

STATIC AND DYNAMIC ELASTIC MODULUS TESTING
OF CONCRETE AND ITS CONSTITUENTS AND
COMPARISON OF RESULTS WITH THEORETICAL
MODELS

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REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE
IN ENGINEERING.

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DECLARATION

I, Frank Grills, declare that this dissertation is my own unaided work. It is being submitted for the degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

A handwritten signature in cursive script that reads "Frank Grills". The signature is written in dark ink and is positioned above a horizontal line.

Frank Grills

10TH day of JULY, 1986

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INTRODUCTION

There is a need to correlate concrete elastic modulus values derived from established theoretical formulae with actual experimental results. This project report examines such correlations. Experimental work involved the measurement of the elastic moduli (E-values) of the various concrete phases i.e. the aggregate, hardened cement paste, mortar, and the composite concrete. In most codes of practice the modulus of elasticity is generally related to the compressive strength. However, factors affecting elastic modulus do not always have a corresponding effect on strength. For example, an aggregate with a higher elastic modulus does not necessarily produce a concrete of greater strength although it will increase the modulus of the concrete.

A major portion of the work reported here was the development of static and dynamic apparatus for achieving accurate estimates of E-values for the various phases. This included the design and manufacture of moulds and setting/hardening apparatus for cement pastes and mortars. The setting/hardening apparatus was designed to ensure uniformity and hence prevent segregation during the setting of specimens. The properties of each material phase are first dealt with by a broad background study and literature survey, followed by results of a laboratory programme to ascertain elastic modulus, both statically and dynamically. Rock cylinder cores were tested for the aggregate E-values; prisms were used for paste, mortar and concrete E-values.

Static apparatus was developed to investigate autographic measuring systems to replace manual recording methods. Ultrasonic apparatus was used to determine dynamic E-values for rock cores. Dynamic apparatus was developed with instrumentation similar to BS 1881: part 5: 1970¹ to measure the dynamic modulus of pastes, mortars and concretes. A selection of

standard mixes was chosen and two local "homogeneous, isotropic" aggregates, one with a moderate E-value (granite) and one with a high E-value (andesite) were selected. (Details of the aggregates are given later in this introduction). The effects of aggregate volume concentration and aggregate moduli were investigated to verify experimental values with theory.

In the two-phase model approach for estimating elastic modulus, the concrete is viewed as a composite material composed of different phases which are chemically and mechanically distinct and are separated by definite interfaces. Hence the structure of the concrete can be modelled mathematically by assuming that the paste, mortar and aggregate phases are each homogeneous and isotropic. This is a reasonable assumption at macroscopic level². Ultimately calibration of a suitable theoretical model for estimating E-values of concrete for different aggregate types is envisaged. Once calibrated, the model could be used to check experimental results. It would also afford designers the opportunity to estimate concrete E-values from a knowledge of the constituent materials.

In this project, no single theoretical equation predicted the E-value of the concrete with consistent accuracy. Although the models have been tested and found reliable³, many problems remain unresolved with respect to estimating the elastic modulus by means of a two-phase model. The main drawback is the lack of data necessary to estimate the different phase stiffnesses. Nevertheless, where data is available the models can be used as a powerful engineering tool in examining the influence of phase stiffness and mix proportions on concrete elastic modulus.

DISTRIBUTION AND LITHOLOGY OF AGGREGATES

Granite Aggregate

The granite aggregate used in the project was received from Jukskei quarry, Halfway House. The granite is part of the Basement Complex, more specifically the Johannesburg-Pretoria granite inlier⁴.

It is a medium even-grained granite characterised by the presence of irregularly developed coarse grained pignatitic areas. The chief minerals are microcline, sodic plagioclase and quartz. These minerals are present in approximately equal amounts and together constitute 90-95% of the rock. Additional phases present in minor amounts are biotite, hornblende, magnetite and zircon. Also present in the rock assemblage are patches and veins of quartz biotite schist and amphibolite⁵. The granite stone and sand particles tended to be rounded to sub-angular in shape, with a relative density (R.D.) of 2,65. The granite crusher sand had a fineness modulus (F.M.) of 3,6.

Andesite Aggregate

The andesite aggregate studied in this project was received from the Eikenhof quarry which lies in the Larjgeleven Formation⁶ approximately 25 km south of Johannesburg. The rock has a dark, greenish-grey colour and is highly amygdaloidal in places. Table I-1 shows the mineralogical composition of the andesite from the Eikenhof quarry⁶.

Both the andesite stone and sand particles have an angular to sub-angular shape. The andesite crusher sand was found to have a fineness modulus (F.M.) of 3,5 and a relative density (R.D.) of 2,89.

Table I-1 Mineralogical Composition of the Andesite from Eikenhof Quarry⁶

<u>MINERAL</u>	<u>% BY VOLUME</u>
AMPHIBOLE (most important mineral))	75
ALBITE)	
PYROXENE	2
CHLORITE	8
EPIDOTE)	
RUTILE)	5
OPAQUE MINERALS (possibly MAGNETITE))	
FREE QUARTZ IN VEINS	10

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CHAPTER 1 DYNAMIC AND STATIC ELASTIC PARAMETERS FOR GRANITE AND ANDESITE ROCKS

1.1 INTRODUCTION

The main objectives of this chapter are:

- 1) To assess the engineering properties of the granite and andesite rocks in order subsequently to determine the elastic modulus of granite and andesite concrete by theoretical models.
- 2) To correlate measured static and dynamic moduli.

A brief literature survey is first presented outlining the factors affecting the modulus of elasticity of the rocks. The laboratory procedure employed to determine the static and dynamic engineering properties and discussion of the results then follows.

1.2 BACKGROUND

Recognition of the discrepancy between statically and dynamically determined elastic moduli of rock was first reported by Zisman¹ in 1933. He tested a granite using a resonant frequency method at atmospheric pressure and found that the dynamic elastic moduli E_d were 20 per cent greater than the static values E_s . In 1936 Ide² also adopted this method of measuring the dynamic modulus. He compared E_d from resonance of bars with E_s measured in static compression tests. The dynamic value ranged from 4 to 87 per cent higher than the static. He found a mean difference of around 20 per cent, similar to Zisman.

In general, the dynamic elastic moduli have been found to be significantly greater than the static values, the discrepancy for 'soft' rocks, such as sandstone, being greater than for

'hard' rocks, such as granite.³ Reinhart³ found that the difference between statically and dynamically measured elastic moduli was generally less than 40 per cent for rocks having a high elastic modulus (approximately 50 GPa) and more than 100 per cent for rocks with a low elastic modulus (15 GPa) with the dynamic value being greater in each case. The difference is even larger for fractured rocks. Simmons and Brace⁴ also reported that at atmospheric pressure a discrepancy of several hundred per cent may exist between static and dynamic values. This can be attributed partly to experimental error, where an uncertainty of 10 or 20 per cent in an experimental value may lead to a 50 per cent difference in the static and dynamic values. A more likely source of the discrepancy in low-pressure values is the cracks which are generally present in most crystalline rocks.

It should be noted that two methods are usually used to determine dynamic elastic constants:

- (i) the resonance method, and
- (ii) the ultrasonic pulse method i.e. measurement of pulse velocities above the audible frequency range.

The wave velocities most commonly determined for these purposes are longitudinal and torsional wave velocities by the resonance method or compression and shear wave velocities by the pulse method. If the material under test is homogeneous, isotropic and elastic, elastic constants determined by either resonance or pulse methods should yield equivalent results. The dynamic method used in this report for rock cores was the ultrasonic pulse method.

The discrepancies between E_d and E_s have been widely attributed to microcracks and pores, though the situation is complicated by the fact that results are dependent on both rock microstructure and level of confinement during tests and that both E_d and E_s are affected differently. Walsh⁵ found

that the elastic modulus of an elastic solid containing cracks was less than that for an identical solid without cracks. An isotropic solid filled with a low concentration of planar elliptical cracks was used as a model. This was an idealization for most rocks, which are collections of grains individually exhibiting elastic anisotropic behaviour. He stated that although the deformation on the scale of individual grains is decidedly anisotropic, behaviour of a large group of grains may be considered isotropic if the grains have no preferred orientation. The cracks also must have a random orientation if the rock is to be isotropic.

At low stresses, cracks in fractured rock are open. As stress is increased cracks close and the rock becomes elastically stiffer, i.e. the elastic modulus increases with stress. As most cracks in rock are flat at low confining pressures of 2-3 kPa, these cracks are closed or largely eliminated.⁶ Above this stress no change in stiffness due to crack closure occurs and the stress-strain curve is linear. The increase in E_d with increasing stress can be attributed to the closing under stress of cracks within the rock, since elastic waves increase in velocity as the material becomes compacted. Volarovich⁷ also found that at a stress of 1 kPa the longitudinal velocity in gneisses increased by 10-12 per cent and the shear wave velocity by 7-9 per cent.

Walsh⁸ also showed that even when the applied stress was sufficient to close all cracks, the modulus would still not be equal to that of a solid material, since frictional sliding at the crack faces can occur. He showed that cracks that had experienced sliding did not immediately slide in the opposite sense when the confining stress was lowered. Qualitatively he explained the lack of agreement between static and dynamic values of elastic modulus, even though measurements were taken at the same confining stress. The passage of a sound wave corresponds to the superposition of a small alternating stress on the existing applied stress (see figure 1.1). The static value is taken as

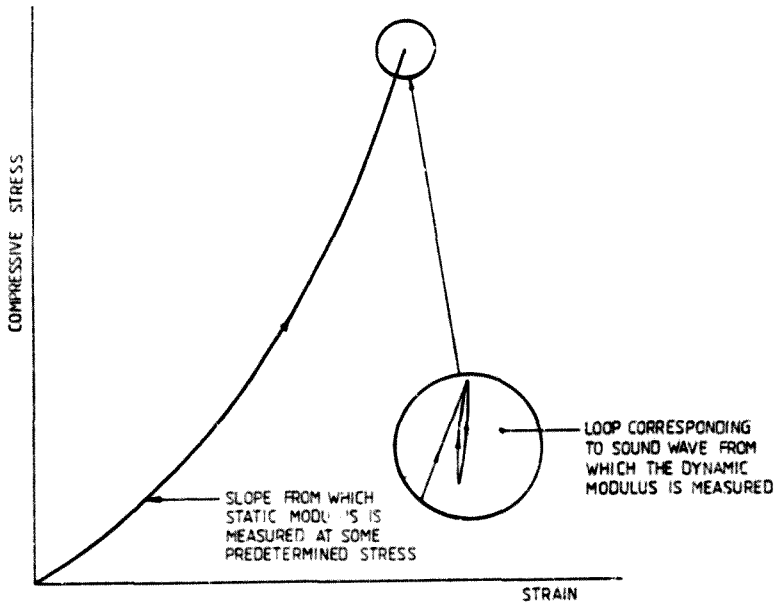


Figure 1.1 Static stress-strain curve with superposition of sound wave for rock (after Walsh⁸)

the slope of the stress-strain curve at a selected value of applied stress. Conversely the modulus calculated from the sonic velocity corresponds to the average slope of the loop representing the sound wave. Thus the dynamic sonic value would be expected to be somewhat higher than the static value.

This explanation is also consistent with the observation of Anderson et al⁹ that the increase in E_d with increasing stress is more marked in the direction of the major principal stress and more pronounced at low stresses. They found that the difference between static and dynamic constants varied from 0 to 300 per cent. The difference between static and dynamic constants was explained qualitatively by Zisman¹⁰ and Ide¹¹ who stated that a wave pulse or packet of energy passing through a rock suffers a loss of energy due to reflection and refraction on entering a cavity (crack, pore) at the air/rock - rock/air interfaces. A small amount of energy is transmitted across the boundary and a large amount is scattered around the cavities. However, as the cavities form a random array in the rock the scattered energy is of negligible importance and the pulse passes through the rock largely unaffected.

The more compact the rock i.e. the less cracks and fissures, the closer the static and dynamic constants agree as evidenced in the following table:

Table 1.1 Static and dynamic constants of Quincy granite and Sudbury norite (after Reinhart et al³)

	MODULUS OF ELASTICITY		POISSON'S RATIO	
	STATIC	DYNAMIC	STATIC	DYNAMIC
QUINCY GRANITE	35	43	0,10	0,33
SUDBURY NORITE	83,6	88,2	0,22	0,27

The Sudbury norite is more compact than the Quincy granite as indicated by the much higher value of elastic modulus and the closer agreement of the static and dynamic values¹².

Reinhart et al¹² also found that when the confining pressure increased from 0 to 0.5 kPa, velocities increased by 10 to 30 per cent of their initial value. For pressure increase from 0.5 to 2.0 kPa the velocities increased from 0.3 to 2.0 per cent per 1 kPa. Generally the propagation velocities in rock increase as pressure increases. Figure 1.2 shows the different methods of applying pressure or stress to a specimen. The results of the different methods do not vary much and results are comparable. Also when uniaxial stress was applied to a rock the velocity in a direction parallel to the stress was approximately 10 per cent higher than the velocity in a direction perpendicular to the stress.

Howarth¹³ investigated this difference between static and dynamic constants by using an apparatus which measured both moduli 'simultaneously'. This effectively eliminated problems and inaccuracies arising from hysteresis effects. The elastic hysteresis of crystalline rocks can also be explained by the presence of cracks and the effect of friction between crack surfaces¹⁴. Howarth found that under triaxial conditions the differences between E_d and E_s are smaller at high confining pressures than at low confining pressures. Circumstantial evidence suggests that microcracks and microfissures in the rocks are responsible for this phenomenon.

A study carried out by the United States Bureau of Reclamation¹⁵ in 1953 found that the ratio of dynamic to static moduli may vary between 0.85 and 2.9. The discrepancy between static and dynamic values is less for rocks which have a greater elastic modulus. This fact highlights the assumption that the equations relating elastic modulus to velocity are based on the consideration that the rock is homogeneous and isotropic. Most rocks do not meet these assumptions perfectly.

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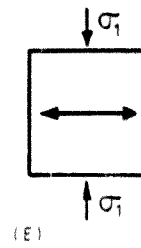
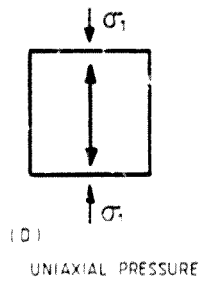
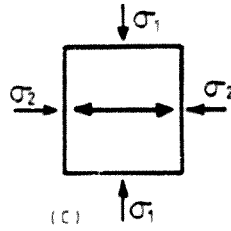
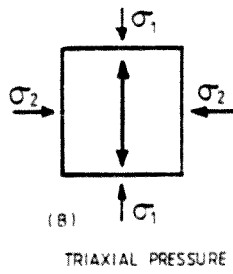
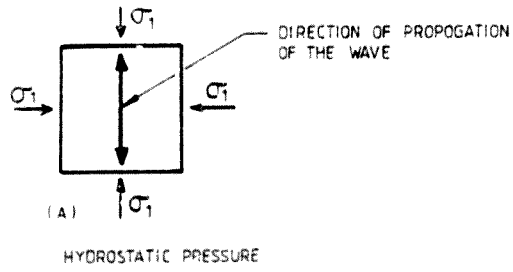


FIGURE 1.2 DIFFERENT PRESSURE SYSTEM APPLIED TO ROCKS FOR ULTRASONIC TESTING (after Reinhart et al. 12)

The rocks used in this project were tested by conventional static and ultrasonic pulse dynamic methods. The rock cylinders were tested uniaxially to obtain the static modulus. No confining stress was applied to the rocks during the ultrasonic test. The specimens could be expected to contain microcracks and microfissures resulting from the blasting procedure at the quarry. Therefore, we would expect differences between the static and dynamic constants of each rock type for the reasons discussed above.

1.3 ELASTICITY OF AGGREGATES

Elasticity is a property of an ideal material. The degree to which materials including rocks approximate to the ideal depends on three major factors:

- a) isotropy
- b) homogeneity
- c) continuity

Isotropy is a measure of the directional properties of the material. In a statistical sense, a granular body will be isotropic if all its grains have random orientation, and a plane of given dimension intersecting the body in any direction exposes an equal number of grains. Thus since most rocks have a preferred particle and crystal orientation, they are anisotropic i.e. react differently to forces in different directions depending on the degree of anisotropy.

Homogeneity is a measure of the physical uniformity of the body. The constituents of a homogeneous material are distributed so that a minute fragment cut from any part of the body would be representative of the whole body. Homogeneity is also largely dependent upon scale.

Continuity is taken to refer to the amount of joint, crack and pore space in a particular rock body. The continuity affects the cohesion and hence the transmission of stress throughout the body.

From the above definitions it is possible to arrive at a rough estimation of the probable elasticity of a rock. In fact all rocks, to some extent, are anisotropic, inhomogeneous and discontinuous. No rock is perfectly elastic. However, some aggregates for practical purposes exhibit elastic properties particularly under low stresses¹⁶.

Most fine grained massive and compact rocks may be regarded as practically elastic. They approximate to a brittle elastic material having a near linear stress/strain relationship to failure and are the so-called quasi-elastic rocks. Coarse grained rocks and fine grained compacted sediments having a low porosity and a reasonable amount of cohesion are termed semi-elastic rocks. Their degree of elasticity is less than that of the fine grained rocks mentioned above. Their stress/strain curve is curvilinear, the slope of the curve decreasing with increasing stress. Lastly, rocks with lower cohesion and with large pore spaces, comprising mainly the weaker sedimentary rocks, are non-elastic and exhibit variable stress/strain characteristics. Typical stress/strain relationships for rocks are shown in figure 1.3.

As far as was practically possible the two aggregates used in this project were chosen to comply with the definition of an ideal elastic aggregate. The rock cores were selected with care to ensure as homogenous, joint and crack free samples as possible.

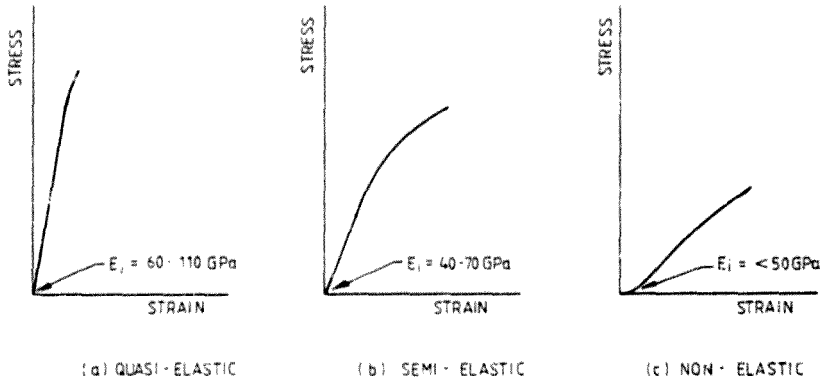


Figure 1.3 Typical stress/strain curves for rock
(after Farmer¹⁶ et al.)

1.4 EXPRESSIONS FOR DYNAMIC ELASTIC CONSTANTS OF ROCK

The dynamic elastic constants of a solid material can be determined indirectly by measuring propagation velocities in the material. For an isotropic solid there are two types of free-medium waves:-

- a) longitudinal or compressional waves which travel with a velocity V_p , and
- b) shear or transverse waves which travel with a velocity V_s .

These velocities are ideally related to the elastic constants by the following equations¹⁷:

$$E_d = \frac{\rho V_s^2 (3V_p^2 - 4V_s^2)}{(V_p^2 - V_s^2)} \quad \dots(1.1)$$

$$G_d = \rho V_s^2 \quad \dots(1.2)$$

$$\nu_d = \frac{(V_p^2 - 2V_s^2)}{2(V_p^2 - V_s^2)} \quad \dots(1.3)$$

where E_d is the dynamic modulus of elasticity (Pa)

G_d is the dynamic modulus of rigidity (Pa)

ν_d is the dynamic Poisson's ratio

ρ is the unit weight of material (kg/m^3)

V_p, V_s have units of m/s

It should be noted that the dynamic elastic constant G_d can be determined from V_s without a knowledge of ν_d .

The relationship for ν_d in (1.3) is obtained from:

$$G_d = \frac{E_d}{2(1 + \nu_d)} \quad \dots (1.4)$$

It is important to note that the above equations are strictly valid only if the material is isotropic, homogeneous and linear-elastic.

1.5 STATIC ELASTIC CONSTANTS OF ROCK

To define any material elastically, two elastic constants are required from the five available:

- a) Modulus of elasticity, E
- b) Poissons ratio, ν
- c) Modulus of rigidity, G
- d) Bulk modulus, K
- e) Lamé's constant, λ

In engineering rock problems E and ν are the most commonly quoted. The three types of rock previously defined in terms of their relative anelasticity as quasi, semi and non-elastic can also be roughly delineated in terms of their apparent elastic moduli. Thus in general a quasi-elastic rock will have an E value between 60GPa - 110GPa, a semi-elastic rock between 40-70GPa and a non-elastic rock less than 50GPa. The values quoted in each case are the initial tangent moduli.

Poisson's ratio for many materials is between 0,15 and 0,35 and is often assumed equal to 0,25¹⁷. In any work involving elastic analysis of rocks, there is adequate evidence available to suggest that a value of 0,25 for Poisson's ratio should be assumed unless there is considerable evidence to the contrary¹⁸. Table 1.2 lists values of static modulus of elasticity and Poissons's ratio for various rock types.

Table 1.2 Initial Tangent Values of Elastic Constants for Rocks (after Farmer et al)¹⁸

ROCK	E_s (GPa)	ν
Granite	20 - 60	0,25
Microgranite	30 - 80	0,25
Syenite	60 - 80	0,25
Diorite	70 - 100	0,25
Dolerite	80 - 110	0,25
Gabbro	70 - 110	0,25
Basalt	60 - 100	0,25

where E_s is the static elastic modulus of elasticity of the rock.

Details of the lithology and distribution of the two rock types tested in the present work have been given previously on page 3 of the Introduction. The rock types were chosen so as to satisfy the overall objective of the investigation, which was to compare the elastic moduli of concretes predicted from theoretical models using the elastic constants of its constituents with experimentally determined values.

Concrete is a multiphase material with a number of constituents including unhydrated cement, cement gel, water, air, and fine and coarse aggregate. The elastic modulus of the concrete, E_c , is a function of the elasticity of the aggregate, E_a , the elasticity of the paste, E_p , and their relative volume concentrations. These factors will be discussed in greater detail in chapter 3. Generally, the modulus of elasticity of concrete increases with increase in modulus of aggregate. In ordinary concrete, for which $E_a > E_p$, the modulus of the concrete also increases with increase in the concentration of aggregate. Therefore, for equal mix proportions, a concrete made with a relatively low modulus aggregate (granite in this case) and another with a relatively high modulus aggregate (andesite in this case) should result in the latter concrete having a higher modulus than the former. Granite and andesite were chosen for these reasons and the other factors detailed in section 1.2, i.e. homogeneity and uniformity of specimens. These conditions were met as far as was practically and economically possible for the project.

1.6 LABORATORY PROCEDURE FOR PREPARATION OF CORES

A total of 42 rock cores was tested, 21 cores from each rock type. The cylindrical cores were nominally 42mm diameter and had a minimum aspect ratio (length to diameter ratio) of 2:1. The granite cores were cut from boulders approximately 300 x 300 x 250 mm. However, due to the hardness of the andesite rock, the andesite cores were drilled by an outside rock

research organisation as the university laboratory equipment was inadequate. Three boulders per rock type were used to obtain the test cores. The granite boulders were :- JA (Juskei granite boulder A) from which ten cores were cut, JC (Juskei granite boulder C) from which four cores were cut and JF (Juskei granite boulder F) from which seven cores were cut. The andesite boulders were :- AA (andesite boulder A) from which seven cores were cut, AB (andesite boulder B) from which seven cores were cut and AC (andesite boulder C) from which seven cores were cut. The granite rock was somewhat variable and to obtain the number of cores essential for testing, more cores were drilled from some boulders than others.

For the detailed laboratory procedure applied to the granite and andesite cores refer to APPENDIX A.

It should be noted that the rock boulders for both aggregate sources were selected from the same location in their respective quarries as the crushed aggregate used in the concrete mixes. This ensured that the elastic constants derived from the rock cores would give a true reflection of the elastic constants of the aggregates used. Considerable differences could arise if the rock boulders and aggregates were taken from different locations in the quarries.

1.6.1 Drilling Procedure and Preparation of Cores

The rock boulders were mounted on a Matheys vertical coring machine, (see Figure 1.4). The drill bit and barrel used was type NX. (Length 200mm, internal diameter 42mm).

Successive cores were cut from each boulder and sawn to length using a Diamant Boart DV20 saw, such that the unfinished cores had an aspect ratio of not less than 2:1. Hence, the length of the finished cores equalled or exceeded 81mm. Approximately one millimetre extra was allowed in the length of the specimen to

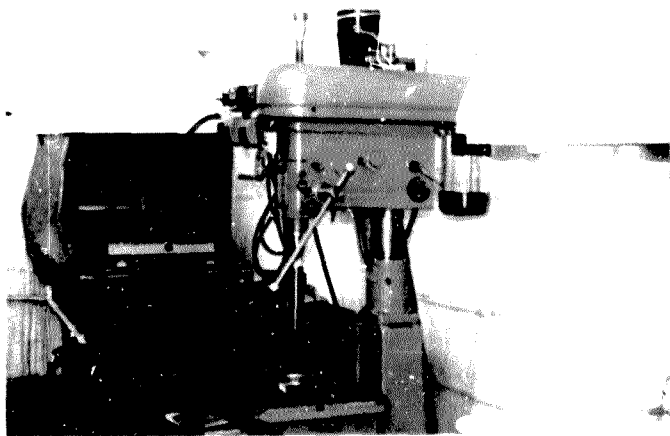


Figure 1.4 Vertical Coring Machine used to Core Rock Specimens

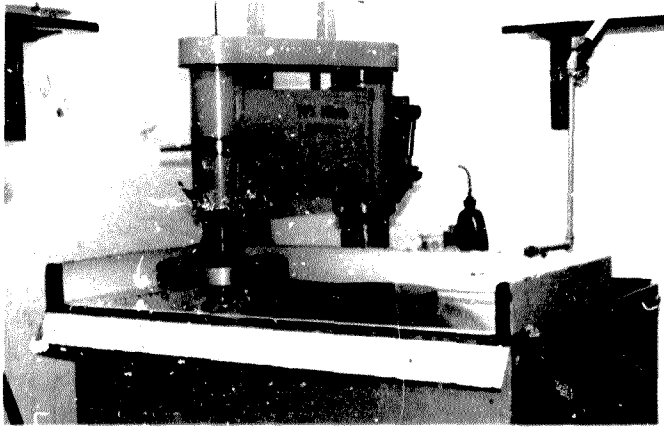


Figure 1.5 Lapping machine used to Grind End Faces of Rock Cores

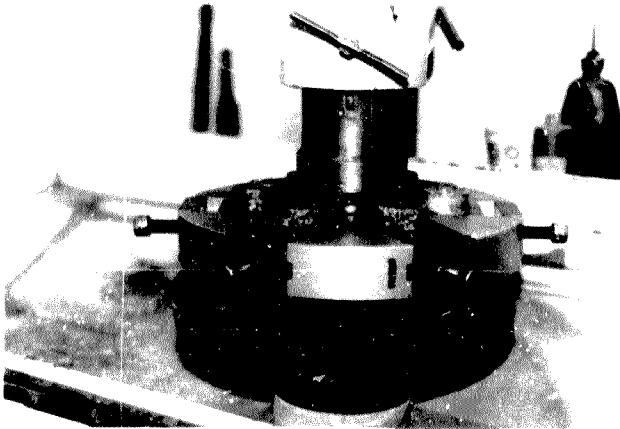


Figure 1.6 Lapping disc and granite cores

compensate for lapping and fine grinding of the specimen ends. The ends of the cores were ground flush and parallel by means of a lapping machine shown in figure 1.5. Figure 1.6 shows specimens held in the lapping disc. The specimens were ground flush by means of fine silicon abrasive filings lubricated by water.

1.7 LABORATORY MEASUREMENT OF DYNAMIC ELASTIC CONSTANTS

The following section describes the equipment and procedures used to measure the pulse velocities of compression and shear waves in the rock cores, and hence the determination of dynamic elastic constants.

The dynamic modulus values frequently do not agree with those determined statically, as discussed in section 1.2. However, the sonic evaluation of the properties are useful for the preliminary prediction of static properties. Dynamic constants measured by this method do not necessarily agree with dynamic constants measured by other dynamic methods.

1.7.1 Apparatus

The sonic velocity equipment shown photographically in figure 1.7 consists of:-

- a) a pulse generator with oscilloscope trigger
 - b) pulsing and sensing heads containing piezo-electric crystals
 - c) oscilloscope
 - d) connecting cables
- a) Sonic Pulse Generator
The pulse generator is designed to produce a voltage pulse with a maximum amplitude of about eight hundred volts. The repetition rate of the output pulses is

Author Grills Frank

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