CHAPTER 2

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CHAPTER 3 ELASTICITY OF CONCRETE

3.1 BACKGROUND

Structural concrete is normally specified in terms of its compressive strength. The type of aggregate is seldom prescribed, unless a lightweight aggregate is utilized. From this limited information an estimate for the elastic modulus of the concrete is calculated. This estimated value is used in design calculations, not only for elastic deformations but also for creep displacements via a creep factor. It is clear that the accuracy of these displacements is dependent on the accuracy with which the elastic modulus has been estimated. It is commonly accepted that the elastic moduli derived from cube strengths are sufficiently accurate for design purposes and this is probably the case in the majority of structures. However, the elastic modulus at a given strength is dependent on the characteristics of the aggregate and it has been found that estimates of elastic modulus could possibly be 50% in error[1].

As traditional sources of aggregate are depleted, different aggregates may be used to provide concretes of adequate strength, but with lower moduli. Over the past few decades most of the mine dump aggregate sources of Witwatersrand quartzite have been exhausted. Engineers have therefore focussed on alternative sources of concrete aggregate. Hence, if the designer is aware of any variations, he could allow greater margins for movement or select aggregates with greater care. It is therefore important to characterise local aggregates and cements and establish useable relationships for design in these materials. The concretes under test in this project are a granite aggregate concrete and an andesite aggregate concrete, with the granite having a lower elastic modulus than the andesite. In chapter 1, results of measurements of the important engineering properties of the rocks were presented and an attempt was made to correlate the static elastic modulus with measured dynamic moduli.

3.2 STRESS-STRAIN CURVE FOR CONCRETE

The term 'elastic modulus' is uniquely defined when applied to perfect linearly elastic, isotropic, homogeneous materials. Concrete, however, is a multiphase composite material. It may be analysed as a two-phase material consisting of aggregate particles embedded in a matrix of cement mortar, if the aggregate phase and mortar phase are considered to be homogeneous and isotropic and if direct interaction between aggregate particles does not occur[2]. The assumption that both aggregates and pastes are elastic is generally true for aggregates but not for pastes[3].

If the ratio of direct stress to the longitudinal strain produced is constant, the constant is referred to as the 'modulus of elasticity' or Young's Modulus. Concrete presents a problem in this respect, as the stress-strain relationship is non-linear. The non-linearity of the stress-strain curve is due to an irreversible deformation of the concrete even after short loading times, which can be measured when the concrete is unloaded. Thus, the term 'instantaneous' strain rather than 'elastic' strain is probably more appropriate. Figure 3.1 shows a typical observation for a test in which two successive cycles of load were applied with the maximum stress being approximately 65% of the concrete strength. The Young's modulus is given by the slope of the stress-strain curves and three possible choices are:

a) the tangent modulus, A, which is measured at the stress under consideration. This value reduces as the stress increases. This modulus is a useful measure of the concrete response to relatively small additional stresses.

b) the initial tangent modulus, B, which is the slope of the tangent to the curve at the origin. This is the closest approximation to a modulus of elasticity derived from truly elastic response. The initial tangent modulus renders the highest value of static elastic modulus. However, this modulus is of little practical significance since it applies only to small stresses and strains. The dynamic modulus of elasticity is a reasonable estimate of the initial tangent modulus, since its determination involves only small displacements of the material.

c) the secant modulus, C, which is a more practical measure of the modulus of elasticity. All strain occurring at a given stress is regarded as elastic, and thus the slope of the secant between the origin and any point on the curve will give the secant modulus for the corresponding applied stress. This value also reduces with stress, but to a lesser extent than the tangent modulus.

Figure 3.1 shows distinct hysteresis loops and a residual strain (OX) after unloading. In addition, the stress/strain curve is at no stage truly linear and the above mentioned moduli are all an approximation to the real behaviour of concrete. The non-linearity becomes more pronounced if:-

a) the load is applied more slowly:- this allows a larger creep strain to occur during the test, so that the slope of the curve diminishes at all stress levels. This effect is reduced by more rapid load application and it is found that the effect reduces on successive loading cycles[4]. This is shown in figure 3.1 by the greatly diminished area enclosed within the second hysteresis loop compared to the first. A recognised experimental technique is to exercise the specimen through several loading cycles before the elastic modulus is estimated. It is also known that the creep rate diminishes with time, but nevertheless specimens loaded in more than two minutes and less than fifteen minutes show no appreciable difference in their stress/strain curves[4].

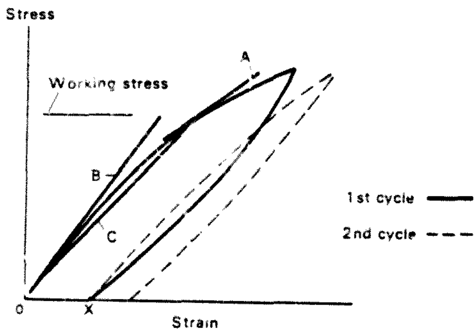

Figure 3.1 Typical Short-Term Stress/Strain Curve for Concrete (after Illston et al[4])

b) the stress is increased to higher levels. Figure 3.2 shows stress-strain curves for rock, concrete and cement paste loaded in uniaxial compression. In general, rocks used as concrete aggregates exhibit approximate linearity up to failure. In contrast, both paste and concrete exhibit marked non-linearity. The non-linearity becomes more pronounced at higher stress levels, particularly approaching failure. The departure from linearity is generally attributed to the development of microcracking. These microcracks start at relatively low stresses and make an increasing contribution to the curvature of the stress-strain curve as the stress level rises. However, as discussed in chapter 2, Maher and Darwin[5] state that the non-linearity of concrete under compressive loading is highly dependent on the nonlinearity of its cement paste and mortar constituents. Cement mortar is not an elastic, brittle material but is nonlinear and damaged continuously under load. These studies seem to diminish microcracking as the most important factor in the nonlinear behaviour of concrete.

It would therefore appear that the elastic modulus for concrete should be quoted with reference to a particular rate of loading and a specified stress level. However, careful definition and a well considered experimental technique can lead to the measurement of values that enable good predictions to be made of immediate strains in concrete after loading.

3.3 ELASTIC MODULUS FOR CONCRETE

The engineering designer has a first responsibility to ensure that his structure will carry imposed loads; this leads him to give priority to the analysis of the stresses in the structure due to external forces. Concrete also undergoes deformation as a result of elastic, creep, shrinkage and thermal strains. These strains are affected in different ways by time dependent factors

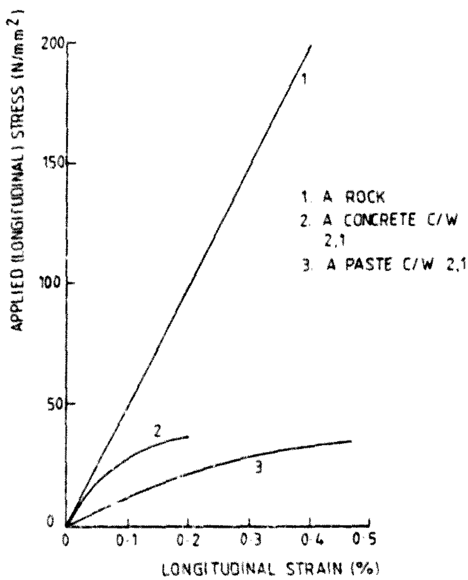

FIG 32 SCHEMATIC STRESS/STRAIN CURVES FOR ROCK, CONCRETE AND CEMENT PASTE (AFTER HOBBS[6])

such as stress, relative humidity and temperature. The reinforced concrete design techniques used a generation ago were relatively simple. The elastic modulus of concrete was taken as 15 GPa and the fact that the actual value was approximately double this figure effectively took care of any shrinkage or creep deformation for normal structures. However, the modern approach is to analyse the various strain components in some depth. Care must be exercised in this situation and sight must not be lost of the fact that concrete is a heterogeneous material and its behaviour cannot be predicted with complete accuracy.

The deflection of a concrete member may conveniently be divided into three components[7] (excluding thermal movements):

1) instantaneous movement induced by applied loads

2) long term movement caused by creep

3) movement caused by drying shrinkage

The modulus of elasticity of concrete is invariably estimated from the concrete strength as discussed later in this chapter. However, this method ignores other factors which influence elasticity. In particular, the modulus of elasticity is dependent upon the moduli of elasticity of the aggregate and the cement paste, and upon the volume concentration of aggregate. The modulus of elasticity of concrete is used in elastic analyses and in creep calculations, so it is important that aggregate elasticity effects are not totally neglected. For example, if a slender bridge were constructed using a high modulus aggregate as opposed to a low modulus aggregate, other mix design properties being equal, deflections and curvatures would be reduced.

Dense aggregates normally used in concrete mixes do not experience creep. Hence, the creep of concrete is determined by the creep of the cement paste and its volume concentration. The magnitude of creep increases as paste content increases. The aggregate effectively restrains creep of the paste, the 'stiffer' the aggregate, the greater the restraint. It follows that for the same cement/water ratio and aggregate concentration, concretes designed with 'soft' aggregates will exhibit greater creep than those made with 'stiffer' aggregates. Creep of lightweight concretes is higher than that of normal weight concretes of the same cement/water ratio, the higher creep being attributable to the lower stiffness of the lightweight aggregate. Various expressions have been suggested to predict concrete creep with time. In all of these equations, the modulus of elasticity of the concrete is involved either directly or indirectly. Hence the elastic modulus of the aggregate which is inextricably linked to that of the concrete will affect deformation calculations.

The coarse aggregate also plays a major role in determining shrinkage movements In certain instances, structures made with dubious coarse aggregate have undergone significantly greater deflections than those made with sound aggregate. The correlation between high shrinkage and low elastic modulus could indicate that these deflections may have been erroneously attributed to drying shrinkage whilst low elastic modulus may, in fact, have been the factor. Hence aggregates previously regarded as unacceptable may be used provided an appropriate value of elastic modulus is used[8].

The contribution of shrinkage to the curvature and deflection of a concrete member is believed to be somewhere between 5 and 30%[9], but greater shrinkage contributions can occur if the applied bending moments are low[10]. Thus, to ascertain reliable predictions for the deflections of reinforced concrete members, estimation of shrinkage must be taken into account. The shrinkage of plain concrete that results from drying is primarily dependent upon the relative humidity of the air surrounding the concrete, the surface area from which moisture can be lost relative to the volume of the concrete, and the mix proportions of the concrete.

Normal dense aggregates for concrete usually have negligible shrinkages. For a paste of given quality, the shrinkage of the concrete will decrease with increasing concentration of aggregate. In fact, there is much similarity in the effect of aggregate on both shrinkage and creep in concrete. The aggregate also restrains the shrinkage of the paste and hence the concrete. The 'stiffer' the aggregate used, the greater the reduction in shrinkage. However, increasing the volume concentration of the aggregate has a greater effect on reducing shrinkage than increasing the rigidity of the aggregate[11]. The shrinkage of concrete may only be 20 percent of that of cement paste of identical cement/water ratio[12].

The effect of aggregate stiffness on the modulus of elasticity and shrinkage of concrete is indicated in an indirect way in figure 3.3 in which shrinkage is plotted against modulus of elasticity of the concrete. It is evident that the shrinkage depends on the modulus of elasticity and decreases with increase in its value. As stated above, the modulus of elasticity depends to a large degree on that of the aggregate. It follows that the shrinkage of concrete will also depend on the modulus of the aggregate. Shrinkage in a lightweight aggregate is greater than that of normal weight concrete. This is supported by figure 3.3 where the shrinkage of lightweight concrete is clearly greater than that of normal-weight concrete. However, for both shrinkage and creep, lightweight and normal-weight concretes of similar strength exhibit the same magnitude of shrinkage[14]. To obtain the same strength, lightweight concrete must be made with a higher cement/water ratio than normal-weight concrete. The higher cement/water ratio reduces shrinkage in the paste, despite the higher cement content which normally accompanies an increase in cement/water ratio. Therefore, the effect of cement/water ratio outweighs that of cement content[15].

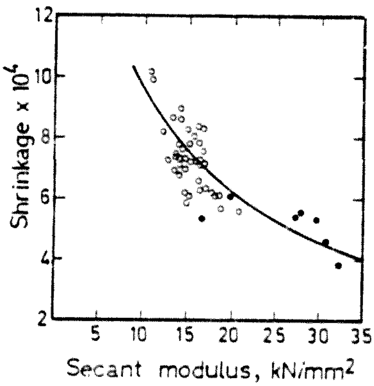

Figure 3.3 Shrinkage of Concrete Plotted Against Its Secant Modulus of Elasticity (after Reichard[13])
●, normal-weight concrete; o, lightweight concrete

Finally, thermal strain occurs in concrete. The coefficient of thermal expansion of concrete is dependent mainly upon the coefficient of expansion of the cement paste, which in turn depends on the moisture content. The coefficient of the concrete is also affected by the coefficient of expansion of the aggregates. Since the coefficient of expansion of the aggregate is less than that of the paste, the coefficient of expansion of concrete will decrease with increase in concentration of aggregate[16]. Thermal strains are calculated from the product of a suitable coefficient of thermal expansion and the anticipated temperature change.

Much of the work carried out to date on the above deformations has been laboratory orientated. The current need is to conduct experimental work in the field and to relate the laboratory measurements to the field results in practical engineering terms. Table 3.1 indicates typical strains that might develop between the start of construction and 30 years of service for a concrete with a 28 day strength of 40MPa loaded at the age of 90 days.

3.4 FACTORS AFFECTING MODULUS OF ELASTICITY OF CONCRETE

There are a number of factors which affect the modulus of elasticity of concrete e.g.

(a) Mix Factors:

1) paste cement/water ratio (and hence strength)
2) paste porosity
3) aggregate concentration and nature
4) stiffness of various phases

(b) Service factors:

1) age of loading
2) moisture content
3) voids and incomplete compaction
4) temperature effects

Table 3.1 Strains predicted for a 40 MPa Concrete (after Parrott[17])

Exposure (in the UK)	Strain components x 10^6				Total strain x 10^6 for a stress of	
	ϵ_{el}	ϵ_{cr}	ϵ_{sh}	ϵ_{th}	0	8 N/mm^2
Internal	26 per N/mm^2	47 per N/mm^2	360	- 140	220	804
External	26 per N/mm^2	29 per N/mm^2	100 $\pm$ 40	$\pm$ 210	- 70 to + 270	370 to 710

where ϵ_{el} = elastic strain

ϵ_{cr} = creep strain

ϵ_{sh} = shrinkage strain

ϵ_{th} = thermal strain

The modulus of elasticity is primarily related to compressive strength and density and can be ascertained from empirical equations. Factors affecting strength also affect the elastic modulus. However, there are exceptions to this, for example, two concretes of similar strength made with different aggregates can have up to a 50% difference in elastic modulus[18].

3.4.1 Aggregate

The modulus of elasticity of the aggregate and its volume concentration both significantly affect the modulus of the concrete. The modulus of elasticity of ordinary concrete, where the aggregate is stiffer than the paste, is greater than that of the paste and increases with increasing aggregate volume concentration and stiffness. When the paste is stiffer than the aggregate, the modulus of elasticity of the concrete is lower than that of the paste and decreases with increasing concentration of aggregate. Generally, the modulus of elasticity of concrete increases with increase in modulus of aggregate[19].

The modulus of elasticity of concrete is also somewhat sensitive to the grading, maximum size and texture of the aggregate. The main influence of these is on the workability of the concrete and hence on aggregate volume concentration. The value of the elastic modulus is partly dependent on progressive microcracking at the paste-aggregate interface. However, these factors are secondary in relation to the modulus of the aggregate, E_a, and the volume concentration of th aggregate.

3.4.2 Paste

The elastic modulus of the hardened cement paste, E_p, also has a significant effect on the modulus of the concrete. The dominant parameter is paste porosity and the modulus of the concrete will decrease markedly as porosity increases, i.e. as cement/water ratio decreases. Porosity changes occur entirely within the paste, and therefore change in the concrete stress-strain behaviour with change in cement/water ratio is due to changes in paste response[20]. The effects of the modulus of the paste, E_p, moisture content, movement of interlayer water and their respective effects on the modulus of elasticity of the concrete are dealt with comprehensively in chapter 2.

3.4.3 Effect of Concrete Strength

Theoretical equations used to predict the modulus of elasticity of concrete are not very practical. They require a knowledge of the moduli of the constituent materials, namely the paste and the aggregate. Consequently, in engineering practice, the modulus is estimated from the compressive strength of the concrete. As stated above, factors affecting strength often affect modulus in a similar way, the exceptions being that a higher E_a results in a higher E_c and this is not necessarily true for strength. Also the wetter the concrete, the higher the E_c (due to capillary pore pressures), whereas an opposite effect is noted for strength. Figure 3.4 shows that paste modulus is related to its strength, and therefore higher strength pastes will impart a higher modulus to the concrete.

Hence, for the same aggregates, it follows that the modulus of elasticity of concrete, E_c will be related to its compressive strength, f_c. An empirical relationship for concrete is in the general form

$$E_c = (k \cdot f_c)^{1/n} \qquad \ldots\ldots (3.1)$$

where E_c = modulus of elasticity of concrete
k = constant depending on specific conditions
f_c = compressive strength of the concrete
n = an integer (generally 2 or 3)

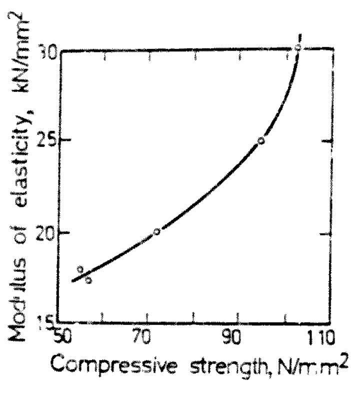

Figure 3.4 Relation between Modulus of Elasticity and Compressive Strength for Cement Paste (after Feldman and Beoudoin[21])

Aggregate properties affect concrete strength. The exact nature of this effect is not always clear, but generally speaking, concrete strength increases with increase in aggregate modulus of elasticity and concentration and with decrease in its particle size. The strength of ordinary concrete is only slightly affected by the strength of the aggregate[22]. These factors, however, affect the modulus of the concrete to a greater extent than their corresponding effects on strength.

3.4.4 Service Factors

The structure of hardened cement paste is sufficiently disordered to ensure that stress-strain relationships are not simple. As stated in section 3.2, the term 'instantaneous' strain rather than 'elastic' strain is more appropriate for concrete even after short loading times. This is because a portion of the strain is not reversible, strain is time dependent and creep occurs. Parrott[23] suggests a formula to estimate the elastic modulus at time of loading, as follows:-

$$E_t = E_{28} \ (0{,}4 + 0{,}6 \ f_t/f_{28}) \qquad \ldots\ldots (3.2)$$

where E_t = modulus of elasticity of concrete at age t days

E_{28} = 28 day modulus of elasticity of concrete

f_t = cube strength at age t days

f_{28} = 28 day cube strength

t = age at loading

Further extrinsic effects include temperature and compaction of the concrete. Monfore and Lentz[24] indicate that at very low temperatures, there is a considerable increase in the modulus of elasticity of concrete. At high temperatures, however, there is a progressive decrease in the modulus of elasticity[25]. Incomplete compaction also affects concrete modulus. Kaplan[26] showed that 5 percent voids can reduce the modulus of elasticity of concrete by 18 per cent.

3.5 EQUATIONS RELATING ELASTIC MODULUS OF CONCRETE AND COMPRESSIVE STRENGTH

The following facts should be borne in mind before the specific equations are stated. There is no direct theoretical basis for a relationship between the compressive strength of the concrete and the elastic modulus of the concrete. All such relationships are empirical and are used for reasons of practicality. The following equations take no cognizance of the affects of age or aggregate properties. The British Standard Code of Practice for the Structural Use of Concrete, CP110[27], and the South African Code SABS 0100[28] state that the static modulus of elasticity is related to the cube strength as follows:-

$$E_c = 9,1\ (f_{cu})^{1/3} \qquad \text{..... (3.3)}$$

where E_c = modulus of elasticity of concrete in GPa
f_{cu} = cube strength of concrete in MPa at time of testing

The CEB/FIP[29] (1970) derive the elastic modulus from cylinder compressive strength. The formula is given by

$$E_c = 6.6\ (f_{cy})^{1/2} \qquad \text{..... (3.4)}$$

where E_c = modulus of elasticity of concrete in GPa
f_{cy} = cylinder strength at the appropriate age in MPa

For lightweight concretes, CEB/FIP recommend the following equation:-

$$E_c = 1.8\ (\rho^3\ f_{cy})^{1/2} \qquad \text{..... (3.5)}$$

where E_c = modulus of elasticity of the concrete in GPa
ρ = density of the concrete kg/m^3
f_{cy} = cylinder strength at the appropriate age in MPa

The American Concrete Institute base their equation on a square root relationship rather than a cube root. The ACI Building code 1977[30] gives E_c in GPa and f_{cy} in MPa. The equation is

$$E_c = 4{,}73\ (f_{cy})^{1/2} \qquad \text{..... (3.6)}$$

This equation was later modified to

$$E_c = 4.9\ (f_{cu})^{1/2} \qquad \text{..... (3.7)}$$

which is based on the cube strength f_{cu} rather than the cylinder strength f_{cy}.

Davis[31] proposed a curve for South African concretes (see figure 3.5(a)). The curve is the line of best fit for approximately 100 experimentally measured points. The test results for all ages from 7 days to one year are plotted. He also showed that age affects compressive strength and elastic modulus differently (see figure 3.5(b)). The curves show that one year after casting, the elastic modulus is approximately 15% greater than at 28 days. However, the compressive strength is 40% greater than the corresponding value at 28 days. Davis considered that the scatter was primarily due to the wide variety of aggregates tested and mix proportions used, rather than the age difference.

The Cement and Concrete Association (U.K.) Development Report No. 3[23] estimates the elastic modulus from a linear relationship incorporating an index of aggregate softness (related to the aggregate) and age effects.

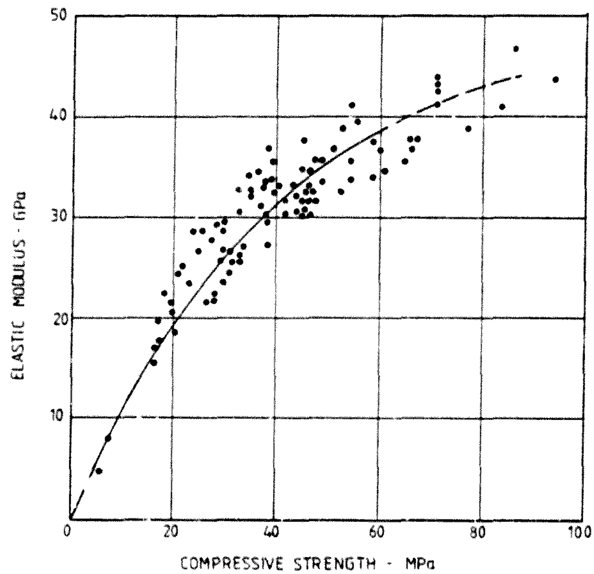

FIG 3.5a RELATIONSHIP BETWEEN COMPRESSIVE STRENGTH AND ELASTIC MODULUS OF CONCRETE (AFTER DAVIS[31])

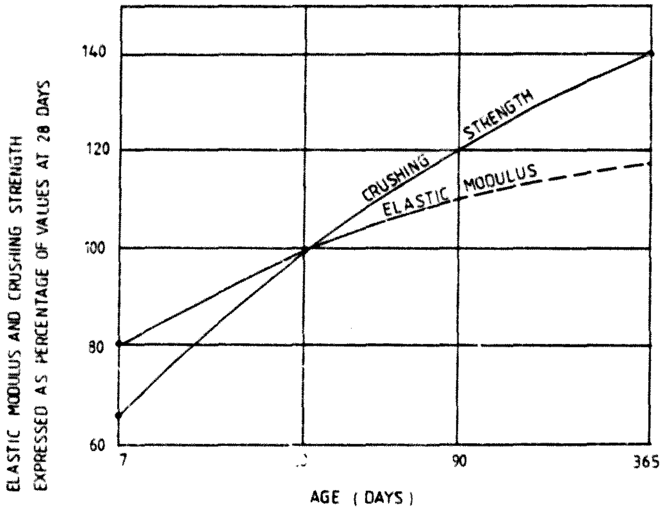

FIG 3.5b ELASTIC MODULUS AND COMPRESSIVE STRENGTH EXPRESSED IN TERMS OF 28 DAY VALUE (AFTER DAVIS [31])

The equation is given in the form,

$$E_{28} = C_o + 0{,}2\ f_{28} \qquad \ldots\ldots (3.8)$$

where E_{28} = 28 day modulus of concrete

f_{28} = the compressive cube strength at 28 days

C_o = factor closely related to the modulus of the aggregate

They propose that C_o has a mean value of 20GPa for normal-weight aggregates and 10 GPa for lightweight aggregates. The modulus of elasticity of concrete at ages other than 28 days is estimated from equation (3.2) quoted earlier in the text. It is also recommended that measured strengths be used as far as possible in equation (3.2), as the use of strength ratios obtained from CP110 or elsewhere would reduce the reliability of the estimated value.

3.6 CURRENT METHODS OF MEASURING STATIC ELASTIC MODULUS

The elasticity of concrete may be assessed by means of cylinder and prism compressive tests and flexural beam bending tests. However, there are differences between moduli measured in these tests. Alexander and Aberdein[32] carried out a series of tests on concrete and mortar under compression and flexure. They found elastic moduli measured in beam tests were about 6 per cent higher than prism moduli, which in turn were about 7 per cent higher than moduli measured in a standard cylinder test. In the case of the beam bending tests, a correction for shear deflection and the Seewald effect, which takes account of deviations from the simple theory of bending in the flexural test, were included. The beam tests yielded a lower spread of values for elastic modulus than the compression tests. The spread of values for the cylinders and prisms were essentially the same. They postulated that the variation in elastic modulus of the different types of test could possibly be explained in terms of the Weibull[33] weakest link theory for the strength of materials. According to this theory, the probability of failure of a specimen is dependent on the stress in the specimen, and the volume of material under stress. For two test pieces identical in every respect except that one has a greater volume of material under maximum stress than its counterpart, the theory infers that the larger test piece will fail at a lower stress than the smaller one. The above argument was applied to stiffness rather than strength, as it is known that isolated cracks are initiated in concrete at a molecular level at very low stresses. The microstructure of concrete can be taken to be a lattice of interlinking aggregate and mortar members and the response of concrete to stress is dependent on the stiffness of the lattice. The authors state that the volume of the larger specimen has a greater probability of 'serious' incipient cracks, and hence a lower stiffness in its lattice structure. It follows that the larger cylinders may be expected to have a lower stiffness than the smaller prisms and beams. Other investigators have shown that the elastic modulus of concrete may depend on, amongst other things, the state of stress in the specimen, specimen and test geometry, and environmental conditions[32].

Hognestad et al[34] in testing prisms and cylinders of similar total volume, found that eccentric compression conditions gave elastic moduli 10 per cent greater than the uniaxial compression condition. They also compared flexural and compression moduli for concrete and found the ratio to be 1,1; they stated the difference between the stress states of flexure and compression accounted for the different moduli. The above considerations must be borne in mind as the writer has used prisms for elastic modulus determination and not cylinders, which are the normally accepted standard. The prisms were used for reasons of economy and ease of casting, as well as to be consistent with the fact that prisms were used to determine elastic moduli for both the pastes and mortars.

3.6.1 Test Procedures

Test procedures for the determination of the static modulus of elasticity of concrete in compression are described in BS 1881: Part 121 : 1983[35] and ASTM C469[36]. Either a secant or a chord modulus is determined. In the American ASTM C469 standard, the stress level at which the modulus is determined is 40 per cent of the ultimate strength of the concrete. In BS 1881 the stress level is one third of ultimate. The ASTM method stipulates a chord modulus from a lower point on the stress/strain curve corresponding to a strain of 50×10^{-6} to an upper point corresponding to a stress of 40 per cent of the ultimate. The lower point is near the origin, but far enough away to be free from any irregularities. The upper point is in the upper range of working stress. Hence it covers the working range of concrete stresses to be encountered in a structure.

The British BS 1881 method stipulates an application of a basic stress of 0,5 MPa, whereafter the stress is increased steadily at a constant rate of $0{,}6 \pm 0{,}4$ N/(mm^2.s) until a stress of one third of the compressive strength of the concrete is reached. This stress is maintained for 60s and strain readings taken during the succeeding 30s. The load is then reduced at the same constant rate to the level of the basic stress, and the specimen recentred if strain readings show that this is necessary. Two additional preloading cycles, using the same loading and unloading rate and maintaining the same basic and upper loading stress constant for a period of 60s are carried out, before a final cycle from which the elasticity measurements are taken.

For paste, mortar and concretes all static elastic modulus tests were carried out according to the principles of BS 1881: Part 121: 1983. The recommended rate of loading was adhered to as closely as possible, as well as the periods of sustained basic and upper loading stress.

3.7 LABORATORY PROCEDURE

At the time the project was undertaken actual values of fineness modulus (FM), relative densities of the aggregates (RD) and compacted bulk densities (CBD) were not measured. However, average values were assumed (private communication M.G. Alexander) as given in table 3.2. The assumptions made in table 3.2 were sufficiently accurate for mix design purposes. Mixes were prepared for each rock type using c/w ratios of 1,5, 1,8 and 2,4. (See also additional note on page 142(a)).

3.7.1 Mixing Moulding and Curing Procedures for Concrete Mixes

The basis of the concrete mix design was as follows. A normal mix (based on Portland Cement Institute method) was designed using the average values of FM and RD. The main objective of this was to arrive at a representative mortar. Then using the basic mix parameters for this mortar, the stone content was varied to arrive at the various concrete mixes. This is best illustrated by showing a typical calculation for a stone content of roughly 1000 kg/m^3, ie designing a concrete of normal proportions in order to abstract representative mortar proportions.

Table 3.2 Physical Properties used for Mix Design

	GRANITE	ANDESITE
FINENESS MODULUS	3,1	3,5
C.B.D. (kg/m^3)	1520	1600
R.D.	2,55	2,85
NOMINAL STONE SIZE (mm)	19	19
SAND DENSITY (kg/m^3)	2650	2850
WATER REQUIREMENT (l/m^3)	195	195

where C.B.D. is compacted bulk density (kg/m3)

R.D. is relative density

In the event actual FM values were lower (see Appendix A). However, this was not critical, since the aim was to design representative mortars combined with three different stone contents, namely 700 kg/m^3, 1000 kg/m^3 and 1300 kg/m^3. This aim was satisfactorily achieved in the mixes.

In the example the following symbols with reference to one cubic metre of concrete, are used:-

M_c	= mass of cement (kg/m^3)	V_w	= volume of water
M_{sa}	= mass of sand (kg/m^3)		
M_{st}	= mass of stone (kg/m^3)	ρ_c	= density of cement (kg/m^3)
V_{st}	= volume of stone (m^3)		
V_m	= volume of mortar (m^3)	ρ_w	= density of water (kg/m^3)
c/w	= cement/water ratio		
Sa/c	= sand/cement ratio	ρ_{sa}	= density of stone and sand (kg/m^3)
M_w	= mass of water (kg/m^3)		

c/w = 1,5 volume = $1 m^3$		mass (kg)		absolute volume (m^3)
water (R.D. = 1,0)		195	$\frac{195}{1000}$	= 0,195
cement (R.D. = 3,14)	195 x 1,5	= 293	$\frac{293}{3140}$	= 0,093
stone (R.D. = 2,75)	$\frac{1010 \times 1560}{1520}$	= 1037	$\frac{1037}{2750}$	= 0,377
		Total abs vol:		0,665

Therefore, sand = 1 - 0,665 = 0,335 m^3
(R.D. = 2,75)
sand = 0,335 x 2750 = 921 kg

(Note that an average value of sand and stone R.D. for the granite and andesite sands is being used).

Mortar ratios:-

(1) Ratio's by mass

$c/w = 1.5$

$Sa/c = \frac{921}{292,5} = 3,15$

$M_{st} = 1000 kg/m^3$

therefore $V_{st} = \frac{1000}{2750} = 0,3636$

therefore $V_m = 1 - 0,3636 = 0,6364\ m^3$

therefore $V_m = M_w \left(\frac{c}{w} \times \frac{1}{\rho_c} + \frac{1}{\rho_w} + \frac{Sa}{c} \times \frac{c}{w} \times \frac{1}{\rho_{sa}} \right)$

$0,6364 = M_w \left(1,5 \times \frac{1}{3140} + \frac{1}{1000} + 3,15 \times 1,5 \times \frac{1}{2750} \right)$

$0,6364 = M_w (3,196 \times 10^{-3})$

therefore $M_w = \frac{0,6364}{3,196 \times 10^{-3}} = 199\ kg$

therefore

$M_c = 1,5 \times 199 = 299\ kg$

$M_{sa} = 3,15 \times 299 = 942\ kg$

$M_{st} = 1000\ kg$

2440 kg

(2) Ratio's by volume

$c/w = 0,477$

$Sa/c = 3,60$

Now using these quantities we can derive the proportions required for the granite and andesite concretes, adjusting mix proportions so as to have equal volumes of stone in the mix.

a) granite

water		=	199 kg
cement		=	299 kg
sand	$942 \times \frac{2650}{2750}$	=	908 kg
stone	$1000 \times \frac{2650}{2750}$	=	964 kg

b) andesite

water		=	199 kg
cement		=	299 kg
sand	$942 \times \frac{2850}{2750}$	=	976 kg
stone	$1000 \times \frac{2850}{2750}$	=	1036 kg

Each mix was designed according to the above approach and then fractioned down to the required batch size. For each of the cement/water ratios ie 1,5, 1,8 and 2,4 and two rock types three coarse aggregate contents were used, i.e. approximately 700 kg/m^3, 1000 kg/m^3 and 1300 kg/m^3. The various mix proportions are shown in tables 3.3 and 3.4.

For the dynamic test thirty six 100 x 100 x 500 mm long concrete beams were cast, which when cut in half provided a total of seventy two 100 x 100 x 250 mm long prisms for the respective static tests. Therefore, for each dynamic test there were two specimens per cement/water ratio and stone content and four for the static test. Four cubes were also cast per cement/water ratio and stone content.

These cement/water ratios and the intermediate stone content were chosen such that the most commonly used structural grade concretes would be represented. The primary reason for the range of stone contents was to see how the elastic moduli obtained from them compared with the theoretical model predictions given in Chapter 4. At 700 kg/m^3 the volume concentration of the stone is quite low and so there is little or no aggregate interaction under load i.e. the aggregate is dispersed through the mortar matrix. At the stone content of 1000 kg/m^3, typical of most structural concrete, there is a certain amount of aggregate interaction and at 1300 kg/m^3 (approaching the maximum possible stone content), there is a greater percentage of aggregate contact. Figures 3.6(a), (b) and (c) show cross sections through a typical andesite concrete of cement/water ratio 1,8 for stone contents of 700, 1000 and 1300 kg/m^3 respectively. (Area marked C, shows aggregate contact in each mix).

Trial mixes of 500 kg/m^3 and 1400 kg/m^3 were carried out and both found to be unsatisfactory. The 500 kg/m^3 mix when cast and cut open was found to contain all its aggregate in the bottom

Table 3.3 Mix Proportions for Granite Concrete

COARSE AGGREGATE CONTENT kg/m^3	C/W RATIO	WATER kg	CEMENT kg	SAND kg	STONE kg
700	1,5	233	350	1100	700
	1,8	233	420	1039	700
	2,4	233	560	916	700
1000	1,5	199	299	908	964*
	1,8	199	359	854	964*
	2,4	199	480	753	964*
1300	1,5	165	248	751	1252*
	1,8	165	297	709	1252*
	2,4	165	399	626	1252*

* The difference between the prescribed and actual stone contents are due to average values being used for the mix design calculations

Table 3.4 Mix Proportions for Andesite Concrete

COARSE AGGREGATE CONTENT kg/m^3	C/W RATIO	WATER kg	CEMENT kg	SAND kg	STONE kg
700	1,5	233	350	1180	751*
	1,8	233	420	1117	751*
	2,4	233	560	986	751*
1000	1,5	199	299	976	1036*
	1,8	199	359	918	1036*
	2,4	199	480	810	1036*
1300	1,5	165	248	808	1347*
	1,8	165	297	762	1347*
	2,4	165	399	673	1347*

* The difference between the prescribed and actual stone contents are due to average values being used for the mix design calculations

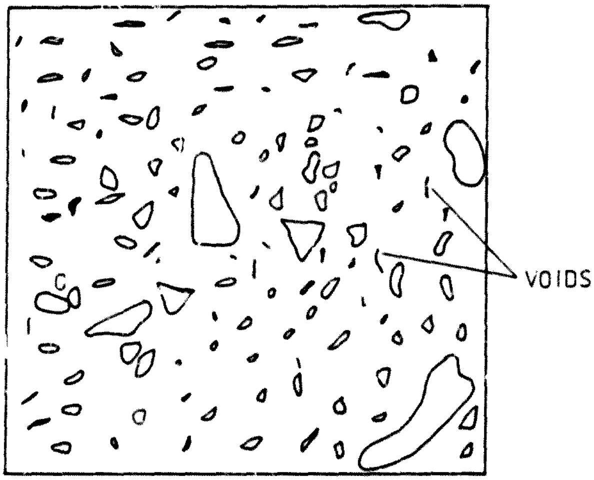

FIGURE 3.6 (a) CROSS SECTION SHOWING AGGREGATE PARTICLES IN ANDESITE CONCRETE, c/w = 1,8, 700 kg/m^3 STONE CONTENT
C = INTERPARTICLE CONTACT

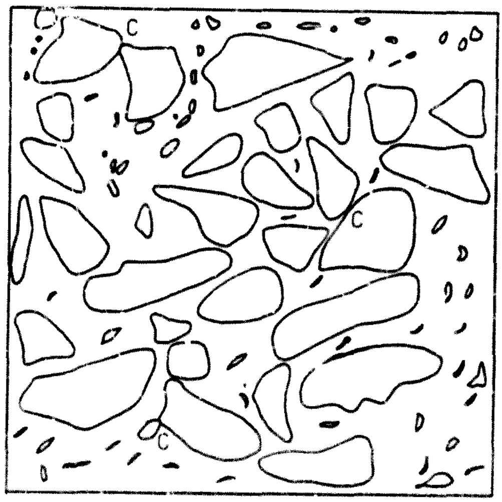

FIGURE 3.6 (b) CROSS SECTION SHOWING AGGREGATE PARTICLES IN ANDESITE CONCRETE, c/w = 1,8, 1000 kg/m^3 STONE CONTENT
C = INTERPARTICLE CONTACT

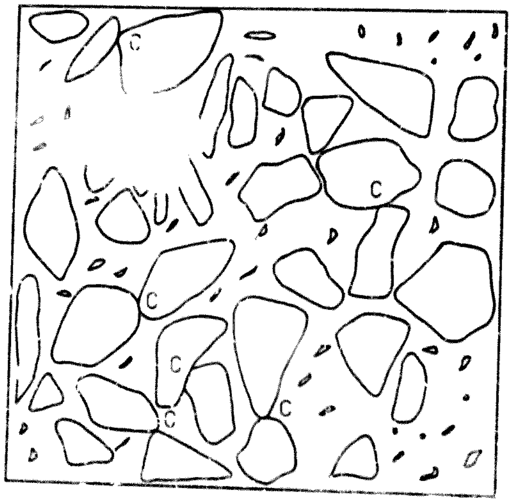

FIGURE 3.6 (c) CROSS SECTION SHOWING AGGREGATE PARTICLES IN ANDESITE CONCRETE, c/w = 1,8 1 300 kg/m3 STONE CONTENTS
C = INTERPARTICLE CONTACT

half of the test beam while the 1400 kg/m^3 mix was exceptionally difficult to compact and the top 30% of the beam resembled a 'no-fines' concrete due to insufficient mortar to fill the voids between the aggregate.

Ordinary Portland cement, complying with SABS 471 (1971), was used for all concretes. The cement was received fresh from the manufacturer in one batch and each pocket was then stored in an air-tight drum in the laboratory. This batch of cement was used for all the tests carried out in this investigation. The static and dynamic testing of each mix was carried out 28 days after casting. All cubes were also tested in uniaxial compression at 28 days after casting.

The main aim of the laboratory testing was to ascertain the effect of increasing volume concentration of aggregate on elastic moduli. The higher the stone content, the greater should be the measured elastic modulus. The mortar and paste mix ratios were identical to the respective mortars and pastes reported in Chapter 2, therefore, only the stone content was varied for the respective cement/water ratios. These measured moduli could then be compared to the estimated theoretical model predictions. It is thought that at the higher stone content where there is definite interparticle contact, the model equations might break down. This is because a main assumption of the model equations is that the concrete is a two-phase material of aggregate dispersed in a mortar matrix, each aggregate particle acting independently of its neighbour.

3.7.2 Mixing Procedure

All the solid materials were weigh batched on a laboratory balance to an accuracy of 100g. The water was volume batched using a measuring cylinder graduated in 20 ml intervals.

The materials were introduced into a 100 kg pan mixer in the following order:

i) Sand
ii) Cement
iii) Stone
iv) Water

The dry materials were first mixed by approximately three turns of the paddles and, with the mixer activated, the water was added over a period of one minute. Mixing was then continued for a further three minutes. At the end of this period, the paddles were raised and, to ensure an even distribution of the constituent materials, the mixture was turned several times using a hand scoop. This manual mixing was necessary as it was found that, after mechanical mixing, the coarse aggregate tended to be concentrated more at the center of the mixing pan. Also in the high stone content mixes, particularly the andesite, the mortar adhered to the side of the pan mixer. Therefore, to ensure a representative mix, further hand mixing was carried out.

3.7.3 Slump and V.B. Tests

For each mix, standard slump and V.B. tests were performed. Three such tests were carried out for each mix. After each slump and V.B. test, the concrete was returned to the pan mixer and re-mixed for a short period. The results of the slump and V.B. tests are given in table 3.5 for granite and andesite mixes. It can be seen that in general the slump and V.B. for each mix were reasonably consistent, implying consistency in the experimental procedure.

Table 3.5 Slump and V.B. Times for Granite and Andesite Mixes

C/W RATIO	STONE CONTENT (kg/m^3)	GRANITE SLUMP (mm)	GRANITE V.B. (s)	ANDESITE SLUMP (mm)	ANDESITE V.B. (s)
1,5	700	50	2,5	90	3,5
		45	3,5	85	2,5
		50	4,0	90	2,5
	Average	48	3,3	88	2,8
1,8	700	75	2,5	95	2,0
		75	3,0	85	3,0
		65	3,0	85	2,5
	Average	72	2,8	88	2,5
2,4	700	70	3,0	50	4,0
		75	3,0	50	3,5
		80	3,5	65	3,5
	Average	75	3,2	55	3,7
1,5	1000	25	6,5	20	4,0
		15	6,5	10	4,5
		25	5,5	10	3,5
	Average	20	6,0	13	4,0
1,8	1000	15	5,0	25	3,5
		10	6,5	10	5,5
		5	10,0	25	5,0
	Average	10	7,0	20	4,7
2,4	1000	20	7,0	20	7,0
		15	6,5	20	6,5
		20	8,0	20	9,0
	Average	18	7,2	20	7,5

Table 3.5 Slump and V.B. Times for Granite and Andesite Mixes (continued)

C/W RATIO	STONE CONTENT (kg/m^3)	GRANITE SLUMP (mm)	GRANITE V.B. (s)	ANDESITE SLUMP (mm)	ANDESITE V.B. (s)
1,5	1300	0	5,0	0	4,0
		0	4,0	0	6,0
		0	5,0	0	5,5
	Average	0	4,7	0	5,2
1,8	1300	0	5,5	0	4,5
		0	5,5	0	5,0
		0	5,0	0	4,5
	Average	0	5,3	0	4,7
2,4	1300	0	4,5	0	7,0
		0	5,5	0	10,0
		0	6,0	0	7,0
	Average	0	5,3	0	8,0

3.7.4 Compaction

The 100 x 100 x 500 mm beam moulds were half filled with concrete and tamped approximately 50 times, before being vibrated on a mechanical vibrating table for the durations given in table 3.6. This compaction procedure was also applied to the 101,6 mm cube moulds, although they were hand held on the mechanical vibrating table and tamped approximately 25 times per layer. The moulds were filled in two layers.

Table 3.6 Vibrating Times for Various Stone Contents, Granite and Andesite Beam Moulds

STONE CONTENT kg/m^3	DURATION OF VIBRATION (PER HALF LAYER)
700	20 s
1000	60 s
1300	2-2,5 min

3.7.5 Curing

After compaction, the concrete in the moulds were covered with plastic sheets to prevent evaporation of water from the exposed surface. Approximately 18 hours after casting the hardened beams and cubes were removed carefully from their moulds and placed in a water filled curing bath. By means of an immersion heater and a small circulation pump the temperature in the curing bath was held constant at 22 ± 1°C. All concrete specimens for this investigation were stored in a curing bath until the time of testing. Due to evaporation and the removal of specimens from the bath, it was necessary to periodically add water to the bath to keep the concrete, the heater and the water pump submerged.

3.7.6 Testing

A brief outline of the testing procedure is given here. However, a fuller procedure is outlined in Appendixes C, F and E for static, ultrasonic and electrodynamic testing respectively. Before being tested, the beams and cubes were removed from the curing bath. After excess water was wiped off, each specimen was weighed to an accuracy of 0,1g. The beam specimens were dimensioned physically in accordance with Appendix C i.e. the length measured in four places and six measurements for both breadth and depth, by means of a vernier scale. The test beams were scribed with centre-lines and gauge lengths to facilitate central positioning of the specimens and fixing of the static LVDT rigs. The sequence of testing was as follows:-

3.7.6.1 Ultrasonic Test

The ultrasonic tester shown in figure 3.7 was calibrated using the standard 10 μs calibration bar. The 100 x 100 x 500 mm long beam specimens were placed between the ends of the ultrasonic transducers. Care was taken to ensure that the ends of the specimens were clean and surface dry before coupling their ends to the transducers with petroleum jelly. The transducers were held firmly at each end and a reading taken. This was repeated three times for each specimen and an average taken for calculation purposes.

3.7.6.2 Electrodynamic Test

The test beams were removed from the curing tank and their ends cleaned of any water and grit. The beam was then clamped into the electrodynamic apparatus and the electrodynamic exciter unit was brought into contact with the centre of the beam end. Similarly, the pick-up was positioned on the opposite face.

Figure 3.7 Concrete Beam in Ultrasonic Test Apparatus
T = Transducers

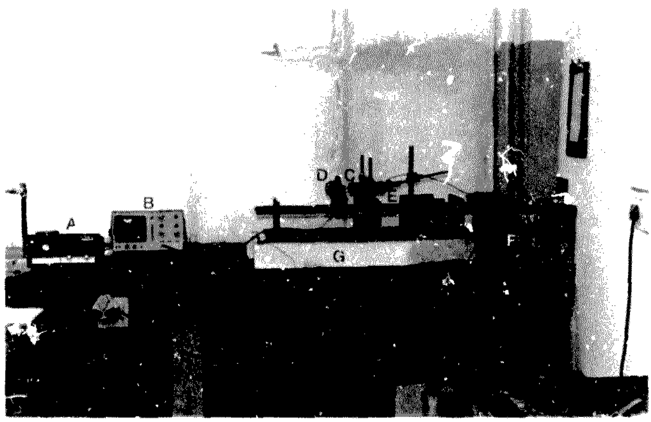

Figure 3.8 Paste Beam under Test in Electrodynamic Apparatus
A = Variable Frequency Oscillator
B = Oscilloscope
C = Specimen
D = Driver
E = Pick-up
F = Power Source (D.C.)
G = Damping Layer

Figure 3.8 shows the test set up (for a paste prism). The variable frequency oscillator was switched on and the frequency varied until a peak output trace was observed on the oscilloscope, indicating the fundamental frequency of the test specimens. The fundamental frequency was subsequently read off the frequency counter. This procedure was repeated three times and a mean resonant frequency recorded. It is recommended that the operator exercises care to ensure the 'peak' trace observed on the scope is that of the fundamental mode, and not a harmonic.

3.7.6.3 Static Test

After the beams were tested ultr[illegible]ically and electro-dynamically, they were cut in ha[illegible]sing a diamond-tipped saw blade. Both ends were ground plane to ensure parallel faces for the static test. The static rig was fixed onto the specimen as shown in figure 3.9 and the specimen positioned centrally in a Tinius Olsen compression machine. Both the LVDTs and the electronic load output from the compression machine were connected to an X-Y recorder. Figure 3.10 shows the overall experimental set up. The load output was read directly from the Tinius Olsen machine. The pre-load given in Appendix C was applied and the LVDTs zeroed by means of a multimeter connected across the X-Y recorder. The load was then increased at 15 MPa/min up to one third of the mean compressive strength. The load was held at this value for one minute and then decreased at the afore- mentioned loading rate to the value of the pre-load. The X-Y recorder pens for longitudinal and transverse deformation were re-zeroed and the operation repeated for two successive plots. The static elastic modulus was determined from the third plot.

Finally, the pens were re-zeroed and at the above loading rate the specimen was loaded to failure. The X-Y recorder calibrations were set so as to give adequate sensitivity for each plot.

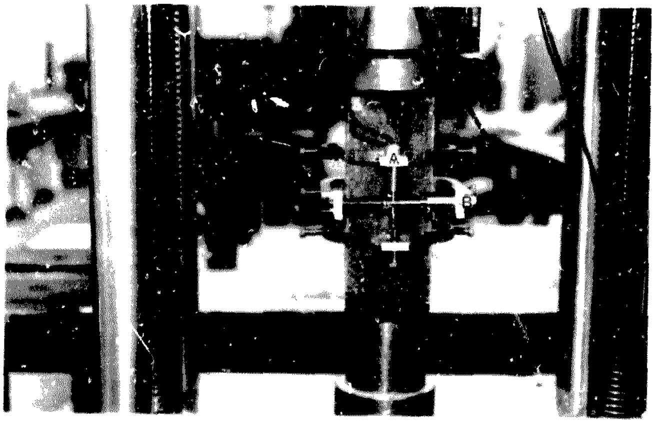

Figure 3.9 Concrete Specimen held in Static Rig
A = LVDT to measure longitudinal strain
B = LVDT to measure lateral strain

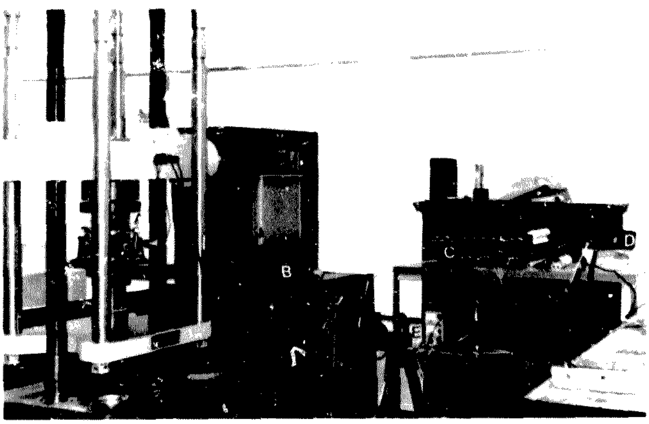

Figure 3.10 Static Test Equipment
A = Specimen held in rig
B = Tinius Olsen Compression machine
C = XY Plotter
D = Multimeter
E = D.C. Power source

3.7.6.4 Cube Strength

After weighing, the cubes were tested on an Amsler type 103 compression testing machine (capacity 2000 kN). The cubes were centred in the machine and loaded to failure at a loading rate of 15 MPa/min. The failure load was recorded to the nearest 1 kN. The results of the cube tests are given in appendix C.

3.8 RESULTS

The results of the static, electrodynamic and ultrasonic elastic modulus tests are given in Appendix C, E and F respectively. The final column of the tables indicates the mode of failure of the various specimens on being loaded to failure.

3.9 DISCUSSION OF RESULTS

Table 3.7 shows 'E' load (i.e. the load corresponding to one third the expected failure strength of the concrete) and the expected failure stress for each of the cement/water ratios. The expected failure stresses are based on cube strength and not prism strength. The actual measured cube and prism failure strengths are tabulated in table 3.8(a) and (b) for both granite and andesite concrete respectively. The results show that the prism compressive strengths are consistently lower than the expected failure stress and vice versa for cube strengths, especially the andesite mixes. The strengths for granite are lower than their andesite counterparts. At the low cement/water ratios and stone contents of 1000 kg/m^3 and 1300 kg/m^3, the percentage difference of the granite prisms is -97,2% and -70,7% respectively, compared with only -30,8% and -37,9% difference in the respective andesite mixes. This may be due to the better bond properties of the andesite aggregate giving rise to higher strengths than their granite counterparts. However, overall the prism compressive strengths are an average of -43% lower than expected failure strengths for the granite mixes (neglecting the results for cement/water ratio 1,5 and stone content 1000 and 1300 kg/m^3) and -34% lower for andesites. These results would help to provide an explanation for the prism results recorded in chapter 2 for pastes and mortars. The prism compressive strengths measured were also considerably lower than expected failure stresses. Concrete is clearly not as sensitive to aspect ratio as pastes; however the prism compressive strengths are consistently low.

Engineering texts[37,[illegible]] discuss comparisons mainly between cube and cylinder strength. In this project prisms were employed. However, several factors affecting cylinder strength also apply to the case of prisms. For example, the restraining effect of the testing machine platens extend over the entire height of a cube, but leaves part of the prism unaffected. This would tend to give higher cube strengths than prism strengths. Although the ratio of cube strength/cylinder strength decreases as strength of concrete increases, it is often assumed that cube strength is 1,25 times that of a cylinder. It is also known that as the specimen size increases, so strength and variability of strength decrease[38].

The aspect ratio (ratio of height to least lateral dimension h/d) also plays an important role. For values of h/d smaller than 1,5 measured strength increases rapidly owing to the aforementioned platen restraining affect. Between a h/d ratio of 1,5 and 4,0 strength is little affected, while for an h/d ratio greater than 5 strength falls off markedly and slenderness effects become apparent. The concrete prisms tested in this project had an aspect ratio of 2,5.

At the lower cement/water ratios combined with the higher stone contents, particularly in the granites, there is a possibility that we may be measuring a different material behaviour with

Table 3.7 Elastic Modulus 'E' Load and Expected Failure Stresses for Concrete

C/W RATIO	'E' LOAD (kN)	EXPECTED FAILURE STRESS (MPa)
1,5	96,3	28,0
1,8	126,0	36,5
2,4	178,9	52,0

Table 3.8(a) Mean Cube and Prism Strengths for Granite Concretes

C/W RATIO	MEAN CUBE COMPRESSIVE STRENGTH (MPa) STONE CONTENT kg/m^3			MEAN PRISM COMPRESSIVE STRENGTH (MPa) STONE CONTENT kg/m^3		
	700	1000	1300	700	1000	1300
1,5	28,5	30,1	31,4	20,5	14,2	16,4
% difference from expected	+1,8	+7,0	+10,8	-36,6	-97,2	-70,7
1,8	37,9	43,5	42,9	25,8	26,2	24,8
% difference from expected	+3,7	+16,1	+14,9	-41,5	-39,3	-47,2
2,4	58,9	62,8	65,6	38,1	34,8	34,8
% difference from expected	+11,7	+17,2	+20,7	-36,5	-49,4	-49,4

Table 3.8(b) Mean Cube and Prism Strengths for Andesite Concretes

C/W RATIO	MEAN CUBE COMPRESSIVE STRENGTH (MPa) STONE CONTENT kg/m^3			MEAN PRISM COMPRESSIVE STRENGTH (MPa) STONE CONTENT kg/m^3		
	700	1000	1300	700	1000	1300
1,5	33,5	35,1	38,3	23,5	21,4	20,3
% difference from expected	+16,4	+20,2	+26,9	-19,1	-30,8	-37,9
1,8	43,7	46,0	48,8	29,1	29,5	26,6
% difference from expected	+16,5	+20,7	+25,2	-25,4	-23,7	-37,2
2,4	63,7	64,6		38,8	35,1	35,1
% difference from expected	+18,4	+19,5		-34,0	-48,1	-48,1

$$\% \text{ DIFFERENCE} = \frac{\text{MEASURED - PREDICTED}}{\text{MEASURED}} \times 100$$

NOTE Both the cube and prism strength values are an average of four test specimens modulus and Poisson's ratio for granite and andesite concrete respectively. The results infer that the modulus is not adversely affected by the low prism strengths. If a different material behaviour were being measured, we would expect that low strength would also result in a low modulus. However, this is clearly not the case. Therefore, the modulus results are acceptable.

3.9.1 Static Elastic Modulus

The results of the static modulus tests are given in appendix C. The average static elastic moduli are tabulated in table 3.9(a) and (b) for granite and andesite concrete respectively. The results show consistent moduli values for each mix.

Figure 3.11(a) and (b) show the plots of cube strength against static elastic moduli for granite and andesite concrete respectively. The plots clearly show that the stiffer aggregate has a higher modulus and compressive strength. The lines fitted to the results are linear regression lines. It is very interesting that both the lines for granite and andesite are parallel. They only differ at the intercept ordinate on the static modulus of elasticity axis (the C_0 value as defined in section 3.5). From the graphs, C_0 for granites is 18,8 GPa and for andesite C_0 is 27,1 GPa). From the graphs we can derive two equations relating the 28-day cube strength f_{cu} and the 28-day static modulus of elasticity E_{28} for granite and andesite concrete. The relationship illustrated in figure 3.11(a) for granite concrete may be expressed in terms of the equation:

$$E_s \text{ (GPa)} = 0{,}210 f_{cu} + 18{,}84 \qquad \text{..... (3.9)}$$

and for andesite as illustrated in figure 3.11(b):

$$E_s \text{ (GPa)} = 0{,}213 f_{cu} + 27{,}13 \qquad \text{..... (3.10)}$$

Table 3.9(a) Average Static Modulus and Poisson's Ratio Values for Granite Concrete

STONE CONTENT kg/m³	C/W RATIO	AVERAGE STATIC MODULUS E_c GPa	AVERAGE STATIC POISSON'S RATIO
700	1,5	23,46	0,17
	1,8	29,93	0,18
	2,4	29,78	0,18
1000	1,5	24,24	0,16
	1,8	27,70	0,16
	[illegible]	31,37	0,20
1300	1,5	24,98	0,16
	1,8	29,30	0,17
	2,4	33,24	0,20

Table 3.9(b) Average Static Modulus and Poisson's Ratio Values for Andesite Concrete

STONE CONTENT kg/m³	C/W RATIO	AVERAGE STATIC MODULUS E_c GPa	AVERAGE STATIC POISSON'S RATIO
700	1,5	[illegible]0,81	0,20
	1,8	[illegible]2,93	0,20
	2,4	36,21	0,21
1000	1,5	34,50	0,20
	1,8	37,34	0,21
	2,4	40,60	0,22
1300	1,5	39,66	0,20
	1,8	42,43	0,22
	2,4	44,35	0,21

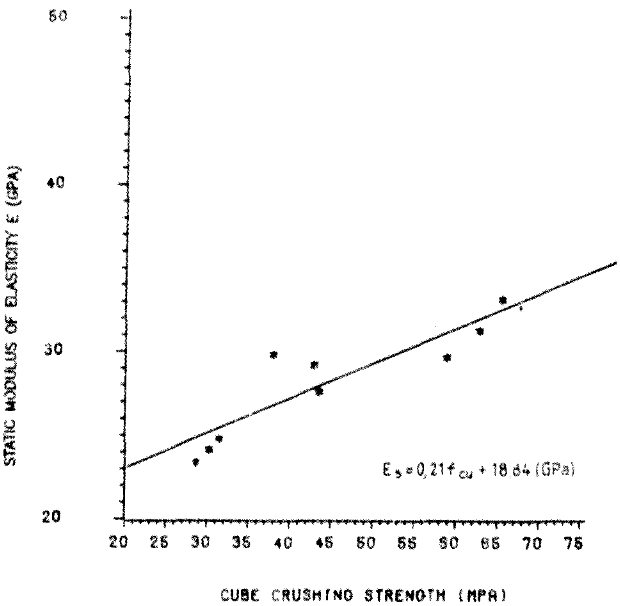

FIG 3.11a CUBE STRENGTH VS. STATIC MODULUS (GRANITE CONCRETE)