximum coincide with the joint direction of 112°, while the second maximum has a difference of 22° with the joint direction of 205°.

3. The K-factor is nearly constant for the normal shots, and shows a difference between the reversed shot values and the normal shot values.
4. The static $E$ modulus shows a large variation ac-

---

**Figure 47:** Seismic parameters NCB-mine against fan-angle.

- normal shot value
- reversed shot value
cording to the joint directions, whereby it is no-
table that:

a. the static E modulus values for normal and
reversed shots are the same.

b. the graphs between the maxima and the minima
appear to be straight lines.

11.4.3 Black Hill quarry

The seismic parameters are listed in appendix B.3 and are
plotted in figure 48.

1. The absorption values show a minimum in the direc-
tion of the excavation cliff. This is likely to be
caused by expanding of the ground-mass in the di-
rection of the excavation cliff over such a large
distance that large open joints has been created.
The raypath of the seismic waves can not go
through these open joints (see section 5.1.4) but
is diffracted at the lower end of these joints.
This means that the seismic wave travels on a
deeper level through a ground-mass which will have
a better structure and which is under a higher ef-
fective stress. Both effects lower the absorption
factor. The existence of large open joints was
already proposed in section 10.2.2.

2. The line frequencies show a sharp minimum at the
direction where the absorption values show a mini-
um as well.
This is in contrary to the theory, but can be ex-
plained by the fact that parallel to the excava-
tion cliff there is a high density of closed
joints.

3. The K-factor and the static E modulus has to be
considered with care because they are not valid
for the same depth. Yet the K-factor seems to re-
fect the two joint directions. The static E mo-
dulus is likely to reflect mainly the measuring
depth.

11.4.4 Magnesium Limestone quarry

The absorption factor, K-factor and the static E modulus
have been calculated for the second and third layer seper-
tately, and have been calculated without regard to the dif-
Figure 48: Seismic parameters Black Hill quarry
ferent layers. The values are listed in appendix B.4 and are plotted in figure 49.

Out of figure 49 the following conclusions can be drawn:

1. The absorption curves show a maximum at a direction of 090°, whereas the line frequencies do not reflect anything at a direction of 090°. This may be caused by the fact that the joint openness in these directions is so large that the delay of the multiple reflections become more than 0.5 x 1/f in which case they arrive after the first half period of the principal signal (see figure 50). Because the frequency is measured on the basis of the first half period only, these multiples do not lower the frequency.

2. The K-values and the static E modulus for the second layer seem to be nearly constant while these parameters for the third layer show a large variation. The K-factor for the calculations without regard to the different layers show two minima, both shifted 220° from the joint directions.

11.4.5 Greehow Hill quarry

The number of measurements was not large enough to allow calculating of the absorption factor and frequencies for the different layers separately. The K-factor and the static E modulus are calculated on base of the same absorption factor and frequency, but with different velocities.

Out of figure 51 and appendix B.5 the following conclusions can be drawn:

1. The absorption factor and the frequencies do not show any relation to the joint directions.

2. The K-factor for the second layer is constant while the K-factor for the third layer shows two maxima which coincide nearly with the joint directions.

3. The static E modulus for the third layer shows a large maximum which coincides nearly with the joint direction of 2920°.
Figure 49: Seismic parameters Magnesium Limestone quarry
11.5 CONCLUSIONS—ATTENUATION ANISOTROPY

On the basis of the results described in the sections above the following general conclusions can be formulated:

1. The absorption factor is not—or nearly not— influenced by the joints themselves; this fact has been mentioned already by King (reference 20).

2. Large open joints cause a smaller absorption factor, because they cause the wave to travel at a greater depth.

3. The K-factor does not reflect the joint directions if the joints are not too large, or if the differ-
Figure 51: Seismic parameters Greehow Hill quarry
ence between joint-infill material and intact ground material is not too large. This demonstrates that a ground-mass can be considered approximately as a visco-elastic model if the discontinuities are not too large.
Samples were taken from each quarry from the excavation cliff, except in the Greehow Hill quarry where the samples were taken from the rock outcrop above the quarry bottom. In the claypit were samples taken from various depths along the excavation cliff (see section 9).

12.1 LABORATORY TESTS ON QUARRY SAMPLES

From each sample was determined:
- density (dry)
- porosity
- ultrasonic velocity (wet and dry)
- Point Load Strength (PLS)
- tensile strength (by mean of a Brazilian test)
- Unconfined Compressive Strength (UCS)
- static E modulus

The last test was done in combination with the determination of the UCS.

The test results are listed in table 12 with, for each test-series, the standard deviation.

12.2 LABORATORY TESTS ON CLAY SAMPLES

Block samples were taken from the excavation slope at depths of: 2.4, 4.7, 7.5, and 9.4 m. On these samples the following laboratory tests were done:
- determination of:
  - bulk density
  - dry-density
  - moisture content
  - ultrasonic velocity
  - sheartest
  - triaxal test

The results showed that:
Table 12.

<table>
<thead>
<tr>
<th>Density (dry)</th>
<th>Mg/m³</th>
<th>2.66</th>
<th>2.12</th>
<th>2.65</th>
<th>11.64</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porosity</td>
<td>%</td>
<td>3.22</td>
<td>17.8</td>
<td>1.38</td>
<td>40.4</td>
</tr>
<tr>
<td>Ultrasound</td>
<td>km/s</td>
<td>5.59</td>
<td>3.06</td>
<td>6.04</td>
<td>2.44</td>
</tr>
<tr>
<td>Velocity</td>
<td></td>
<td>5.39</td>
<td>2.41</td>
<td>5.68</td>
<td>2.37</td>
</tr>
<tr>
<td>PLS</td>
<td></td>
<td>9.42</td>
<td>1.59</td>
<td>4.15</td>
<td>1.36</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>MN/m²</td>
<td>26.9</td>
<td>3.01</td>
<td>9.86</td>
<td>1.83</td>
</tr>
<tr>
<td>UCS</td>
<td></td>
<td>135.</td>
<td>18.2</td>
<td>61.5</td>
<td>5.99</td>
</tr>
<tr>
<td>Static E modulus</td>
<td>x 10⁴</td>
<td>4.1</td>
<td>0.49</td>
<td>2.5</td>
<td>0.45</td>
</tr>
</tbody>
</table>

(values between brackets are the standard deviation)

1. The bulk density increased slightly with depth, as expected. (see table 13).

2. The dry density and moisture content show a variation that can be caused by a locally higher content of silt. More silt in a clay layer will make the clay more porous and permeable. A higher porosity will give a higher moisture content with a lower dry density. A higher permeability will allow the clay to loose more water during the drying process.
Table 13.

<table>
<thead>
<tr>
<th>depth</th>
<th>bulk density</th>
<th>dry density</th>
<th>moisture content</th>
<th>ultrasonic velocity</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>ton/m³</td>
<td>%</td>
<td>km/s</td>
<td></td>
</tr>
<tr>
<td>2.4</td>
<td>1.863</td>
<td>1.424</td>
<td>30.7</td>
<td>1.51</td>
</tr>
<tr>
<td></td>
<td>(0.008)</td>
<td>(0.02)</td>
<td>(0.8)</td>
<td>(0.14)</td>
</tr>
<tr>
<td>4.7</td>
<td>1.867</td>
<td>1.411</td>
<td>32.4</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>(0.02)</td>
<td>(0.02)</td>
<td>(0.7)</td>
<td>(0.35)</td>
</tr>
<tr>
<td>7.5</td>
<td>1.870</td>
<td>1.416</td>
<td>32.0</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>(0.02)</td>
<td>(0.02)</td>
<td>(1.0)</td>
<td>(3)</td>
</tr>
<tr>
<td>9.4</td>
<td>1.878</td>
<td>1.433</td>
<td>31.1</td>
<td>1.62</td>
</tr>
<tr>
<td></td>
<td>(0.03)</td>
<td>(0.04)</td>
<td>(1.6)</td>
<td>(25)</td>
</tr>
</tbody>
</table>

(values between brackets are the standard deviation)

3. The ultrasonic velocity measured horizontally is about constant (1.55 km/s); measured perpendicular to the bedding there is a light increase with depth. (see table 13 and figure 52).

4. Shear tests (using a shear box) were done on 10 samples (see appendix C.1) from various depths. For all tests the average cohesion was 36 KN/m² with an angle of internal friction of 110°. It is likely that the three encircled points belonged to a shear plane in a silt layer or in a more silty clay layer, so that the non-circled points reflect the shear values of the clay. The cohesion of the clay becomes then: 51.4 KN/m² and the angle of internal friction becomes 1.960°; for the more silty layers the values become respectively 0 KN/m² and 30°. The first values agree with the already proposed idea of a firm clay.

5. Triaxal test (see appendix C.2 and table 14) samples had to be cut with a tube from the block-samples. To do this exactly perpendicular or parallel to the bedding-planes of the clay was nearly im-
possible. So the variations in the triaxial test results will be caused by inexact perpendicularity or parallelism of the samples to the bedding-planes.

The combination of the results of both shear tests and triaxial tests shows a firm clay with a cohesion of 50 KN/m² and an angle of internal friction of about 20°, in which there are, up to a depth of about 5 meter, some more silty layers with a cohesion of about 0 KN/m² and an angle of internal friction of about 25°.
Table 14.

<table>
<thead>
<tr>
<th>depth (m)</th>
<th>cohesion (kN/m²)</th>
<th>angle of internal friction (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.4</td>
<td>2.5</td>
<td>21</td>
</tr>
<tr>
<td>4.7</td>
<td>(50)</td>
<td>(0)</td>
</tr>
<tr>
<td>7.5</td>
<td>40</td>
<td>3</td>
</tr>
<tr>
<td>9.4</td>
<td>50</td>
<td>0</td>
</tr>
</tbody>
</table>

(Values between brackets are the standard deviation)
SECTION XIII

PHOTO INTERPRETATION

Stereo-photographs were taken in the quarries from the excavation cliffs, except for the Greehow Hill quarry where the photos were taken from the outcrop above the quarry bottom. An interpretation of the jointing pattern was made on the basis of these photographs and is shown in appendix D. In table 15 the measured joint densities are listed.

Table 15.

<table>
<thead>
<tr>
<th>Quarry</th>
<th>Jointing Pattern</th>
<th>Joint Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCB-Mine</td>
<td>(112°)</td>
<td>1.8 joints/m</td>
</tr>
<tr>
<td></td>
<td>(205°)</td>
<td>1.6 joints/m</td>
</tr>
<tr>
<td>Black Hill</td>
<td>(000°)</td>
<td>2.1 joints/m</td>
</tr>
<tr>
<td></td>
<td>(070°)</td>
<td>1.5 joints/m</td>
</tr>
<tr>
<td>Greehow Hill</td>
<td>(285°)</td>
<td>5.9 joints/m</td>
</tr>
<tr>
<td></td>
<td>(015°)</td>
<td>2.1 joints/m</td>
</tr>
<tr>
<td>Magn. Limest.</td>
<td>(063°)</td>
<td>2.2 joints/m</td>
</tr>
<tr>
<td></td>
<td>(153°)</td>
<td>2.7 joints/m</td>
</tr>
</tbody>
</table>
SECTION XIV
COMPARISON OF LABORATORY, PHOTOS, AND FIELD MEASUREMENTS

14.1 VELOCITIES
The intact ground-mass velocities as estimated in section 10 do not show any correlation with the laboratory values. The quotients of the estimated joint densities are also not relatable to the quotients of the joint densities estimated on the photographs. The joint densities themself are, of course, not relatable, because the joint densities in formulas (10.1) and (10.9) are in joint volume per cubic meter of intact ground volume, while the joint densities measured on the photographs are in number of joints per meter length, without measurements of joint aperture. Accordingly, the correlation between velocity maxima and joint directions seems to be the only reliable one.

14.2 ATTENUATION
The fact that the K-factor seems to be constant for a particular ground-mass (under the restrictions summarized in section 11.5) makes it possible to use the K-factor for calculation of a static E modulus out of seismic wave parameters according to a visco-elastic model. Because a static E modulus of the whole ground-mass in the different quarries is not known (plate-bearing tests were not done), the only correlation which could be checked is the relationship between the static E modulus and the seismic E modulus calculated out of laboratory experiments.

\[ E(\text{stat}) \cdot f(\sqrt{\text{stat}}) = E(\text{seis}) \cdot f(\sqrt{\text{seis}}) \cdot K \]
\[ \rho \ V^2 = E(\text{seis}) \cdot f(\sqrt{\text{seis}}) \]

these give:

\[ E(\text{stat}) \cdot f(\sqrt{\text{stat}}) = \rho \ V^2 K \]
If is assumed that stat is between 0.2 and 0.4, what is likely for most rocks, then \( f(\sqrt{\text{stat}}) \) is between 1 and 2.
### Table 16.

<table>
<thead>
<tr>
<th></th>
<th>Static E modulus calculated with $E_{\text{stat}}$ = 1</th>
<th>E_{\text{stat}} = 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>$f(\sqrt{\text{stat}}) = 1$, $f(\sqrt{\text{stat}}) = 2$</td>
<td>laboratory</td>
</tr>
<tr>
<td></td>
<td>I</td>
<td>II</td>
</tr>
<tr>
<td>NCB-mine</td>
<td>10.887</td>
<td>7.7</td>
</tr>
<tr>
<td>Black Hill</td>
<td>10.447</td>
<td>1.2</td>
</tr>
<tr>
<td>Greehow Hill</td>
<td>10.891</td>
<td>8.6</td>
</tr>
<tr>
<td></td>
<td>10.586</td>
<td>8.6</td>
</tr>
<tr>
<td>Magnesium</td>
<td>10.738</td>
<td>0.92</td>
</tr>
<tr>
<td>Limestone</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

I are the static E moduli calculated without the K-factor
II are the static E moduli calculated with the K-factor

Calculations of the static E modulus with a $f(\sqrt{\text{stat}})$ of 1 and calculation the static E modulus with $f(\sqrt{\text{stat}})$ of 2 are listed in table 16.

As well are listed the static E moduli calculated without the K-factor for both $f(\sqrt{\text{stat}})$, and are listed the static E moduli determined in the laboratory by means of a compressive test.

As table 16 shows do the static E moduli determined from laboratory values fit better in the allowable range when the static E modulus has been calculated with the K-factor.

Using the K-factor out of the field measurements for calculation of a static E modulus on base of ultrasonic velocities, as is done above, means that is assumed that the K-factor for ultrasonic velocities through small samples is equal to the K-factor measured in the field, and thus that there is only a scale factor between the discontinuities in the sample and in the field. This will be, in general, only approximately true, so that the relationship as stated above, if there is any relationship at all, must be consid-
ered rather as an empirical relationship than as an exact relationship.
SECTION XV

CONCLUSIONS AND RECOMMENDATIONS

The main points of the investigation can be summarized as follows:

1. Velocity anisotropy reflects the joint directions and can be used to determine the joint directions.

2. Calculation or estimation of any other ground-mass parameter out of the velocity anisotropy may be possible in optimum situations, but can give completely wrong values in situations where either the jointing is too different from the "penny-shaped joints" for which the theoretical functions are developed, or in more complicated geological situations.

3. The K-factor seems to be significant for a particular ground-mass and seems to be independent of the joint directions if the joints are not too large. The ground-mass can then be considered to behave as visco-elastic model. This could mean as well that the static E modulus calculated out of seismic parameters according to a visco-elastic model is a reliable static E modulus for a ground-mass.

4. The K-factor is likely to be useful to determine large (open-) joint directions, because these disturb the visco-elastic system and will cause a minimum or maximum in the K-values.

Recommendations:

1. In this investigation it was not possible to use shearwaves, because a good signal generator was not available. It is likely that comparing shearwaves with P-wave parameters will increase the accuracy and make it possible to calculate the Lame constants.
2. Placing the geophones in lines gives the possibility that a series of geophones are exactly placed upon a joint or in the direct neighbourhood of a joint. The signal received by a geophone placed upon or in the neighbourhood will be different from the signal received by a geophone upon intact ground. When this happens for a whole line or for a large number of geophones along a line, the measured seismic parameters will be systematically different from other lines because of the phenomena described above. Placing the geophones randomly avoids this problem, but the calculations become a lot more difficult.

3. When the visco-elastic model describes the ground-mass exactly all coefficients of the ground-mass deformation tensor are known, and the elastic moduli in every direction (e.g. parallel or perpendicular to the surface) can be calculated. But when the anisotropy of the ground is too large for a visco-elastic model (e.g. large open joints) the coefficients of the ground-mass deformation tensor will depend on the measuring direction. In this latter case there is a difference in the deformation tensor calculated from refracted waves (parallel or under a certain angle with the surface) and the deformation tensor calculated from reflected waves (about perpendicular to the surface). Because this second one is more important for engineering purposes, it would be an improvement to use reflected waves.
Appendix A

FIRST ARRIVALS AGAINST DISTANCE
Appendix A.1; Arrival-time P-wave against distance, NCB-mine

fan-angle 112.5°

\[ V_2 = 1.60 \text{ km/s} \quad V_1 = 0.82 \text{ km/s} \]

\[ T_i = 0.99 \text{ ms} \]

fan-angle 135°

\[ V_2 = 1.50 \text{ km/s} \quad V_1 = 1.04 \text{ km/s} \]

\[ T_i = 0.64 \text{ ms} \]

fan-angle 157.5°

\[ V_2 = 1.35 \text{ km/s} \quad V_1 = 1.00 \text{ km/s} \]

\[ T_i = 0.85 \text{ ms} \]

fan-angle 180°

\[ V_2 = 1.17 \text{ km/s} \quad V_1 = 0.80 \text{ km/s} \]

\[ T_i = 0.74 \text{ ms} \]
Appendix A.1: Arrival-time P-wave against distance, NCB-mine

### Ti = 1.69 ms

- **Fan-angle**: 202.5°
- **V₂ = 1.42 km/s**
- **V₁ = 1.04 km/s**

### Ti = 0.94 ms

- **Fan-angle**: 225°
- **V₂ = 1.03 km/s**

### Ti = 1.01 ms

- **Fan-angle**: 247.5°
- **V₂ = 1.23 km/s**
- **V₁ = 1.01 km/s**

### Ti = 1.13 ms

- **Fan-angle**: 270°
- **V₂ = 1.47 km/s**
- **V₁ = 0.97 km/s**

---

Appendix A.2: Arrival-time P-wave against distance, Black Hill

fan-angle 330°

\[ V_1 = 1.34 \text{ km/s} \]

fan-angle 352.5°

\[ V_2 = 2.25 \text{ km/s} \]

fan-angle 015°

\[ V_2 = 2.40 \text{ km/s} \]

[Graphs showing travel times and distances for each fan-angle]
Appendix A.2; Arrival-time against distance, Black Hill

- Fan-angle 0°:
  - \( V_2 = 1.62 \text{ km/s} \)
  - \( T_1 = 2.42 \text{ ms} \)
  - \( V_1 = 0.43 \text{ km/s} \)

- Fan-angle 0.5°:
  - \( V_2 = 1.84 \text{ km/s} \)
  - \( T_1 = 3.76 \text{ ms} \)
  - \( V_1 = 0.26 \text{ km/s} \)

- Fan-angle 0°:
  - \( V_2 = 3.65 \text{ km/s} \)
  - \( T_1 = 5.70 \text{ ms} \)
  - \( V_1 = 0.27 \text{ km/s} \)
Appendix A.2: Arrival-time P-wave against distance, Black Hill

fan-angle 092.5°  
$V_2 = 2.63 \text{ km/s}$  
$T_1 = 4.27 \text{ ms}$  
$(V_1 = 0.23 \text{ km/s})$

fan-angle 115°  
$V_2 = 1.73 \text{ km/s}$  
$T_1 = 1.73 \text{ ms}$  
$(V_1 = 0.43 \text{ km/s})$

fan-angle 137.5°  
$V_2 = 1.53 \text{ km/s}$  
$T_1 = 1.03 \text{ ms}$  
$(V_1 = 0.67 \text{ km/s})$

Appendix A.3, Arrival-time P-wave against distance, Greneway Hill

\[ V_3 = 2.37 \text{ km/s} \]

Time (ms) vs. Distance (m)

\[ T_{i3} = 3.63 \text{ ms} \]
\[ T_{i2} = 2.37 \text{ ms} \]

Fan-angle 217.5°

\[ V_2 = 1.71 \text{ km/s} \]

Time (ms) vs. Distance (m)

\[ T_i = 1.21 \text{ ms} \]

Fan-angle 240°

\[ V = 1.48 \text{ km/s} \]

Time (ms) vs. Distance (m)

\[ T_{i3} = 4.03 \text{ ms} \]

Fan-angle 262.5°

\[ V_3 = 3.23 \text{ km/s} \]
\[ V_1 = 1.08 \text{ km/s} \]
Appendix A.3. Arrival time P-wave against distance, Greathow Hill

\[ V_2 = 1.66 \text{ km/s} \]
\[ V_1 = 0.67 \text{ km/s} \]
\[ V_3 = 3.08 \text{ km/s} \]
\[ T_{12} = 0.90 \text{ ms} \]
\[ T_{13} = 3.50 \text{ ms} \]

\[ V_2 = 1.78 \text{ km/s} \]
\[ V_3 = 3.70 \text{ km/s} \]
\[ T_{12} = 1.31 \text{ ms} \]
\[ T_{13} = 3.24 \text{ ms} \]

\[ V_2 = 1.63 \text{ km/s} \]
\[ V_3 = 2.96 \text{ km/s} \]
\[ T_{12} = 1.16 \text{ ms} \]
\[ T_{13} = 3.09 \text{ ms} \]
Appendix A.3; Arrival-time P-wave against distance, Greehow Hill

Time (ms)

Distance (m)

Fan-angle 352.5°

\[ V_1 = 0.65 \text{ km/s} \]

\[ V_3 = 2.37 \text{ km/s} \]

Fan-angle 015

\[ V_1 = 0.65 \text{ km/s} \]

\[ V_3 = 4.00 \text{ km/s} \]
Appendix A.4; Arrival-time against distance, Magnesium Limestone quarry

- Diagrams showing arrival times and fan angles for different distances.

Appendix A.4: Arrival time against distance, Magnesium Limestone quarry

![Graphs showing arrival time against distance for different fan angles.]

- **Fan-angle 063°**
  - $V_1 = 0.34 \text{ km/s}$
  - $V_2 = 0.41 \text{ km/s}$
  - $V_3 = 0.54 \text{ km/s}$
  - $T_{13} = 8.22 \text{ ms}$
  - $T_{12} = 2.55 \text{ ms}$

- **Fan-angle 085.5°**
  - $V_1 = 0.35 \text{ km/s}$
  - $V_2 = 0.47 \text{ km/s}$
  - $V_3 = 0.53 \text{ km/s}$
  - $T_{13} = 12.7 \text{ ms}$
  - $T_{12} = 0.55 \text{ ms}$

- **Fan-angle 108°**
  - $V_1 = 0.34 \text{ km/s}$
  - $V_2 = 0.47 \text{ km/s}$
  - $V_3 = 0.66 \text{ km/s}$
  - $T_{13} = 7.96 \text{ ms}$
  - $T_{12} = 2.38 \text{ ms}$

Appendix A.4; Arrival-time against distance, Magnesium limestone quarry

Fan-angle $130.5^\circ$

- $V_1 = 0.34 \text{ km/s}$
- $T_{12} = 2.77 \text{ ms}$
- $T_{13} = 9.44 \text{ ms}$
- $V_2 = 0.52 \text{ km/s}$
- $V_3 = 0.88 \text{ km/s}$

Fan-angle $153^\circ$

- $V_1 = 0.26 \text{ km/s}$
- $T_{12} = 4.65 \text{ ms}$
- $T_{13} = 8.03 \text{ ms}$
- $V_2 = 0.67 \text{ km/s}$
- $V_3 = 0.93 \text{ km/s}$
Appendix A5.1: Arrival-time against distance, Claypit

Fan-angle 090°

\[ V = 1.61 \text{ km/s} \]

\[ T_1 = 3.29 \text{ ms} \]

Fan-angle 113°

\[ V = 1.55 \text{ km/s} \]

\[ T_1 = 2.78 \text{ ms} \]

Fan-angle 135°

\[ V = 1.65 \text{ km/s} \]

\[ T_1 = 3.36 \text{ ms} \]
Appendix A.5.1: Arrival-time against distance, Claypit

- Fan-angle 158°
  - Time: 3.16 ms
  - Velocity: 1.63 km/s

- Fan-angle 180°
  - Time: 2.42 ms
  - Velocity: 1.49 km/s

- Fan-angle 203°
  - Time: 3.39 ms
  - Velocity: 1.65 km/s

---

Appendix A 5.1. Arrival-time against distance, Claypit

- Fan-angle 225°
  - $V = 1.67 \text{ km/s}$
  - $T_i = 3.32 \text{ ms}$

- Fan-angle 248°
  - $V = 1.54 \text{ km/s}$
  - $T_i = 2.81 \text{ ms}$

- Fan-angle 270°
  - $V = 1.58 \text{ km/s}$
  - $T_i = 3.32 \text{ ms}$
Appendix A.5.2 Arrival-time against distance, long-distance line, Claypit

\[ V_2 = 1.55 \text{ km/s} \quad V_3 = 3.13 \text{ km/s} \]

\[ T_{i1} = 2.63 \text{ ms} \quad T_{i2} = 17.44 \text{ ms} \]

\[ T_{i1} = 1.85 \text{ ms} \quad T_{i2} = 17.23 \text{ ms} \]
Appendix B

ENERGY VALUES

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### Appendix B.1; Claypit

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Mean and standard deviation are calculated without the values in brackets.
Appendix B.3; Black Hill quarry

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Appendix B.4: Magnesium limestone quarry, 1\textsuperscript{st}+2\textsuperscript{nd} layer separately

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Mean and standard deviation are calculated without the values in brackets.
### Appendix B.4: Magnesium limestone quarry, both layers

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<td>0.97</td>
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<td>153</td>
<td>0.280</td>
<td>32.8</td>
<td>165</td>
<td>0.829</td>
<td>0.118</td>
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<td>0.352</td>
<td>32.3</td>
<td>165</td>
<td>0.738</td>
<td>0.110</td>
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<tr>
<td>St.dev.</td>
<td>0.083</td>
<td>0.5</td>
<td>18</td>
<td>0.155</td>
<td>0.065</td>
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</table>
Appendix B.5; Greehow Hill quarry

<table>
<thead>
<tr>
<th>fan-angle</th>
<th>a</th>
<th>ln E₀</th>
<th>f</th>
<th>K</th>
<th>Eₚstat</th>
<th>corr. coef.</th>
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</thead>
<tbody>
<tr>
<td>degrees</td>
<td>1/m</td>
<td>Hz</td>
<td></td>
<td></td>
<td>MN/m²</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>I</td>
<td>II</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>I</td>
<td>II</td>
<td></td>
<td></td>
</tr>
<tr>
<td>217</td>
<td>0.299</td>
<td>30.0</td>
<td>306</td>
<td>0.810</td>
<td>0.670</td>
<td>0.628</td>
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<tr>
<td>240</td>
<td>0.298</td>
<td>31.1</td>
<td>321</td>
<td>0.867</td>
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<td>0.503</td>
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<tr>
<td>262</td>
<td>0.387</td>
<td>32.6</td>
<td>336</td>
<td>0.890</td>
<td>0.356</td>
<td>0.275</td>
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<tr>
<td>285</td>
<td>0.274</td>
<td>33.3</td>
<td>386</td>
<td>0.826</td>
<td>0.523</td>
<td>0.603</td>
</tr>
<tr>
<td>307</td>
<td>0.126</td>
<td>23.5</td>
<td>325</td>
<td>0.964</td>
<td>0.855</td>
<td>0.810</td>
</tr>
<tr>
<td>330</td>
<td>0.234</td>
<td>30.9</td>
<td>348</td>
<td>0.913</td>
<td>0.744</td>
<td>0.643</td>
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<tr>
<td>352</td>
<td>0.164</td>
<td>28.2</td>
<td>576</td>
<td>0.966</td>
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<td>1.438</td>
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<tr>
<td>015</td>
<td>0.314</td>
<td>32.2</td>
<td>344</td>
<td>0.370</td>
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<td>1.569</td>
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<tr>
<td>mean</td>
<td>0.262</td>
<td>30.2</td>
<td>355</td>
<td>0.891</td>
<td>0.586</td>
<td>0.700</td>
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<tr>
<td>St. dev.</td>
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<td>3.2</td>
<td>91</td>
<td>0.062</td>
<td>0.204</td>
<td>0.364</td>
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</table>

Appendix C

SHEAR AND TRIAXAL TESTS

<table>
<thead>
<tr>
<th>Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>Value</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>
RESULTS SHEARTEST

For all points: \( \sigma_{sh} = 36.11 + 0.195 \sigma_n \);
without the encircled points:
\( \sigma_{sh} = 51.40 + 0.0343 \sigma_n \)

\( \sigma_{shear} \) (kN/m²)

<table>
<thead>
<tr>
<th>depth</th>
<th>2.4 m</th>
<th>4.7 m</th>
<th>7.5 m</th>
<th>9.4 m</th>
</tr>
</thead>
</table>

51.40 kN/m²

\( \sigma_{normal} \) (kN/m²)
SAMPLE DEPTH: 2.4 m

- $\sigma_{\text{nor}} = 0.05882 \text{ MN/m}^2$
- $\sigma_{\text{nor}} = 0.05882 \text{ MN/m}^2$
- $\sigma_{\text{nor}} = 0.02941 \text{ MN/m}^2$
- $\sigma_{\text{nor}} = 0.05882 \text{ MN/m}^2$

Shear force at failure

Shear area: $3.025 \times 10^{-3} \text{ m}^2$

Displacement: $3 \text{ mm}$
SAMPLE DEPTH 4.7 m

- $\sigma_{nor} = 0.19451 \text{ MN/m}^2$
- $\sigma_{nor} = 0.09726 \text{ MN/m}^2$

--- Shear force at failure

Shear area $3.025 \times 10^{-3} \text{ m}^2$

Displacement $\frac{1}{(\text{mm})}$
Shear force at failure

Sample depth 7.5 m

$\sigma_{\text{nor}} = 0.11764 \text{ MN/m}^2$

$\sigma_{\text{nor}} = 0.16210 \text{ MN/m}^2$

Shear area $3.025 \times 10^{-3} \text{ m}^2$
SAMPLE DEPTH 9.4 m

\[
\sigma_{nor} = 0.19451 \text{ MN/m}^2
\]

\[
\sigma_{nor} = 0.16210 \text{ MN/m}^2
\]

Shearforce at failure

Shear area 3.025 \times 10^{-3} \text{ m}^2
Appendix C.2; triaxial test
Appendix C.2; triaxial test
Appendix G.2; triaxal test
Appendix C.2; triaxal test

Appendix D

PHOTO INTERPRETATION
Appendix D.1; photo interpretation, NCB-mine

photo direction: 180°
joint density direction 112°: 1.8 joints/m
joint density direction 205°: 1.6 joints/m
Appendix D.2: photo interpretation, Black Hill quarry

photo direction $250^\circ$
joint density direction $000^\circ$: 2.1 joints/m
$070^\circ$: 1.5 joints/m
Appendix D.3; photo interpretation, Greehow Hill quarry

photo direction: 285°
joint density direction: 285°
5.9 joints/m

photo direction: 015°
joint density direction: 015°
2.1 joints/m
Appendix D.4; photo interpretation, Magnesium Limestone quarry

photo direction: $243^\circ$
joint density direction: $063^\circ$
2.2 joints/m

photo direction: $333^\circ$
joint density direction: $153^\circ$
2.7 joints/m


BIBLIOGRAPHY


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