

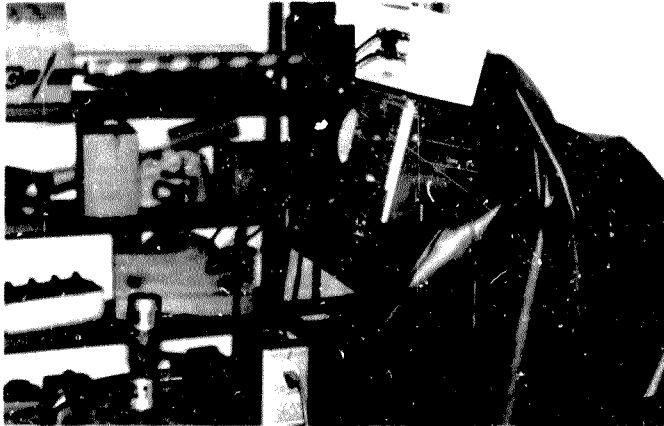


Figure 1.7 Ultrasonic Velocity Equipment

controllable and it was found that the best pulse rate was around 60 pulses per second. This produced a steady trace on the oscilloscope. The same repetition rate was used for all the tests.

b) Pulsing and Sensing Heads

The pulsing and sensing heads are identical in every respect and are interchangeable. One set of transducer heads generates and senses P-waves (compression waves); a second set generates and senses S-waves (transverse waves). The heads had no restriction on the size limit of cores and a rock core of as little as 20mm diameter could be tested. The resonant frequencies of the transducer crystals were 600kHz for P-wave and 800kHz for S-wave heads¹⁹.

Figure 1.8 shows a granite core fixed between a pair of pulsing heads.

c) Oscilloscope

The voltage pulse applied to the pulsing heads and the voltage output of the sensing head is displayed on a cathode-ray oscilloscope. Travel times can be measured on the oscilloscope by measuring along the scale imprinted on the screen. For the transducers used in this project the time base of the oscilloscope was required to be as fast as 10 $\mu\text{sec/cm}$ and the vertical scale amplitude 0.02 volts/cm.

1.7.2 Test Specimens

Test specimens require two flat parallel surfaces between which the travel times of the P- and S-waves are measured. This requirement was met by the procedure described in paragraph 1.6.1 and in APPENDIX A. The ground faces of the rock cores were flat to ± 0.0254 mm and parallel to ± 0.005 mm per mm separation. The



Figure 1.8 Granite Core held between Pulsing Heads

samples were tested in a dry condition.

It was also recommended in the Experimental Manual¹⁹ that the following stipulations be met.

- 1) The condition of infinite extent. This is satisfied when the average grain size < wave length of the pulse < minimum specimen dimension.
- 2) The lateral minimum dimension (normal to the direction of wave propagation) is recommended to be not less than ten times the wave length. ASTM recommendation D2845²⁰ stipulates five times the wave length.
- 3) The travel distance of the pulse through the rock should be at least ten times the average grain size.
- 4) The ratio of the pulse travel distance between parallel faces and the minimum lateral dimension is not to exceed five if reliable free medium velocities are required.
- 5) The wave length should be at least twice the average grain size of the material.

All the above stipulations were met with the exception of (2). Calculating the wavelengths of the respective pulses using the resonant frequencies of the pulsing heads results in the following:-

a)	for granite	P-wave length	= 10,1 mm
		S-wave length	= 4,1 mm
b)	for andesite	P-wave length	= 11,5 mm
		S-wave length	= 4,8 mm

Therefore although condition (2) is met for S-waves it obviously is not for P-waves. It must be realised that practically speaking condition (2) is not easy to meet. If it was rigidly adhered to with this type of pulsing head the resulting core dimensions would be 120mm diameter

and 240mm long. The very fact that the ASTM recommendation stipulates that the minimum dimension should be five times the wave length and not ten shows the apparent lack of agreement on this stipulation. (See also additional note on pages 28(a) and 28(b)).

1.7.3 Measurement of Travel Distance and Density

Travel distances (i.e. core lengths) were measured by means of a vernier scale between flat parallel surfaces. The weight of each specimen was measured to an accuracy of 0.1g and the density calculated from measurements of length and diameter.

1.7.4 Attaching Pulsar Heads to Specimens

Gel coupling between pulsing heads and specimen is essential for effective energy transmission. This was achieved by use of a silicon compound releasing agent, trade name Dow Corning 7 compound. The gel was applied to the pulsing head and the test piece rotated firmly on the pulsing head. The gel was forced out creating a vacuum and hence an excellent connection. The pulse heads were orientated so that the cable connectors were directly above one another.

1.7.5 Compression P-Waves

The first arrival time of the compression wave on the scope was easily determined since it was marked by the first deviation of the wave form from a straight horizontal line. The wave deviated in a downward direction. The start of the wave was carefully zeroed and the vertical amplitude of the scope adjusted to a maximum in order to produce a sharp 'knee' in the trace. Figure 1.9 shows diagrammatically a typical trace of a P-wave. The travel time T_p is simply measured from the start of the trace to the 'knee'.

Reference 21 discusses three methods for determining the velocity of propagation of elastic waves in laboratory rock testing.

1. First method - a high frequency ultrasonic pulse technique - similar to that described in the text.
2. Second method - a low frequency ultrasonic technique for bar-like or rod-like specimens.
3. Third method - A resonant frequency method. By determination of the resonant frequency of the dilational vibration of bar-like cylindrical rock specimens the dilational wave propagation can be calculated.

Table 1.2(a) Different Ultrasonic Methods for Rock Testing, from Ref. 21

	FIRST METHOD	SECOND METHOD
1. Pulse generator frequency range	100kHz - 2MHz	2 - 30kHz
2. Repetition frequency of the pulse generator	10 - 1000s ⁻¹	10 - 100s ⁻¹
3. Transducer frequency range	100kHz - 2MHz	2 - 30kHz
4. Optimal aspect ratio	2:1	3:1

The method outlined in the current text essentially refers to the first method. The pulse generator used had a repetition frequency of 60 per second and the resonant frequency of the P-wave transducer was 600kHz. The aspect ratio of the specimens tested was 2:1. The recommendations given in the text apply to the first method. However, as mentioned previously, there is lack of agreement on the requirement for minimum lateral dimension. The ASTM recommendation²⁰ would require, for the current tests, a minimum lateral dimension of about 50mm. This is slightly greater than the actual

28(b)

dimension of 42mm. Further, the writer discussed and checked the test technique with senior members of the Mining Engineering Department who utilize the same experimental procedure for dynamic rock testing. They were satisfied with the chosen approach in every respect. It was not possible to drill large diameter cores to check the minimum lateral dimension recommendation; consequently the current test technique was adapted as a practical compromise, and with the endorsement of Rock Mechanics staff.

Actually a minute portion of the travel time (approximately 1 μ s) is not recorded, due to the fact that the scope triggers about 1 μ s after the pulse is initiated. However, this small time was accounted for when a correction was made for the time delay involved in the waves passing through the pulsing heads.

1.7.6 Shear S-Waves

First arrival time of shear-waves is not so readily discernible on the scope. The shear-wave arrival may be obscured by vibrations due to 'ringing' of the transducers and reflections of the compression wave²¹. Ringing may occur at high pulse repetition rates and is caused by the previous pulse not having sufficient time to completely die away before the next pulse occurs. 'Ringing' causes unsteady traces on the scope¹⁹.

The shear plate type of piezo electric crystal used here invariably produces some compression wave components which arrive ahead of the shear wave. In fact the shear-wave heads can be used to measure the travel times of both P and S waves. However the initial 'knee' of the compression wave is not so clearly defined when using the shear pulse heads. Some time after the P-wave has arrived at the sensing head, the S-wave arrives and causes a large upward sweep on the oscilloscope. Figure 1.9 shows a typical trace for granite. The S-wave travel time T_s is obtained by measuring from the start of the trace to the start of the strong upward sweep, which usually occurred at the bottom of a wave trough, although it could occur at the peak of a trace. Hence the accuracy of the S-wave arrival time depends on the skill of the operator. If many specimens are under test and the material is uniform the inaccuracies can be resolved to a large extent.

It was found that the travel time of the S-wave was approximately twice that of the P-wave for a particular specimen. This was a useful guideline to locate the S-wave travel time. The estimation of the shear wave arrival time can be performed more easily on

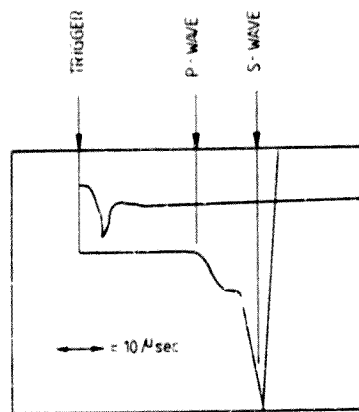


Figure 1.9 Typical P- and S- Wave Trace for Granite
(after Terrametrics 19)

specimens whose length is optimised e.g. an aspect ratio of 2:1 is preferable to a ratio of 1:1⁽²¹⁾.

1.7.7 Calibration Procedures

The travel times measured on the different rock specimens were corrected by subtracting the zero correction time (the travel time taken for the pulses to move through the pulsing heads themselves). The zero correction time was obtained by two methods:-

- 1) Placing the transducers in direct contact with each other and measuring the delay directly. This method is not recommended for shear pulsing heads, as slight misalignment can produce large errors²¹.
- 2) Measuring the travel times of several specimens of the same material but of different lengths. Travel time versus travel distance was plotted for several andesite cores. The graph was extrapolated back to zero travel distance to give the travel time between the heads.

Method 2 was adopted for this dissertation. The results are shown in figure 1.10 and figure 1.11. From the graphs the correction times were 1.3 μ s for the P-waves and 4.6 μ s for the S-waves.

1.7.8 Calculation of Dynamic Elastic Constants

The wave velocities were calculated from the corrected travel times and the travel distance x between the transmitter and receiver, by using the equations:

$$V_p = x / T_p \quad \text{..... (1.5)}$$

$$V_s = x / T_s \quad \text{..... (1.6)}$$

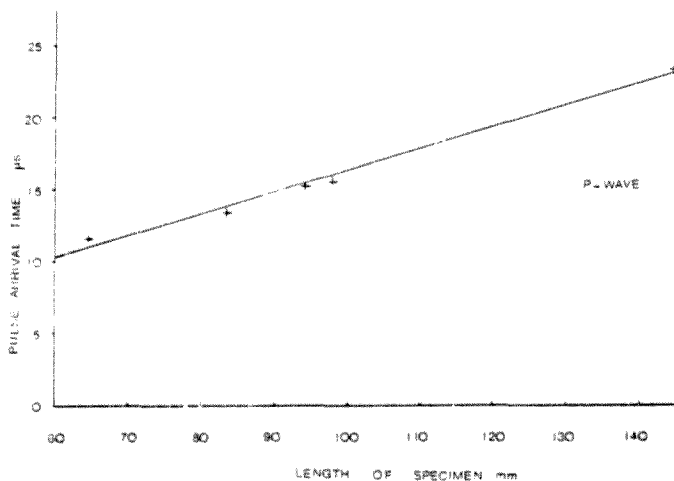
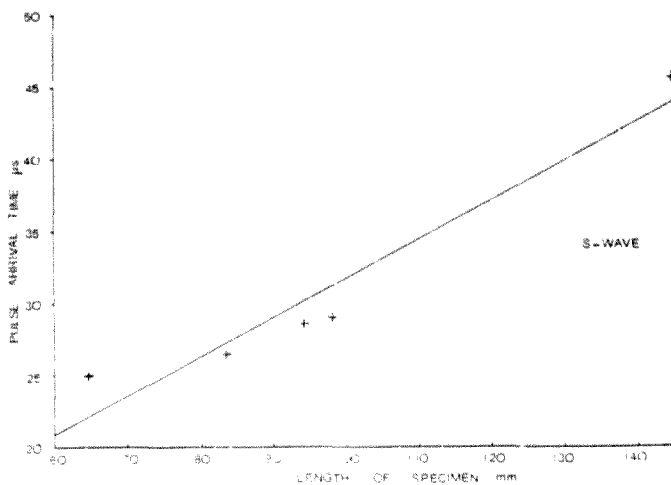


Figure 1.10 Graphs Showing Correction Times for P-Wave Piezo Electric Head



Figures 1.11 Graphs Showing Correction Times for S-Wave Piezo Electric Head

where V_p is the velocity of the longitudinal wave
 V_s is the velocity of the shear wave
 T_p and T_s are the times taken to travel
distance l of the P- and S-wave respectively.

The dynamic elastic constants were calculated from equations (1.1), (1.2) and (1.3) as given in section 1.4.

1.7.9 Results

Table 1.3 shows the results for the granite cores and Table 1.4 shows results for the andesite cores. For repeatability of results refer to Appendix B. For specimen properties and transmission times (T_p for P- wave travel time and T_s for S- wave travel time) refer to Appendix B tables B2 and B3 for granite and andesite cores respectively.

The results for the granite specimens JA3 and JA4 were excluded from subsequent calculations on the basis of a statistical outlier test. The values recorded were obviously not representative of the granite rock.

Table 1.5 shows the mean and standard deviation of measurements for both types of aggregate. Although there is a large variation in both P-wave and S- wave velocity especially for the granite cores, there is only slight deviation about the mean for the dynamic elastic constants E_d and ν_d . This is because the P-wave and S-wave velocity variations tend to cancel out in equations (1.1) and (1.3). The results imply that andesite is more homogeneous than granite and has possibly fewer pores and cracks.

Table 1.3 Transmission of body waves through Granite rock specimens

MATERIAL	SPECIMEN NUMBER	CALCULATED VELOCITY		DYNAMIC ELASTIC CONSTANTS		
		V_p m/s	V_s m/s	E_d GPa	G_d GPa	ν_d
GRANITE	JA1	6029,2	3252,0	75,2	29,1	0,30
	JA2	5723,4	3312,3	73,7	29,5	0,25
	JA3	4953,1	2336,6	39,2	14,4	0,36
	JA4	5422,6	2554,7	46,8	17,2	0,36
	JA5	5883,5	3312,3	73,5	29,0	0,27
	JA6	7518,0	3161,0	76,2	27,4	0,39
	JA7	5825,2	3155,3	67,5	26,1	0,29
	JA8	5863,3	3208,7	69,6	27,0	0,29

NB: Correction times for pulsing heads:
P-Wave - 1,3 μ sec
S-Wave - 4,6 μ sec

Table 1.3 Transmission of body waves through Granite rock specimens/continued

MATERIAL	SPECIMEN NUMBER	CALCULATED VELOCITY		DYNAMIC ELASTIC CONSTANTS		
		V_p m/s	V_s m/s	E_d GPa	G_d GPa	ν_d
GRANITE	JA9	6270,7	3418,0	79,7	30,9	0,29
	JA10	5991,6	3413,3	77,5	30,8	0,26
	JC1	6057,6	3314,9	74,6	29,0	0,29
	JC2	6176,5	3282,8	74,5	28,6	0,30
	JC3	6420,0	3446,7	81,7	31,5	0,30
	JC4	6188,9	3355,4	77,2	30,0	0,29
	JF1	5780,9	3131,3	68,7	26,6	0,29
	JF2	5943,6	3252,6	73,5	28,6	0,29

NB: Correction times for pulsing heads:
P-Wave - 1,3 μ sec
S-Wave - 4,6 μ sec

Table 1.3 Transmission of body waves through Granite rock specimens/continued

MATERIAL	SPECIMEN NUMBER	CALCULATED VELOCITY		DYNAMIC ELASTIC CONSTANTS		
		V_p m/s	V_s m/s	E_d GPa	G_d GPa	ν_d
GRANITE	JF3	5536,3	3082,7	64,1	25,1	0,28
	JF4	5943,6	3129,4	69,2	26,5	0,31
	JF5	5856,1	3083,3	65,0	24,9	0,31
	JF6	5806,9	3315,0	75,0	29,6	0,26
	JF7	5904,8	3254,6	71,2	27,8	0,28

NB: Correction times for pulsing heads:
P-Wave - 1,3 μ sec
S-Wave - 4,6 μ sec

Table 1.4 Transmission of body waves through Andesite rock specimens

MATERIAL	SPECIMEN NUMBER	CALCULATED VELOCITY		DYNAMIC ELASTIC CONSTANTS		
		V_p m/s	V_s m/s	E_d GPa	G_d GPa	ν_d
ANDESITE	X1					
	X2	CORES USED EXCLUSIVELY FOR CORRECTION TIMES				
	X3					
	X4					
	AA1	6910,5	3818,1	107,0	41,8	0,28
	AA2	6903,4	3751,1	104,1	40,4	0,29
	AA3	6906,7	3669,2	100,6	38,6	0,30
	AA4	6794,8	3590,2	97,4	37,3	0,31

NB: Correction times for pulsing heads:
P-Wave - 1,3 μ sec
S-Wave - 4,6 μ sec

Table 1.4 Transmission of body waves through Andesite rock specimens/continued

MATERIAL	SPECIMEN NUMBER	CALCULATED VELOCITY		DYNAMIC ELASTIC CONSTANTS		
		V_p m/s	V_s m/s	E_d GPa	G_d GPa	ν_d
ANDESITE	AA5	6855,9	3673,4	100,5	38,7	0,30
	AA6	6876,9	3653,3	100,0	38,4	0,30
	AA7	6911,8	3755,7	104,1	40,3	0,29
	AB1	6859,9	3727,5	103,1	40,0	0,29
	AB2	6833,3	3837,9	107,9	42,5	0,27
	AB3	6837,4	3840,2	108,0	42,5	0,27
	AB4	6947,7	3928,3	111,0	43,9	0,27
	AB5	6886,6	3836,4	107,6	42,2	0,28

NB: Correction times for pulsing heads:
P-Wave - 1,3 μ sec
S-Wave - 4,6 μ sec

Table 1.4 Transmission of body waves through Andesite rock specimens/continued

MATERIAL	SPECIMEN NUMBER	CALCULATED VELOCITY		DYNAMIC ELASTIC CONSTANTS		
		V_p m/s	V_s m/s	E_d GPa	G_d GPa	ν_d
ANDESITE	AB6	6964,9	3837,2	108,6	42,3	0,28
	AB7	7050,4	3884,3	112,4	43,8	0,28
	AC1	6810,3	3739,6	103,4	40,3	0,28
	AC2	6813,0	3826,5	107,0	42,1	0,27
	AC3	6807,6	3705,0	101,8	39,5	0,29
	AC4	6863,4	3738,1	103,5	40,2	0,29
	AC5	6826,6	3715,3	102,3	40,0	0,29
	AC6	6825,2	3747,8	103,9	40,5	0,28
	AC7	3615,7	98,0	37,6	0,31	

NB: Correction times for pulsing heads:
P-Wave - 1,3 μ sec
S-Wave - 4,6 μ sec

This is implied by the greater density and higher P- and S-wave velocities measured in the andesite cores. Hence the andesite is a 'stiffer' rock and has a higher dynamic modulus than granite.

The mean and standard deviations of the specimens taken from each individual rock boulder were also calculated to assess whether there were any localised differences i.e. variation within the sample taken from a specific area in the quarry. The results are shown in table 1.6. It can be seen that only small differences are apparent between the different rock boulders and it was considered that these differences would not affect the later tests significantly. Once again the andesite showed less variation than the granite. It could be concluded that for the particular quarry face from which these rocks were taken there is good uniformity of aggregate.

1.8 LIMITATIONS

The previous sections show that this technique of measuring dynamic elastic constants in rocks is fairly rapid and gives preliminary predictions of static properties. However certain limitations must be borne in mind. The elastic constants measured here are carried out on small laboratory specimens. This could accentuate inhomogeneity and anisotropy and hence give a distorted picture of the elastic constants of the rock mass as a whole. This is a definite drawback even when coupled with statistical analysis²². The coupling between the piezoelectric transducer and the specimen may be inferior and lead to interface losses. This is especially important when shear transducers are used. Equations (1.1), (1.2), and (1.3) are applicable to isotropic rock and when degrees of anisotropy are present further errors may result.

Table 1.5 Mean and Standard Deviation Values of Dynamic Elastic Constants for Granite and Andesite Rock Cores

ROCK TYPE	DENSITY kg/m ³	V _D m/s	V _S m/s	E _d GPa	ν _d
GRANITE	2659,8	6034,6	3256,9	73,3	0,292
(19 cores)	+ 37,3	+ 411,5	+ 111,6	+ 4,7	+ 0,029
ANDESITE	2874,4	6871,8	3756,7	104,4	0,287
(21 cores)	+ 12,4	+ 62,9	+ 89,4	+ 4,1	+ 0,012

Table 1.6 Variations of Dynamic Elastic Constants of Rock Cores Drilled from Different Boulders

ROCK TYPE	MEAN AND STANDARD DEVIATION E _d (GPa)	VARIATION OF E _d FROM GRAND MEAN OF GRANITE OR ANDESITE %	MEAN AND STANDARD DEVIATION ν _d
GRANITE JA (8 cores)	74,1 + 4,0	+ 1,5	0,293 + 0,043
GRANITE JL (4 cores)	77,0 + 3,4	+ 5,2	0,295 + 0,006
GRANITE JF (7 cores)	69,5 + 4,1	- 5,0	0,289 + 0,018
ANDESITE AA (7 cores)	102,0 + 3,3	- 2,4	0,296 + 0,010
ANDESITE AB (7 cores)	108,4 + 3,0	+ 3,7	0,279 + 0,007
ANDESITE AC (7 cores)	102,8 + 2,7	- 1,5	0,287 + 0,013

1.9 Determination of Static Elastic Modulus of Aggregates

The 42mm nominal diameter and 84mm nominal length rock cylinders were instrumented with electrical resistance strain gauges. Six cores were randomly selected from both the granite and andesite rock. Two each of 10mm gauges were used to measure the longitudinal and lateral strains of the specimen by attaching the gauges at mid-height, the corresponding strain gauges being diametrically opposite one another. The corresponding gauges were connected in series thus giving the average of the strain on two sides of the core. Strain readings were taken over one cycle of loading and unloading. The strain gauges were connected to a data logger and the rock specimens tested in an Amsler 200 tonne Compression Machine.

Initially, two cores from each rock type were loaded to failure. The average failure loads for the granite and andesite cores were 330 kN and 300 kN respectively. The cores were loaded at about 30 Mpa/min. The cut off load for the determination of the static elastic moduli for both rock types was chosen as 60% of the respective failure load. Six cores were then chosen at random from the granite cores and a similar number from the andesite cores. A length to diameter ratio ranging from 2.0 to 2.5 was used to ensure a fairly uniform stress distribution in the sample and to increase the possibility of a failure plane being free to form without intersecting the testing head²³.

It should also be noted that for many rock types the slope of the initial or virgin stress-strain curve for loading and unloading is less than the slope of the curve for subsequent cycles. The cause of this difference has not been definitely determined, although as pointed out earlier in this chapter several investigators have attributed it to the opening of micro-cracks at the time the specimen is separated from its surroundings and its confining stresses are relieved. Since the difference between the slope of the stress-strain curve for the initial and

subsequent cycles may be large, and because the difference between successive cycles usually decreases, it is common practice to subject the specimen to a number of loading cycles before recording the data cycle and to specify the data cycle²⁴. The difference between successive stress/strain curves was not thought to be significant for the present granite and andesite rocks, and therefore the readings were taken over the second loading cycle.

In point of fact, a series of tests carried out by Pells and Ferry²⁵ on determination of Young's Modulus showed that from an engineering judgement viewpoint, it did not make any significant difference whether the specimens had a length to diameter ratio of 2.0 or were cut non-parallel (but with flat) ends. Therefore there is evidence to suggest that some recommendations relating to Young's Modulus determination for rocks are unnecessarily stringent.

1.9.1 Results

The results for static elastic constants are given in table 1.7 for the granite and andesite cores. Table 1.7 clearly shows that andesite has a higher static elastic modulus than granite, the ratio of the modulus of andesite to granite being approximately 1.3. The Poisson's ratio value for andesite is also higher than that of the granites. Values for the two rock types vary about the mean in a similar manner as shown by their respective standard deviations. Typical curves of stress against strain as plotted by a computer graphics programme are shown in figure 1.12 and 1.13 for granite and figure 1.14 and 1.15 for andesite. The static elastic modulus was determined by the best fit straight line through the plotted points up to 30% of the failure stress.

Both sets of curves for the granites and the andesites are linear over this stress range. The unloading curve of granite specimen

JA5 shows the hysteresis effects for granites. Hysteresis effects varied from specimen to specimen even when the cores were taken from one boulder. For example specimen JA5 (refer to figure 1.12) shows a clear hysteresis loop in the longitudinal strain while JA10 exhibited clear hysteresis in the lateral direction. The plot for JC4 (refer to figure 1.13) shows the lateral strain tending to the horizontal. This was due to the fact that the specimen failed at this stress level. For the andesites, the plot of specimen AC2 (refer to figure 1.14) shows much more reduced hysteresis effects than the granites and in general the andesite plots exhibited greater linearity than the granites. The hysteresis effect is also greatly reduced in both the longitudinal and lateral directions in the andesite rock.

The static elastic modulus and corresponding Poisson's ratio was calculated at 30% of the failure stress for each rock type. The average stress and strain were calculated from the best fit straight line. The static elastic modulus E_s is given by:

$$E_s = \frac{\sigma_a}{\epsilon_a} \quad \text{..... (1.7)}$$

σ_a = average stress

ϵ_a = average strain

1.10 DISCUSSION

The results of the average static and dynamic moduli and Poisson's ratio are shown in table 1.8 and table 1.9 respectively. The dynamic method gives better repeatability of results. Table 1.8 shows that for granites there is about a 10 per cent variation between the static and dynamic elastic modulus E_d , with E_s being greater than E_d . This appears at first to be an anomaly. However, as stated in section 1.2, the ratio of dynamic to static

Table 1.7 Static Elastic Modulus and Poissons's Ratio for Granite and Andesite Rock

SPECIMEN	GRANITE		SPECIMEN	ANDESITE	
	STATIC MODULUS OF ELASTICITY E_s GPa	POISSON'S RATIO ν		STATIC MODULUS OF ELASTICITY E_s GPa	POISSON'S RATIO ν
JA5	81,70	0,22	AA2	101,83	0,32
JA10	86,22	0,30	AB7	112,72	0,27
JC1	92,84	0,32	AC2	102,62	0,29
JC4	77,59	0,30	AC3	108,71	0,29
JF1	72,08	0,17	AC4	110,79	0,30
JF2	79,04	0,24	AC6	89,71	0,25
Average	81,58	0,26		104,40	0,29
Standard Deviation	7,22	0,06		8,42	0,02

NB: VALUES TAKEN AS BEST FIT STRAIGHT LINE UP TO 30% OF FAILURE STRESS

GRANITE ROCK
SPEC NO. JAS

SOLID LINE: LONGITUDINAL
BROKEN LINE: LATERAL

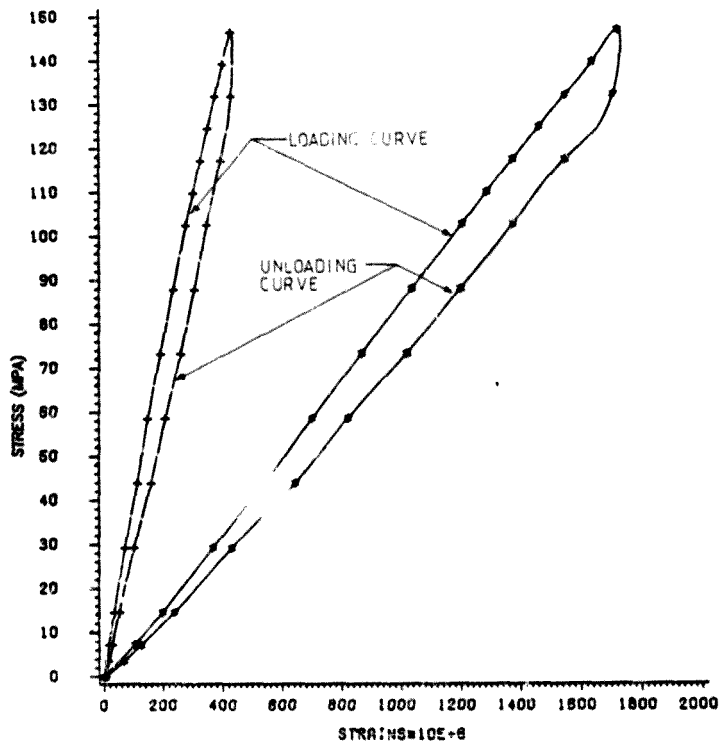


FIG 1.12 PLOT OF LATERAL AND LONGITUDINAL STRAIN
VS. STRESS FOR GRANITE ROCK (JUKSKEI QUARRY).

GRANITE ROCK
SPEC NO. JC4

SOLID LINE: LONGITUDINAL
BROKEN LINE: LATERAL

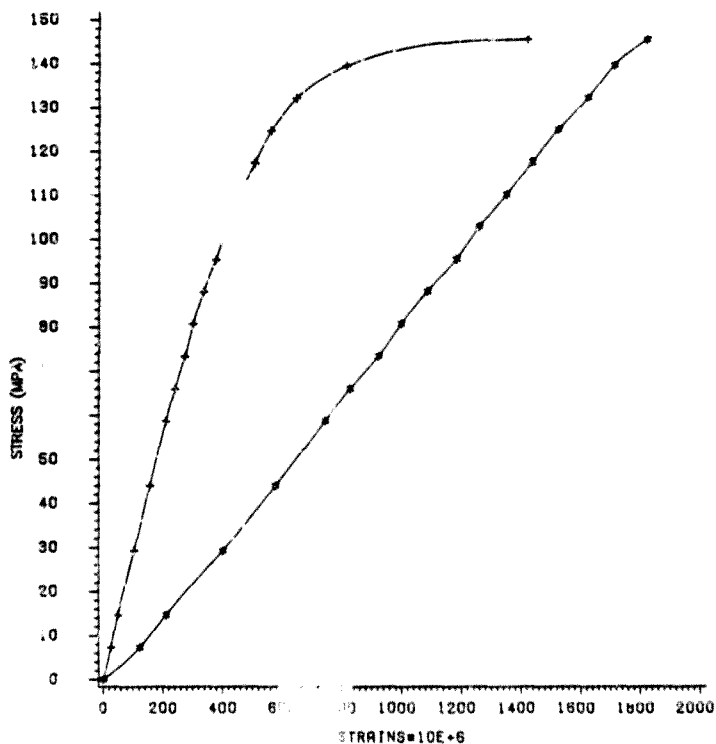


FIG 1.13 PLOT OF LATERAL AND LONGITUDINAL STRAIN
VS. STRESS FOR GRANITE ROCK (JUKSKEI QUARRY).

ANDESITE ROCK
SPEC NO. AC2

SOLID LINE: LONGITUDINAL
DASHED LINE: LATERAL

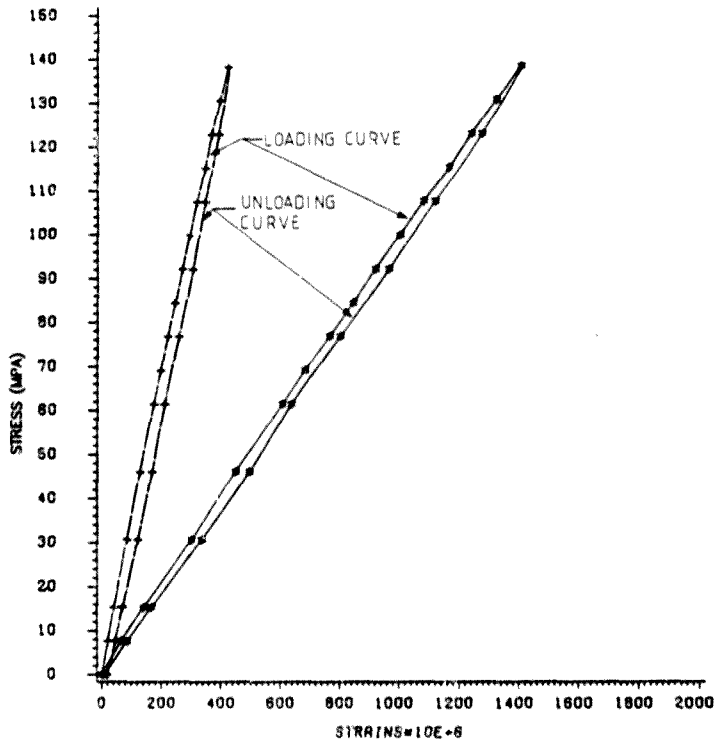


FIG 1.14 PLOT OF LATERAL AND LONGITUDINAL STRAIN
VS. STRESS FOR ANDESITE ROCK (EIKENHOF QUARRY).

ANDESITE ROCK
SPEC NO. AC4

SOLID LINE: LONGITUDINAL
BROKEN LINE: LATERAL

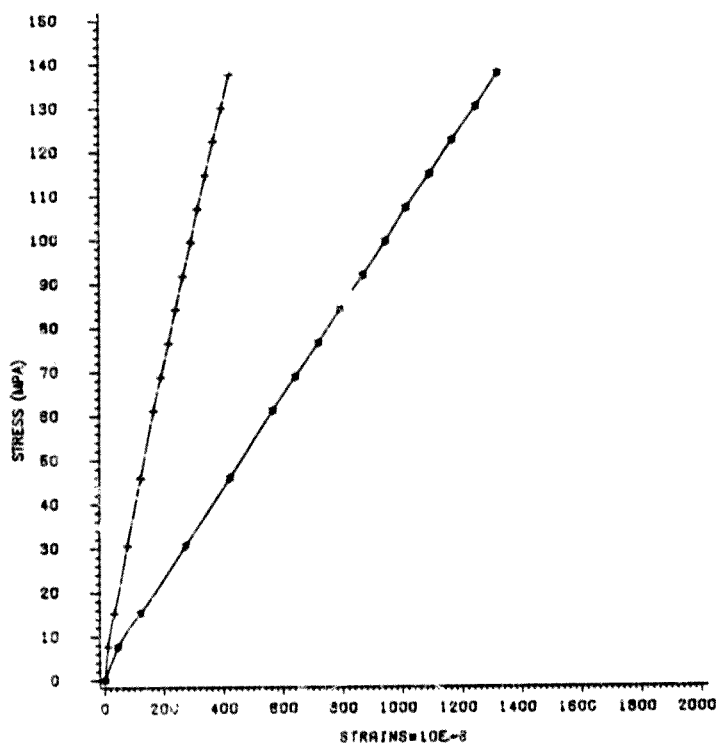


FIG 1.15 PLOT OF LATERAL AND LONGITUDINAL STRAIN
VS. STRESS FOR ANDESITE ROCK (EIKENHOF QUARRY).

moduli may vary between 0,85 and 2,926. Bhattacharyya^{27,28} also states that in tests he carried out on various rocks, the static modulus was greater than the dynamic modulus. In the case of Bald Hill Claystone the ratio of dynamic to static moduli ranged from 0,52 to 2,6 (5 cores tested) and for Hawkesbury Sandstone the ratio varied from 0,34 to 0,70 (10 cores tested). He states that the relatively low dynamic modulus values resulted from low transmission velocities of the P-waves, probably caused by micro-cracks and inhomogeneities in the instance of Bald Hill Claystone, and by a high degree of porosity and low water content for the Hawkesbury Sandstones. The possible causes of the low dynamic elastic moduli in the case of the granites in this project could possibly be micro-cracks and inhomogeneities.

The ultrasonic equipment was checked by ascertaining the dynamic elastic modulus of a steel cylinder. For isotropic, homogeneous, linearly elastic materials such as steel which lack the textural discontinuities characteristic of rocks, E_s and E_d should be equal²⁹. The dynamic elastic modulus of the steel cylinders ranged from $E_d = 217 - 224$ GPa. The average value quoted in texts is 208 GPa for the static elastic modulus of steel³⁰. The difference is about six per cent and therefore it was considered that the equipment was functioning correctly.

Table 1.7 shows that the static modulus E_s of the andesites is approximately 28% greater than that of the granites.

The modulus results for andesite show that the result obtained from the dynamic method is virtually identical to that of the result obtained from the static method. When one considers the time involved in static testing, it is obviously more beneficial to use the dynamic method to evaluate the modulus of elasticity for this andesite. Similar comment would also apply to the results given in table 1.9 for Poisson's ratio values.

Table 1.8 Comparison of measured values of E_d and E_s for Granite and Andesite rock cores

ROCK TYPE	E_d GPa	E_s GPa	RATIO $\frac{E_d}{E_s}$
GRANITE	$73,03 \pm 4,74$	$81,58 \pm 7,22$	0,895
ANDESITE	$104,39 \pm 4,05$	$104,40 \pm 8,42$	1,000

Table 1.9 Comparison of measured values of v_d and v_s for Granite and Andesite rock cores

ROCK TYPE	v_d	v_s	RATIO $\frac{v_d}{v_s}$
GRANITE	$0,29 \pm 0,029$	$0,26 \pm 0,058$	1,115
ANDESITE	$0,29 \pm 0,012$	$0,29 \pm 0,024$	1,000

Table 1.10 Summary of mean values of dynamic and static elastic constants for Granite and Andesite rock

ROCK TYPE	DENSITY kg/m^3	P-WAVE VELOCITY v_p m/s	S-WAVE VELOCITY v_s m/s	DYNAMIC ELASTIC MODULUS E_d GPa	DYNAMIC POISSON'S RATIO ν_d	STATIC ELASTIC MODULUS E_s GPa	STATIC POISSON'S RATIO ν_s
GRANITE	2660	6035	3257	73	0,29	82	0,26
ANDESITE	2874	6677	3757	104	0,29	104	0,29

Table 1.10 summarises the test results for granite and andesite rock. These values were used for calculations in the present work. However, further work needs to be undertaken in order to assess variability of rock properties from different sources, or within one quarry.

1.11 CONCLUSIONS

- a) The average static elastic modulus of andesite rock was 28% greater than the elastic modulus of granite.
- b) On average the static elastic modulus of granite was greater than its dynamic elastic modulus, thought to be due to the presence of inhomogeneities in the granite rock assemblage resulting in low transmission velocities of the P-waves and hence low dynamic modulus results.
- c) The dynamic elastic modulus, E_d of the andesite rock was equal to that of its static modulus, E_s .
- d) The andesite rock is less variable than the granite rock producing more uniform results.
- e) Both quarry locations gave fairly uniform results from different rock boulders.
- f) For homogeneous rocks such as andesite the dynamic method outlined in this chapter is a faster and more reliable method of determining a value of both elastic modulus and Poissons's ratio. However, this is conditional on the accuracy with which the pulse arrival times can be estimated, especially the S-wave value.
- g) Further work is required to assess the variability of rock properties from different sources, or within one quarry.

CHAPTER 1

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CHAPTER 2 ELASTICITY OF HARDENED CEMENT PASTES AND MORTARS

2.1 BACKGROUND

Many attempts have been made to predict the dependence of concrete stiffness upon the stiffness of the paste and aggregate and their volume concentrations. Within the scope of this dissertation this chapter deals with the estimation of elastic modulus for pastes and mortars related to the various concrete mixes discussed in chapter 3. It is known that the stiffness of hardened cement paste and concrete increases as cement/water ratio is increased¹. However, as the load is increased, the stiffness of both paste and concrete decreases continuously.

It is often stated in the literature (see Shah and Winter²) that cement paste specimens exhibit an approximately linear relationship between stress and longitudinal strain, but this has been shown to be incorrect by Spooner³. This misconception may have arisen because pastes of cement/water ratio possibly as high as about four may have been tested; such pastes cannot necessarily be regarded as representative of paste in concrete⁴. The structure of paste is also sufficiently disordered and heterogeneous to ensure that stress-strain relationships are not simple. The stress-lateral strain curves for paste are also non-linear⁴. The rate of lateral strain increases continuously as the applied stress increases, and this becomes very marked as the maximum stress is approached.

The relation between stress and longitudinal strain for rock, paste and concrete is shown in Figure 2.1(a). The relations between stress and lateral strain and volumetric strain for rock, paste and concrete are shown in Figure 2.1(b). It can be seen that at low stress levels the curve for the cement paste departs

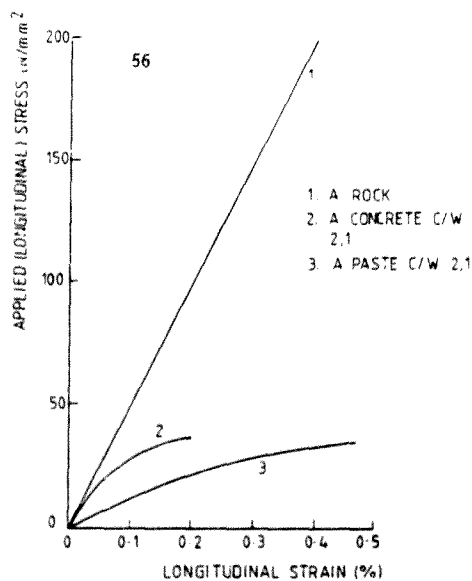


Figure 2.1(a) Relation between Stress and Longitudinal Strain for Rock, Paste and Concrete (after Hobbs⁴)

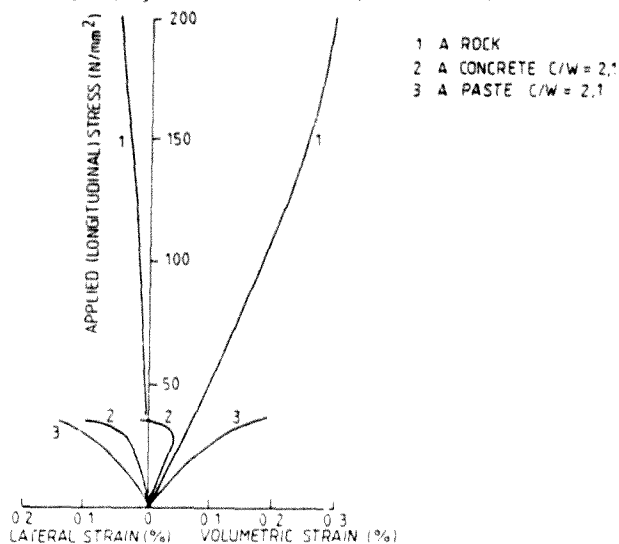


Figure 2.1 (b) Relation between Stress and Lateral Strain and between Stress and Volumetric Strain for Rock, Paste and Concrete (after Hobbs⁴)

slightly from linearity. The approximate longitudinal strains at failure for aggregate, paste and concrete are of the order of 0,4 to 1,0%, 0,6 to 0,8% and 0,1 to 0,25% respectively. The approximate lateral strains at failure for pastes and concretes are 0,3 to 0,5 of their longitudinal failure strains.

Hobbs⁴ states, "The stress-volumetric strain curves for aggregate, paste and concrete are generally non-linear (as shown in figure 2.1(b)). For pastes there is an ongoing reduction in volume as applied stress is increased. As stress increases the gradient of the stress-volumetric strain curve decreases. This would seem to imply that pastes compact or consolidate continuously until failure stress is reached. Consequently no evidence of paste cracking prior to failure is apparent from the stress-volumetric strain curve. The approximate maximum volumetric strains for rocks, pastes and concretes are 0,1 to 0,4%, 0,1 to 0,3% and 0,04 to 0,08% respectively."

This chapter outlines the factors affecting the modulus of elasticity of both pastes and mortars. It also deals with the laboratory programme to determine the static, electrodynamic and ultrasonic elastic moduli of pastes and mortars for specified cement/water ratios. The objectives of the laboratory programme were to:

- (a) ascertain the engineering properties of pastes and mortars for use in theoretical models related to concrete stiffness.
- (b) attempt to correlate the static elastic modulus, E_s to the electrodynamic and ultrasonic dynamic moduli, E_{ed} and E_{ud} respectively.

2.2 ELASTICITY AND HARDENED CEMENT PASTE

2.2.1 Porosity

When a load is applied to hardened cement paste (h.c.p.) the forces are carried internally by the solid structure, made up of both the larger crystalline hydration products and the calcium silicate gel⁵. The solid skeleton is interspersed with a large volume of gel and capillary pores with a wide range of sizes. It is conceivable that water within the pores initially carries some load, and hence comes under pressure. The gradual dissipation of the pressure in the water would lead to time-dependent strain which would be observed as creep, and the elastic (immediate) strain for wet concrete would differ from dry concrete. However, the evidence indicates that this effect, although it may be present, does not result in more than a small part of the load being diverted from the solid material. It follows that the elastic response of the h.c.p. depends on the rigidity of the skeletal structure⁶.

It seems reasonable to assume that the stiffness of the various solid hydration products and of the unhydrated cement grains incorporated in the structure are not too dissimilar and that the configuration does not change with the development of hydration. The difference between one h.c.p. and another then reduces to different densities of structure. This implies that the elastic modulus of h.c.p., E_p , will increase with increasing degree of hydration and increasing cement/water ratio⁶. This expectation is confirmed by the experimental results shown in figure 2.2 in which the development of hydration is represented by increase in age⁷.

These two separate influences can be incorporated in a single, more fundamental quantity - porosity - which may be either capillary or total porosity. The capillary porosity refers to capillary space only, while total porosity includes gel pores as well as capillaries. When considering the properties of h.c.p.

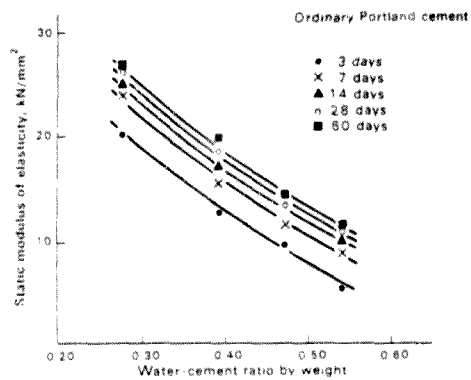


Figure 2.2 The Effect of Cement/Water Ratio and Age on the Static Modulus of Elasticity of Saturated Cement Paste (after Hirsch⁷)

there is no agreed preference to use capillary or total porosity as a measure of porosity.⁸ Figure 2.3 shows the relationship between E_p and capillary porosity, p_c and this can be expressed as:-

$$E_p = E_g (1 - p_c)^3 \quad \dots (2.1)$$

where E_g is the elastic modulus of cement gel; from figure 2.3, E_g is the value of E_p when p_c is zero, ie 32 GPa.

The modulus of elasticity of h.c.p. is related to its compressive strength, increasing with increase in the latter (see figure 2.4). It is generally accepted that porosity constitutes the most important single factor in determining strength. It follows that any factors which affect porosity of the paste also affect its strength. In this context the most important ones are c/w ratio and degree of hydration, which directly affect porosity. The relation between strength of the paste and the c/w ratio is similar to that between strength and porosity.⁹ Consequently factors which affect strength also affect the modulus of elasticity of the paste. Moisture content of the paste is the only exception affecting the two properties differently. Generally speaking, the strength of the cement paste decreases with increasing moisture content (see figure 2.5) whereas its modulus of elasticity increases¹⁰.

2.2.2 Environment

Drying removes water from the larger pores. Since the water, as stated above, may be load bearing, there is a reduction of elastic modulus with decreasing moisture content of the h.c.p. More significantly, water is also removed by drying from the finer pores and from between the layers of the solid material. This water is bound strongly and it can be regarded as part of the solid material, contributing to its stiffness. The loss of this water is the second cause of a reduction in modulus with drying¹⁴.

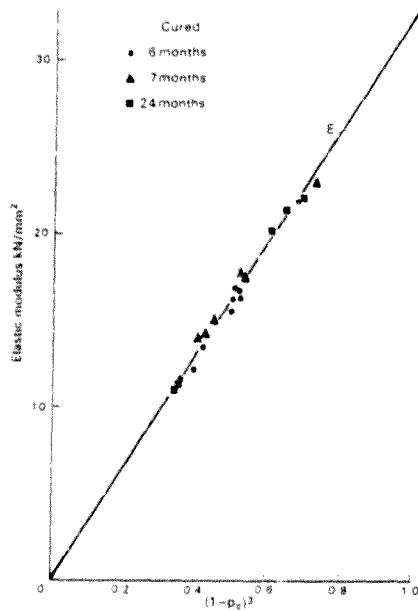


Figure 2.3 Elastic Moduli of Hardened Portland Cement Pastes of Various Capillary Porosities, p_c , Cured 6, 7 and 24 Months (after Helmuth and Turk¹¹)

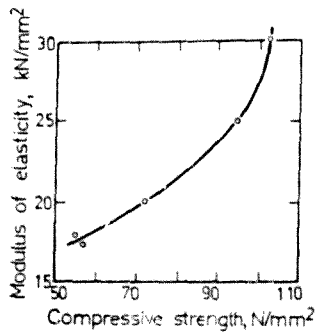


Figure 2.4 Relation Between Modulus of Elasticity and Compressive Strength for Cement Paste (after Feldman and Beaudoin¹²)

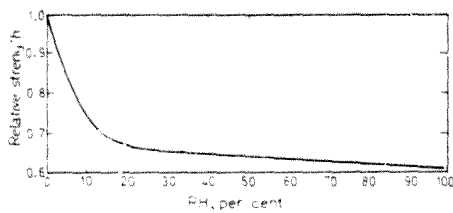


Figure 2.5 Effect of Moisture Content on Compressive Strength of Cement Paste (after Sereda and Feldman¹³)

The effect of moisture content on the modulus of elasticity of the cement paste is demonstrated in figure 2.6¹⁵. It can be seen that drying of the saturated paste at 0% relative humidity decreases the modulus of elasticity considerably. Rewetting, however, starts to affect the modulus only at a moisture content corresponding to a state of equilibrium at approximately 55% relative humidity, and on redrying the modulus is affected only at the very low end of the relative humidity scale.

The Feldman and Sereda¹³ model attributes variation in the modulus of the cement paste with change in moisture content to movement of interlayer water. The layered structure of the C_3S particles is described in figure 2.7 by two parallel short lines. The water between the two lines, marked X in the figure, is interlayer water. The right hand part of the figure describes the variation in the modulus of elasticity of the paste with the change in moisture content (ie figure 2.6) and the left hand part is the isotherm for the interlayer water. The letters A to G represent the same stages in all parts of figure 2.7.

On increasing the relative humidity, water starts to enter the structure from the edges causing expansion by opening up the layers (stages A to C). Figure 2.7 implies that the greater part of the swelling of the cement paste takes place at this stage. The water is not strongly bound at this stage and acts like a web or cross links in a sandwich-type construction; therefore there is no significant increase in the modulus. This stiffening effect and the associated increase in modulus occurs from C to D and reaches a maximum when the layers are saturated (stage D). On drying from D to E, virtually no interlayer water is lost and therefore the modulus remains unaffected. From E to F a small amount of interlayer water is lost from the edges, the modulus being slightly affected. Only when the interlayer water is removed from the middle (ie F to G) is the modulus of elasticity considerably decreased¹⁶.

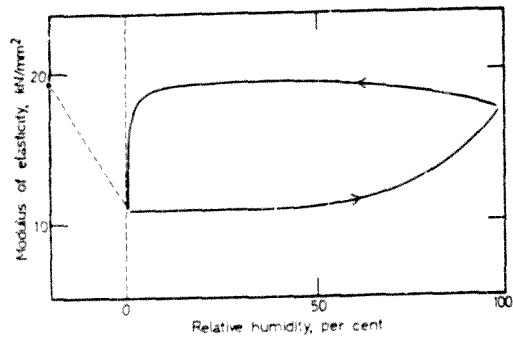


Figure 2.6 Effect of Moisture Content on Modulus of Elasticity of Cement Paste (after Sereda et al¹⁵)

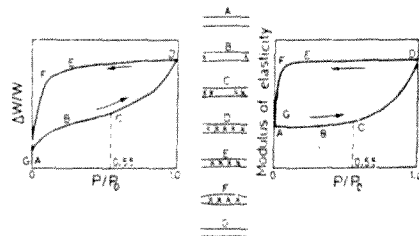


Figure 2.7 Effect of Exit and Re-entry of Interlayer Water on Modulus of Elasticity of Cement Paste (after Feldman and Sereda¹³)

$\frac{\Delta W}{W}$ = relative weight loss

$\frac{P}{P_0}$ = relative water vapour pressure

P_0 = saturation vapour pressure

Temperature, too, affects the elastic modulus of hardened cement pastes. Higher temperatures result in lower moduli. The temperature dependence may well be related to the increased mobility of the moisture at higher temperatures, giving some loss of stiffness in the solid structure¹⁷.

2.3 ELASTIC MODULUS OF CEMENT MORTAR

From figure 2.8 it is apparent that the stress-strain characteristics of mortar are very similar to those of concrete. However, concrete exhibits greater inelasticity leading to greater curvature of a stress-strain plot. Concrete also has a greater capacity to carry a reduced stress at higher strain levels. This gives rise to the descending branch of the stress-strain curve. (However, this depends very much on the stiffness of the testing machine).

As the load increases the mortar softens. Hairline cracks begin to appear shortly after crossing the peak of the stress-strain curve. As the strain increases further the size and the length of the cracks increase. At larger strains sliding occurs in the cracked zone. This appears to be the major component of the longitudinal strain in the descending branch of the stress-strain curve. The nonlinearity of concrete under compressive loading is dependent on the nonlinearity of its cement paste and mortar constituents. Cement mortar is not an elastic brittle material as previously supposed, but is nonlinear and is damaged continuously under load. The process of damage in concrete is also continuous and begins at very low strains¹⁸. These recent findings seem to suggest that microcracking is not necessarily the most important factor in the nonlinear behaviour of concrete¹⁹. Maher and Darwin²⁰ postulated that the many similarities between paste, mortar and concrete suggest that not only the factors affecting damage but also the nonlinear behaviour of mortar and paste dominate the behaviour of concrete.

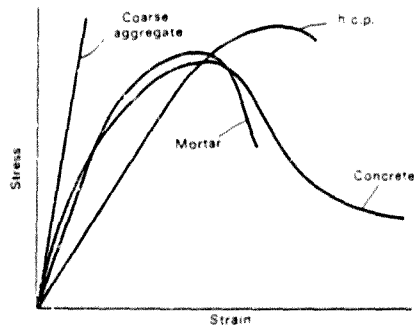


Figure 2.8 Typical Stress-Strain Characteristics of Aggregate, Hardened Cement Paste, Mortar and Concrete (after Swamy and Rao²³)

Other authors²⁰ state that the stress/strain relationship of concrete and mortar is dominated by the development of internal microcracking under increasing load. Hardened cement paste and aggregate combine to form a two-phase material, the resultant stress/strain relation becoming curvilinear due to the presence of interfaces at which microcracking can occur²¹. However, as previously stated although rock which is used as aggregate exhibits an approximately linear relationship between stress and longitudinal strain up to failure, paste exhibits markedly non-linear behaviour²².

Davis²⁴ carried out tests to determine the relationship between compressive strength and static elastic modulus for mortars. He found that rich mortars, because of their high paste content had significantly lower moduli relative to their respective concretes. Conversely, lean mortars rendered relatively high elastic moduli but owing to their low cement content rarely exceeded a compressive strength of 10 MPa. It is obvious that stress-strain relationships for mortar are not simple and deserve considerably more attention than they have received to date.

2.4 LABORATORY PROCEDURE

The objectives of the laboratory tests were:-

- 1) to assess the engineering properties of cement pastes and mortars in order subsequently to determine the elastic modulus of concrete by theoretical models.
- 2) to correlate measured dynamic elastic moduli with measured static elastic moduli.

A representative range of cement/water ratios were chosen and the constituent paste and mortar mixes were extracted from the concrete mix design (see Chapter 3 for details of mix design).

Therefore for each concrete at a particular cement/water ratio, the paste and mortar elastic modulus values for that specific mix could be examined. Hence knowing the elastic modulus of the aggregate, and from measurements of the elastic modulus of paste and mortar (at a specified cement/water ratio), a theoretical estimate of the composite concrete could be made by a mathematical model and compared to the actual measured elastic modulus.

2.4.1 Cement Pastes

Prismatic specimens with cement/water ratios of 2,4, 2,1, 1,8, 1,5 and 1,2 were cast. All specimens were tested in a saturated state at 28 days. Four specimens per cement/water ratio were cast. The selected prism dimensions were 40 x 40 x 160 mm for the paste specimens.

Since cement pastes tend to segregate if conventional casting methods are used, it was decided to adopt an approach similar to that of Spooner²⁵. The technique finally adopted was to cast the specimens in steel moulds, the moulds being finely machined and sealed by the use of petroleum jelly. The sealed moulds were then rotated at 6 revolutions per minute for 15-18h before the specimens were demoulded. Figure 2.9a shows a partially dismantled mould and figure 2.9b shows the apparatus that was assembled to rotate up to six such moulds.

The lower cement/water ratio specimens of 1,2 and 1,5 were left in the moulds for approximately 36 hours to facilitate curing. However after numerous attempts it was decided to omit the paste specimens of cement/water ratio 1,2 as it was found impossible to obtain a void free specimen, even when an admixture was used to try and disperse the cement particles and reduce flocculation.

Spooner²⁵ compared two specimens both of 1,9 cement/water ratio. One prism was cast conventionally on its side and the other rotated, similar to that described above. He found there

Author Grills Frank

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