STABILIZATION OF LATERITE WITH BITUMINOUS EMULSION

By

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DECLARATION

I hereby declare that this research work was original and solely carried out by me. No part of it has been tendered for any previous degree, and all sources of information are duly acknowledged.

Signature.....

Date. ....18—7—94.....

CERTIFICATION

This thesis titled " STABILIZATION OF LATERITE WITH BITUMINOUS EMULSION " by Bajeh Ibrahim meets the regulation governing the award of Master of Science (Geotechnical Engineering) by ABU, Zaria, and is approved for its contribution to knowledge and literary presentation.

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I am very grateful to all those whose efforts have been very vital to the success of this work. My profound gratitude goes to my supervisor, Dr. K.J. Osinubi whose contribution was very important, and also to my colleagues whose understanding had been essential.

This work was aimed primarily at stabilising A-7 laterite using bituminous emulsion (MS?) as the main stabilizer Also, MC1 and cement stabilizers were as well used for comparative purposes. The prooperties of the stabilized soils were determined by means of compaction, CBR, unconfined compression and water absorption tests©

From the analysis of the results, only cement was able to stabilize the soil to meet the specifications for base course. It is conclusive that 3 to % of mulsion (MS2) and MCI ara economically suitable for the stabilization of the oil to achieve the specifications for sub-base materials only while a maximum cement content of 3% is desirable for base course.

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CHAPTER 1

I N T R O D U C T I O N

1.0 General Introduction: Laterites, as soil rather than a well defined material, are most commonly found in the leached soils of the humid tropics. They are formed as a result of weathering process typical of laterization; the important factor being the decomposition of ferro-alumino silicate minerals and the deposition of the sesquioxides of iron and aluminium (Gidigasu, 1976).

Most lateritic soils in their natural states are at best suitable mainly as sub-base course materials where roads are expected to carry heavy wheel loads. This constraint in the availability of standard base course materials may be overcome by designing the road to fit the sub-standard site material, or replacing the sub-standard material with a standard fill material. Alternatively, the unsuitable site material may be improved by the application of additives which are normally mixed with soil in predetermined economical proportions to achieve favourable modifications of the soil properties (i.e strength properties). This process is termed stabilization.

Different types of stabilizers have been used in different parts of the world to enhance the engineering properties of laterite (Moh, 1962). The most common ones are cement, lime and bitumen (Rahaman, 1968). However, the choice of a particular stabilizer depends on the soil type and degree of effectiveness of the stabilizer. It will be the target of the construction engineer to use an economic percentage of a stabilizer to achieve the desired strength, permeability and durability of a base course.

Bitumen can stabilize soils by acting either as a water-proofing agent (sealant) or a binding agent (lubricant/adhesive). In the waterproofing applications, bitumen is mixed with a cohesive soil which has a useful mechanical strength at a given moisture content. In such a case, the strength of the soil is preserved by the bitumen which seals the system against moisture changes. Such applications are uncommon. Most often, bitumen is applied as a binding agent for non-cohesive materials which may range from a dense continously-graded crushed rock system to local sand (Williams, 1968).

Cement is commonly used as a stabilizer bacause of its effectiveness. Ola (1974), concluded that cement is the most effective stabilizer of all the stabilizers for the greatest range of lateritic soils found in Nigeria.

Lime, though often used as a stabilizer, is very costly. As such, it was not considered in this research work which was aimed at providing a cheap stabilizer for a particular case of a lateritic soil, A-7 type.

1.1 Literature Review:

A lot of work had been done in soil stabilization, and the results of some of them are briefly discussed in this section.

In designing a road structure which comprises a layer of stabilized soil with a bituminous surfacing, it is first necessary to establish whether the structure will behave as a rigid or as a flexible base, that is, whether the stabilized soil acts in a way similar to a material like a hard core or crushed stone (Maclean et al, 1952). The essential difference between the two types of base is that the rigid base has sufficient flexural strength to enable it to bridge local areas of the sub-grade which are not affording full support to the base, whereas the flexible base has a small flexural strength with the result that the base remains in full contact with the sub-grade at all times. This means that a rigid base acts as a perfectly elastic material when subjected to stress conditions for which it has been designed (Maclean et al 1952). The deformation of the sub-grade, on the other hand, is generally not entirely recoverable owing to the plastic flow of the soil. In the case of a flexible base, the material has a very small flexural strength similar to that of a soil, with the result that after the application of a loaded wheel, the base assumes a permanent deformation equal to that of the sub-grade.

Hubert (1953), in his soil stabilization work, concluded that in the consideration of the properties of different types of stabilized soils that are commonly used, only cement-stabilized soil possesses a significant flexural strength, the values in the case of bitumen-stabilized and mechanically-stabilized soils being negligible. Thus, the latter two types of materials behave as flexible materials.

The two principal tests used in Great Britain for measuring the strength of stabilized soils are the test associated with the California Bearing Ratio (CBR) method of pavement design and the test for measuring the unconfined compressive strength (Maclean et al, 1952).

In the CBR test, the standard material which was employed by the originators of the test was a well-compacted sample of crushed stone which is known to be a satisfactory base material, when covered with a bituminous surfacing, for roads carrying heavy traffic. Reference to the CBR design curve for main roads shows that a base is required to have a CBR of only 60% percent to give a satisfactory performance when it is covered with a bituminous surfacing that is about 10cm thick (Hubert, 1953).

The unconfined compression test is normally used in stabilization mainly to investigate the suitability of soils for stabilization with cement, for which it is usual to measure the strength of specimens cured at a constant moisture content

and temperature for 7 days. Hubert (1953), concluded that under these conditions, a minimum laboratory strength of 1760 KN/m^2 has been found to be a suitable criterion for ensuring satisfactory performance of the material in the construction of secondary roads. Furthermore, he surmised that a field strength at 7 days of about 1050 KN/m^2 has proved adequate for a soil-cement base covered with only a surface dressing.

Another significant factor to be considered in soil stabilization is the weathering effect (Carter, 1944). Whether the CBR or the unconfined compression test is used, the effect of climatic factors can be examined by suitably modifying the test procedure. This is necessary to examine the softening effect which can occur in bases under a pervious surfacing. The weathering effect is normally simulated in the laboratory by immersing the specimens in water for a period before measuring their strength. Under an impervious surfacing, the base material in many climates tends to an average moisture content which depends on the ground water condition (Hubert, 1953).

There are several soil stabilizers in use but this work is restricted to three; cement, MC1 and MS2.

In soil stabilization using portland cement, the proportions of cement added to the soil usually range between 5 and 15 percent by weight of the soil (Maclean et al, 1952). Secondary additives are sometimes necassary to produce a stabilized soil with the required characteristics; these are usually employed in proportions

of 1 to 2 percent by weight of the soil. When mixed with soil, Portland cement usually hydrates normally to form what may be regarded as a large number of rigid bonds in the soil (Hubert, 1953). In soils like laterites which contain a certain amount of clay it is not possible, with the normal proportion of cement used, to do more than bond aggregations of the particles so that the stabilized soil comprises a combination of rigid cement bonds, which are unaffected by water, and of plastic soil bonds whose strength is reduced by an increase in their moisture content. Maclean (1952) surmised that the unconfined compressive strength of soils stabilized with 10 percent of normal Portland cement may range at an age of 7 days from about 700 KN/m^2 with well-graded granular soils to less than 140 KN/m^2 with some heavy clay soils, when they are cured at a constant moisture content. In the case of stabilized soils immersed in water, the corresponding strength of granular soils should be slightly affected, but a more pronounced reduction in strength is usually obtained in the case of stabilized clay soils (Maclean et al, 1952).

Hubert (1953) surmised that in order to keep the strength of the soil within the prescribed limits, the cement content of the stabilized soil should be about 5 percent or even less. Higher percentages of cement will lead to stronger materials which can be liable to form more widely spaced and therefore more open cracks that do not permit adequate interlocking, and thus leading

to a seriously defective road structure. In the stabilization of lateritic soils, an improvement in the properties of the stabilized soil can be achieved by the addition of about 2 percent of hydrated lime (Carter, 1944). This also has the effect of reducing the plasticity of the clay and making the soils more amenable to mixing with cement.

There are two main forms of stabilizing soils with bitumens, the object in one form being to obtain a material with a higher stability than that of the soil itself and in the second to increase the resistance of soil to the absorption of water and so assist in the preservation of the natural stability it possesses when in a dry and well-compacted condition (Hubert, 1953). Various laboratory investigations have shown that bitumens are most successful in providing increased stability in the case of clean granular soils such as gravels and sands.

In the case of laterites and other clayey soils, the function of bitumens appear to be more of replacing the cohesion imparted by the pore water by the higher cohesions of films of bitumen (Hubert, 1953). To achieve a satisfactory result in bituminous stabilization of clayey soils, less than 6 percent of bitumen, which is either rapid- or medium-curing type has been recommended (Williams, 1968). The increased resistance of clayey soils to the absorption of water is believed to be caused by the formation of a film of bitumen on the surface of water in the pores of the soil.

The entry of water into the soil is hindered by the resistance of this bitumen film to displacement.

Paraffin wax added to bitumens helps to form films of greater rigidity, and waxed bitumens have been marked especially for the waterproofing of lateritic soils (Hubert, 1953). These are usually employed with 1 or 2 per cent of hydrated lime whose function is to flocullate the clay particles in the soil, and thus bring the soil to a condition more suitable for the effective use of the bitumen.

The choice of stabilizing soils normally result from economic considerations. In Nigeria where lateritic soils are common and easy to win, it may be cheaper to resort to soil stabilization rather than hauling gravel, crushed stones or other fill materials over a considerable distance. Cement and bituminous materials, mostly cutback bitumen and shelmac, are common stabilizers that have an extensive application in this country (Ola, 1977). These stabilizers act by improving the soil, thereby preventing the absorption of water which can lead to failure in the strength of a road structure.

Johnson et al (1964), in a road construction work in northern Nigeria, concluded that bituminous materials are particularly favourable in the stabilization of cohesionless sand which could be problematic if a cement stabilizer is used.

In a bituminous stabilization work carried out by Williams (1968), he concluded that a clean well-graded material is

advantageous to use. Furthermore, he asserted the choice of using bituminous materials over cement in stabilizing sand that is predominantly single-sized with a coefficient of uniformity that is less than 3. An inherent problem in such a soil type is the lack of internal friction or cohesion.

Dunn (1971) obtained satisfactory results from a soil stabilization work and concluded that lateritic soils can be stabilized effectively by using about 4 to 7 per cent cement for a base construction. However, for sub-base materials, Gidigasu (1976) suggested 1 to 3 per cent bitumen, 0.5 to 4 per cent cement or lime as modifiers and 4 to 6 per cent bitumen as waterproofing agent. He further suggested that in soils having liquid limits that range between 20 and 60 per cent, as in tropical clayey gravel soil, the soil can be stabilized to meet the standard for road bases by adding about 4 percent by weigth of either cement or hydrated lime.

Though qualities of bituminous stabilization that are normally sought include provision of cohesion, resistance to soaking or swelling, and the ability to penetrate and adhere to the soil, one major problem is, however, the vagary of climate (Lee, 1974). Practical difficulties have occurred in areas subjected to frequent, high or irregular rainfall (Katty et al, 1960; Edwards, 1968).

1.2 Importance of the Project:

In many countries, including Nigeria, where lateritic gravels are found abundantly, such gravels are good for gravel roads. However, they are too plastic and variable to provide a standard road base under a bituminous surfacing. Owing to lack of suitable base and subgrade materials, burrow pits are normally employed, which may be accompanied by large haulage distances that can increase the cost of contruction tremendously. For this reason, stabilization of available lateritic soils, which are common, is important.

A lot of stabilizers are currently being used. They include cement, lime, bitumen, calcium chloride, sodium silicate, fly ash, etc. However, some of them, for example lime are very expensive. Since Nigeria is a petroleum-producing country and has a lot of bitumen as a by-product, which is relatively inexpensive, there is need to adopt bituminous stabilization, especially in the northern parts of the country where sandy and lateritic soils are common.

In Nigeria, bituminous stabilization using cutback or shelmac is largely in use (Ola, 1977). But the use of bitumen emulsion for the stabilization of lateritic soils is not well documented. Hence, this research work was directed towards an effective use of bitumen emulsion, MS2, for stabilizing A-7 laterite.

1.3 Aim of the Project:

Primarily, this project was aimed at improving the strength and other desirable properties of A-7 laterite using bitumen emulsion, MS2. In view of this aim, it was hoped that the following objectives would be met:

(i) To find the range of percentage of bitumen emulsion needed to stabilize the laterite to meet the fill, sub-base and base specifications.

(ii) To find out an economic percentage of the emulsion needed for effective stabilization, and

(iii) To draw a parallel between the results of using an emulsion for stabilization and those of using MC1 and cement.

CHAPTER 2

METHODOLOGY AND SOIL STABILIZATION TECHNIQUES

This section entails the laboratory method of investigation. The methodology and some common stabilization techniques normally used are described below.

2.1 Methodology:

Figure 2.1 is the geological map of Nigeria showing the study area. Samples of laterite were taken from burrow pits at depths of 1m below the ground surface, to avoid organic matter, at the new hospital site A.B.U., Zaria. Various laboratory tests were performed in accordance with the British Standard (BS) or American Society of Testing and Materials (ASTM).

These tests include the following:

(a) Moisture content and spedific gravity tests,

(b) Atterberg limit tests,

(c) Grain size analysis,

(d) Determination of the properties of the bituminous stabilizers,

(e) Compaction tests to determine the relationship between the dry density and percentage of stabilizer content as well as the relationship between the optimum moisture content and percentage of the stabilizer content,

(f) CBR tests for soaked and unsoked conditions,

(g) Unconfined compression tests for soaked and unsoaked. samples and

(h) Water absorption tests.

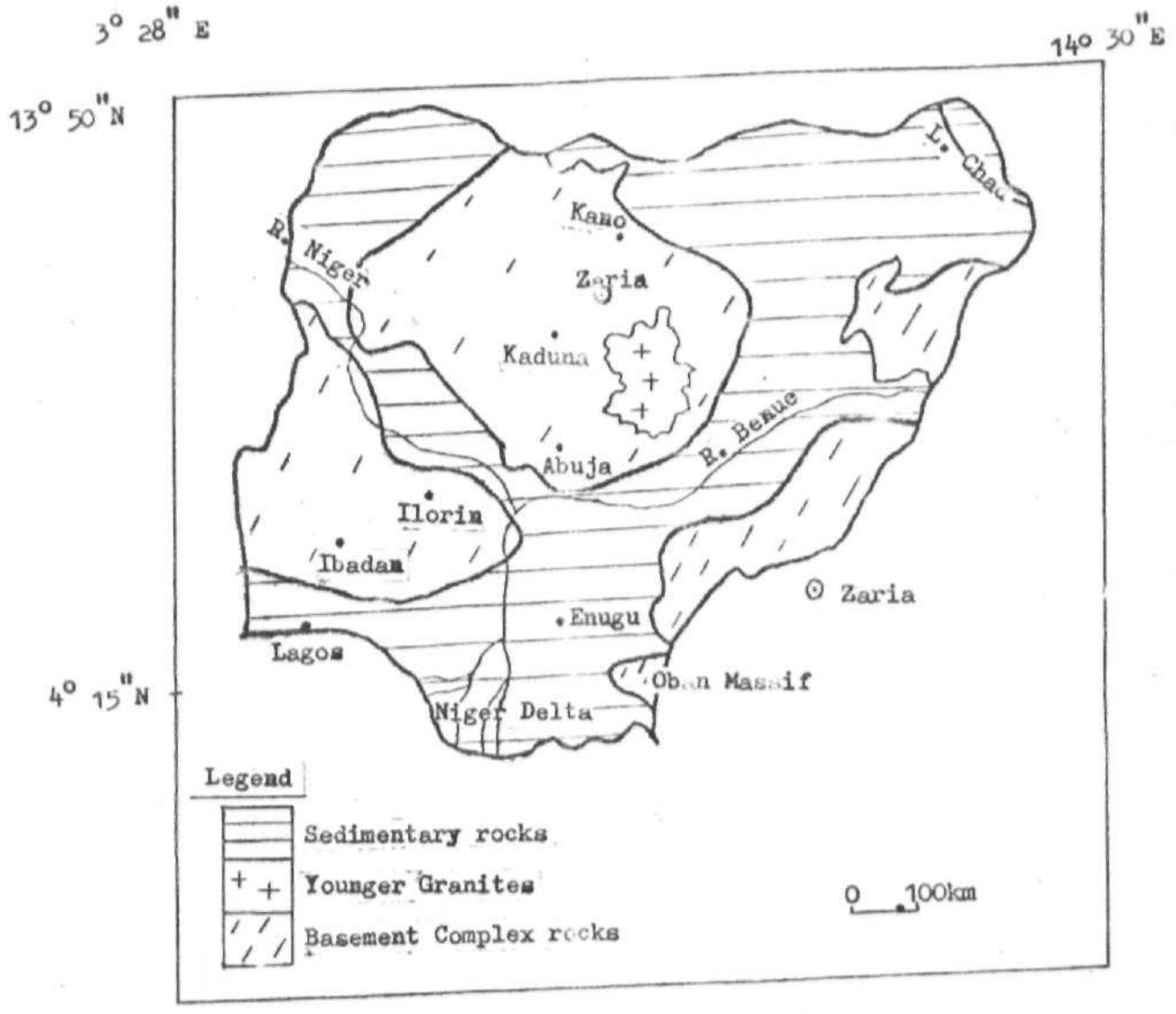


FIG. 2.1: Geological map of Nigeria (showing the site of sam collection) (After Kogbe, 1973).

The percentages of the stabilizers used were: 0, 1, 3, 5, 7 and 8. Higher percentages were not necessary because the bituminous stabilizers will tend to saturate the soil samples.

2.2 Modes of soil Stabilization:

There are several methods of stabilizing a soil. A particular method to be employed depends on the properties of the stabilizer to be used.

Sandy and silty soils may be stabilized by drainage method or the injection of slurries or solutions into the voids of the soil. These injected materials harden to varying degrees and impart cohesion to the injected soil. Since they partly fill the voids, these stabilizers reduce the soil permeability (Moh, 1962).

The injection of cement grout in stabilization has shown that the method may lead to satisfactory results only if the soil is relatively homogenous and unstratified, and if the grain size is not too small (Schroeder, 1975). The grout will not penetrate the voids of a loose soil with an effective size D_{10} less than about 0.5mm or a dense soil with an effective size less than about 1.5mm, thus cement grouting is not appropriate for soils much finer than coarse sands.

Although grouting soils with clay slurries has been attempted, it was found that the penetration of the slurries is impeded by the formation of a filter skin that seals the voids (Moh, 1962). The development of the filter skin appears to be strongly influenced by the electrolytes present in the ground water; this

influence introduces an element of considerable uncertainty regarding the effectiveness of the procedure. Though grouting with clay slurries may greatly reduce the permeability of a sand, it does not significantly increase the strength.

Stabilization of soils by the injection of chemicals has a wide practice. A common procedure comprises the successive injection of solution of water glass and calcium chloride which react with the soil to form a cohesive binder (Schroeder, 1975). In a variation of this method, a single solution is injected containing a buffer that delays and controls the time of setting. These methods are highly successful in clean relatively homogenous sands having an effective size greater than 0.1mm, but the efficacy of the procedures decreases rapidly as the grain size or the homogenity of the sand decreases. Also it depends greatly on the chemical composition of the groundwater.

Furthermore, considerable use can be made of polymers that are mixed with catalysts and retarders before injection and that react after a lapse of time to form an almost impervious gel (Schroeder, 1975). However, all the injection procedures are expensive and, even under favourable conditions, are uncertain.

Generally, if the coefficient of permeability of a soil lies about 10^{-4} and 10^{-6} cm/sec, no injection procedure is satisfactory (Schroeder, 1975). In such a case, stabilization may be achieved by electro-osmosis, which is an electrical method

for creating seepage pressures and producing consolidation.

It may sometimes be possible to render soils impervious and stable by freezing the water in the voids (Schroeder, 1975). This method has been used extensively in sinking shafts or in tunnelling, but it has also been successfully employed in the temporary stabilization of a large landslide in silt. Freezing is accomplished by circulating a refridgerant through a series of double pipes inserted in the soil to be stabilized. The process of freezing may take several weeks or months, as such, this method of stabilization can be very expensive.

Laboratory stabilization methods can be classified generally into five groups. They are:

(1) Proportion:

This is the proportioning of soils, after compaction, to give the desired results.

(2) Modifying Agents:

These modyfying agents improve the soil enhancing its performance. An example of a modifying agent is lime.

(3) Cementing Agents:

When cementing agents like cement, lime, or fly ash are used to stabilized a soil, they increase the strength and stability of the soil. Bituminous stabilizers add binding qualities to non-cohesive soils like sand.

(4) Water-proofing and Repellant Agents:

Bituminous materials are normally used for the purpose of waterproofing (Schroeder, 1975). They minimize the

with a repelling agent such as vinsol resin, which prevents water from infiltrating the stabilized soil by coating the soil particles, the stabilized soil will be waterproof.

(5) Water Retaining Agents:

For sufficient stability and strength, some non-cohesive soils need a minimum amount of moisture below which they become loose and less stable (Schroeder, 1975). Some additives such as calcium chloride, when added to a soil will continue to absorb moisture from the atmosphere and thereby maintaining the optimum moisture content of the soil. Also, treatment with calcium chloride reduces dust nuisance on laterite roads.

CHAPTER 3

LABORATORY TESTS

3.0 Tests Carried Out:

As earlier stated in chapter one, the essence of this research work is to determine the effectiveness of stabilizing A-7-6 laterite using a bituminous emulsion (MS2) instead of the more conventional types of stabilizers. The properties of the stabilized soil will be determined and correlated with some recommended standard for base, sub-base and fill materials.

In view of the above aim, preliminary tests were carried out on the soil samples. These tests included the determination of the natural moisture content, specific gravity, particle size distribution and Atterberg limits of the soil. Further tests after stabilizing the soil included the compaction test (Standard Proctor), water absorption tests, California Bearing Ratio (CBR) test and unconfined compression test. Also a brief identification test was carried out on the MC1 and MS2 stabilizers used. The results are presented in the following sections.

3.1 Determination of the natural moisture content and specific gravity of the soil:

Tests were conducted in accordance with BS1377: part 2: 1990. The natural moisture content and also the specific gravity of three samples of the soil were determined. The averages of the results were calculated and presented as shown in data sheets 1 and 2 (see Appendix A). A summary of these results is as shown in table 3.1.

Table 3.1: Natural moisture content and specific gravity of the soil.

Type of test	Samples			Average
	A	B	C	
Natural moisture content (%)	6.60	6.70	6.80	6.70
Specific gravity of the soil	2.39	2.48	2.45	2.44

3.2 Determination of particle size distribution of the soil:

By using grain size analysis, the particle size distribution of the soil was determined. The procedures of analysis used were those of combined analyses, which entail sieve and hydrometer methods.

The sieve method consists of shaking the soil through a stack of wire screens with openings of known size ; the definition of particle diameter for a sieve test is, hence the definition of a square hole (Lambe, 1951). The hydrometer method is based on Stoke's equation for the velocity of a freely falling sphere. The definition of particle diameter for a hydrometer test is, hence, the diameter of the sphere of the same density which falls at the same velocity as the particle in question. The combined analysis employs both the sieve and hydrometer tests, therefore the definition of particle size is the size of a square opening for the larger grains and the diameter of the equivalent sphere for the smaller soil particles

(Lambe, 1951). For the general procedures employed, see BS 1377; Part 2: 1990.

The result of the sieve analysis is recorded in data sheet 3 (see Appendix A) while that of the hydrometer (sedimentation) analysis is presented in data sheet 4. The combined results are presented on data sheet 5.

To calculate the particle diameter, D(mm), at a given time, T(min.), in hydrometer analysis, Stoke's law was used as follows:

$$D = K\left(\frac{L}{t}\right)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.1$$

where $K = \dfrac{18n}{\gamma_s - \gamma_w}$ 3.2

n = the viscosity of the suspending medium,

γ_s and γ_w are the unit weights of the soil solids and water respectively.

L = effective depth.

Let P be the fraction of particle finer than D. Hence P can be expressed as:

$$P = \frac{aR_c}{W} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.3$$

$$\text{where } a = 1.53 - 0.2G_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.4$$

$$R_c = R_a + C_m + M_t - X + C_w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.5$$

R_c = Corrected hydrometer reading

R_a = Reading at the top of miniscus

C_m = Miniscus correction

X = Dispersing agent correction

C_w = Water density correction

G_s = Specific gravity of the solid soil.

W = Oven dry mass of sedimentation sample

M_t = Temperature correction.

For the hydrometer No. 773122 used,

$M_t = 0.5$

$X = 4.0$

$C_w = 1.8$

$C_m = 0$

From equation 3.5, hence,

$$R_c = R_a - 1.7 \quad \text{..... 3.6}$$

$$\text{Fraction passing the 2mm sieve} = \frac{M1}{M0}$$

where M_o = Gross mass of soil

M_1 = Mass passing through 2mm sieve.

The graphical presentation of the result of sieve analysis is as shown in figure 3.1.

For the sieve analysis of the soil, the fraction of the soil retained on a sieve is expressed as:

$$\text{Fraction retained} = \frac{\text{mass retained on sieve}}{M2}$$

where M2 = mass used in sedimentation analysis.

fraction finer (of total sample) = (1-cummulative fraction retained).

From hydrometer analysis,

$$\text{Fraction finer (of total sample)} = P. \frac{M1}{M0}$$

For the combined analysis,

$$\text{Uniformity coefficient, } Cu = \frac{D60}{D10} \quad \text{..... 3.7}$$

$$\text{Coefficient of curvature, } Cc = \frac{(D_{30})^2}{D60 \times D10} \quad \text{..... 3.8}$$

where D10= maximum particle size of the smallest 10% (by weight) of the sample (effective size)

D30 = maximum particle size of the smallest 30%

D60 = maximum particle size of the smallest 60%

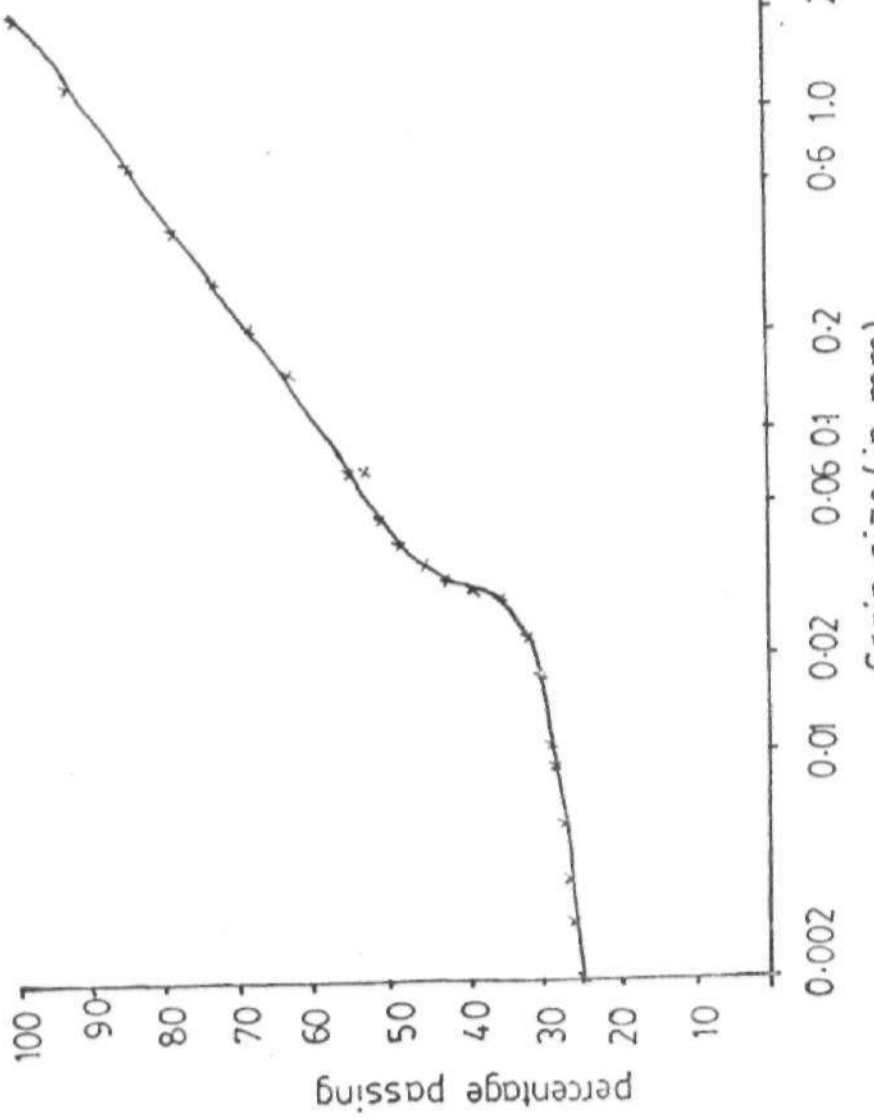


FIG. 3.1: Particle size distribution of A-7 Laterite.

3.3 Determination of the Atterberg limits:

The liquid limit is the water content at which the soil has such a small shear strength that it flows to close a grove of standard width when jarred in a specified manner. The plastic limit is the water content at which the soil begins to crumble when rolled into threads of specified size (Smith, 1978). The plasticity of a soil is measured by the plasticity index, which is equal to the liquid limit minus the plastic limit.

The procedures for these tests are in accordance with B.S 1377; Part 2; 1990

The liquid limit (LL) can be expressed as:

$$LL = W_N\left(\frac{N}{25}\right)^{0.121} \quad \ldots\ldots\ldots\ldots\ldots\ldots 3.9$$

where W_N = the water content of the soil which closes in N blows in the standard liquid limit device.

From the lab data, LL= 42.0%(Moisture content corresponding to 25 blows).

$$\text{Plasticity index, Ip, } = LL - PL \quad \ldots\ldots\ldots\ldots 3.10$$

where PL = Plastic limit (24.9%).

$$\text{Group Index} = (X-35)\left\{0.2 + 0.005\,(LL-40)\right\} + 0.01\,(X-15)(Ip-10) \ldots . 3.11$$

where X= percentage passing No. 200 sieve, expressed as a whole number.

The results of the Atterberg limit tests are recorded in data sheet 6. A summary of the properties of the soil determined is shown in table 3.2.

Table 3.2: Summary of the properties of the soil.

Test	Result
Liquid limit, LL	42.0%
Plastic limit, PL	24.9%
Plasticity Index, Ip,	17.1%
Group Index	7
Specific gravity, Gs	2.44
Natural water content	6.7%

Since the plastic limit (PL) is less than 30, the soil is classified as A-7-6 laterite.

3.4 Determination of the properties of the bituminous emulsion (MS2) used:

The test was performed in accordance with B.S 434:1984. The summary of the results is shown in table 3.3.

Table 3.3: Results of the properties of MS2 in comparison with specifications:

Test carried out	Results	Specifications (O'Flaherty,1974)
Test on emulsion		
Viscosity, Saybolt Furol at 25° (sec)	100	100min
Storage stability test, 24-hr (%)	0.5	1 max.
Sieve test (%)	0.08	0.10max.
Residue by distillation (%)	65	65 min.
Test on residue from distillation:		
Penetration at 25°C, 100g, 5 sec.	140	100-200
Ductility at 25°C, 5cm/min,cm	45	40 min.
Solubility in trichloroethylene,%	98.0	97.5min.

3.5 Determination of the properties of the Cutback bitumen (MC1) used:

The procedure for this test is in accordance with BS 434; 1984. A summary of the results is as shown in table 3.4 that follows. The lab result of the specific gravity determination is presented in data sheet 7.

Table 3.4 Results of the propertiesof the MC1 in comparison with specifications:

Test carried out	Results	Specifications (Edwards, 1968)
Flash point (°C)	90	37.8
Viscocity at 50°C (secs)	75	75
Solubility in CCl_4 (%)	99.0	99.5
Specific gravity	1.01	0.97-1.04

3.6 Determination of the maximum dry density and optimum moisture content (using the standard Proctor compaction test):

For the detailed procedure used, see B.S. 1924; Part 1: 1990. The relationships between the moisture content, bulk density and dry density are as follows:

$$Y_b = \frac{W_w + W_s}{V} \quad \text{.....3.12}$$

$$Y_d = \frac{W_s}{V} \quad \text{.....3.13}$$

$$W = \frac{W_w}{Ws} \quad \text{.....}$$

where

Y_b = bulk density of the soil

Y_d = dry density of the soil

W = moisture content of the soil

W_w = weight of water in soil

W_s = weight of dry soil

V = volume of mould (or soil in the mould)

From equation 3.14,

$$W_w = wW_s$$

substituting the above expression in equation 3.12,

$$Y_b = \frac{W_s}{V}(1 + W) \quad \text{.....} \quad 3.15a$$

$$Y_d = \frac{Y_b}{1 + W} \quad \text{.....} \quad 3.15b$$

The graphs of dry density plotted against moisture content are presented in Appendix B. Summaries of the results of the Proctor compaction tests on the soil stabilized with bitumen emulsion, MC1 and cement are as shown in tables 3.5-3.7.

Table 3.5: Effects of emulsion on compaction

Stabilizer content(%)	OMC(%)	MDD (kg/m^3)
0	19.2	17.0
1	18.9	17.1
3	18.4	16.9
5	17.9	16.5
7	17.5	16.0
8	17.3	16.2

Table 3.6: Effects of [illegible] on compaction

Stabilizer content (%)	OMC (%)	MDD (kg/m^3)
0	19.2	17.0
1	17.8	17.1
3	17.0	16.9
5	16.5	16.5
7	16.0	16.4
8	15.9	15.8

Table 3.7: Effects of cement on compaction

Stabilizer content (%)	OMC (%)	MDD (kg/m^3)
0	19.2	17.0
1	19.9	16.9
3	20.4	16.7
5	20.8	16.6
7	21.0	16.3
8	21.2	16.0

The plot of the maximum dry density versus the stabilizer content (%) is presented in figure 3.2. Also, the plot of the optimum moisture content (%) versus the stabilizer content (%) is shown in figure 3.3.

3.7 The California Bearing Ratio (CBR) test:

The CBR test was developed as an ad hoc test by the California State highway Department for the evaluation of subgrade strengths, and was later extended to cover fill, sub-base and base materials. The test is a measure of the shearing resistance of soil under

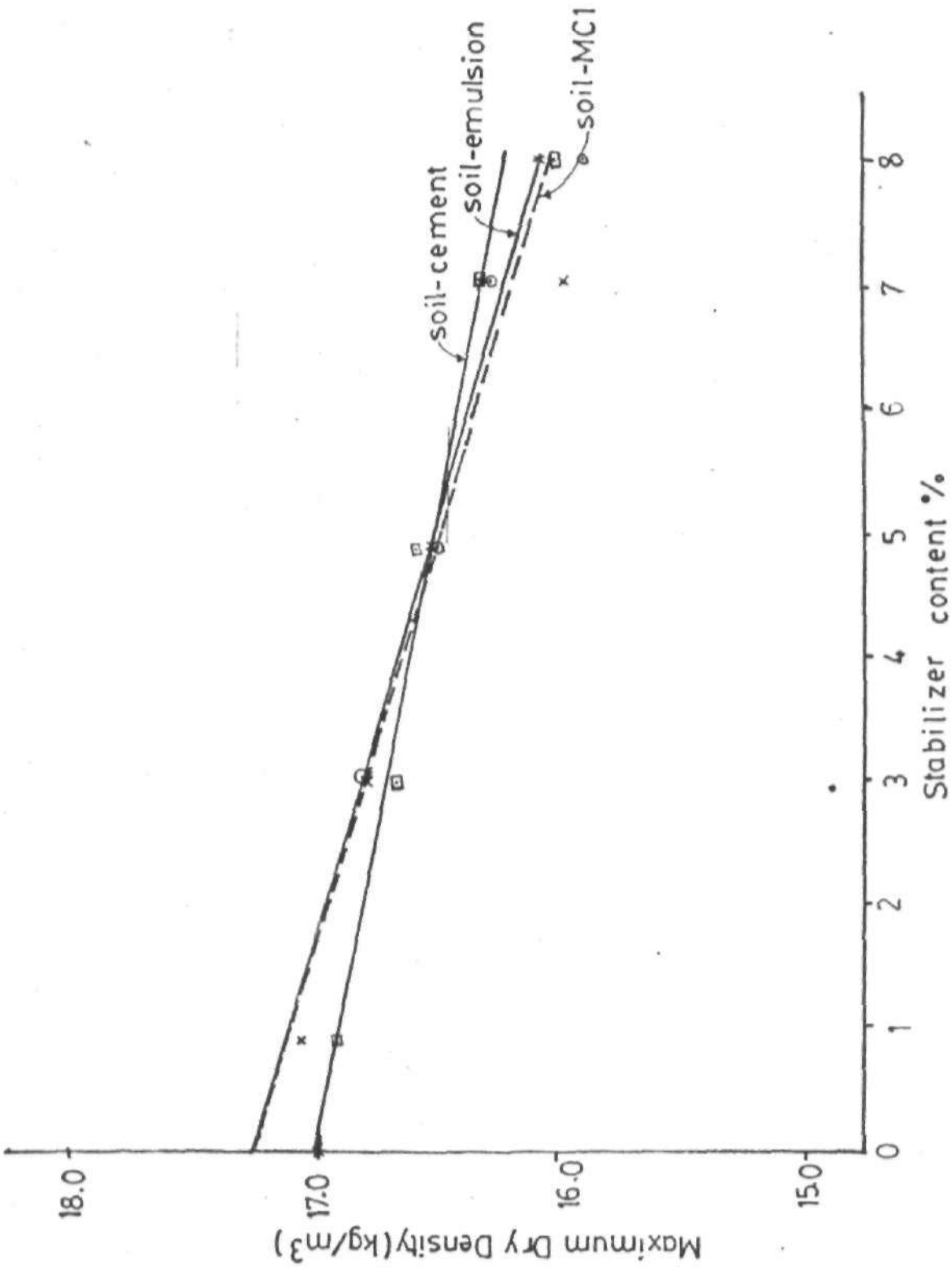


FIG. 3.2: Variation of maximum dry density with stabilizer content.

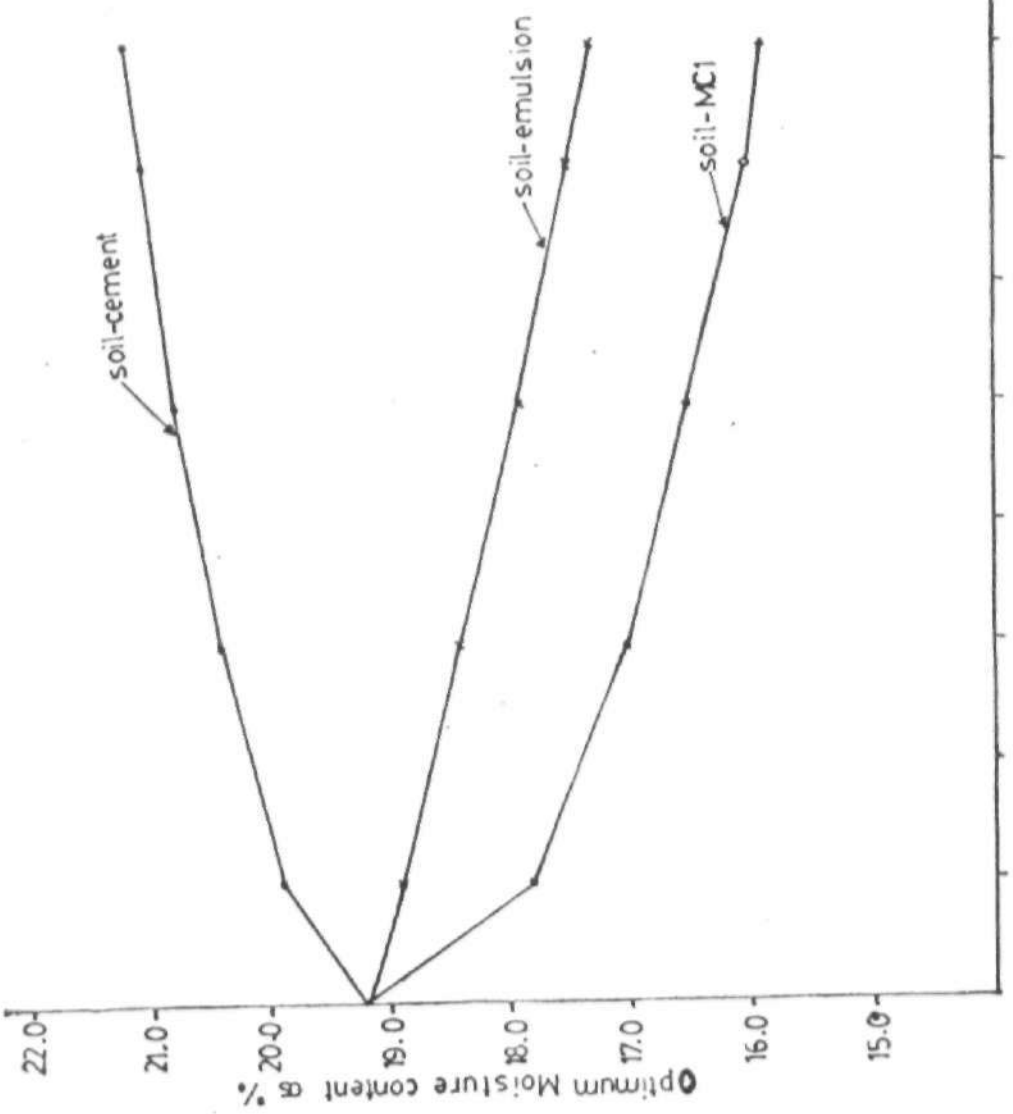


FIG. 3.3: Variation of optimum moisture content with stabilizer content.

carefully controlled density and moisture conditions and is used with empirical curves for designing flexible pavements (O'Flaherty, 1974). It is expressed as a percentage of the unit load required to force a piston into the soil divided by the unit load required to force the same piston to the depth into a standard sample of crushed stone (Head, 1980). The test is usually carried out using either soaked or unsoaked specimen.

Soaking of specimen is sometimes needed so as to subject the soil (or stabilized soil) to the worst conditions which exist prior to construction. So, soaking is done to see whether the soil will resist water seepage and uphold its strength when subjected to severe water conditions that may last for hours or days.

For details of test procedure, see BS 1924: Part 1: 1990. The results of the CBR tests are presented in tables 3.8, 3.9 and 3.10 (see Appendix C for the laboratory data). The plots of the CBR data values versus the stabilizer content(%) are shown in figures 3.4, 3.5 and 3.6 for both oven-dried and soaked specimen.

Table 3.8: Results of the average CBR (%) (oven-dried for 1 day)

Stabilizer content (%)	Emulsion	MC1	Cement
0	13	13	13
1	16	15	19
3	26	28	35
5	24	27	38
7	19	18	40
8	13	12	44

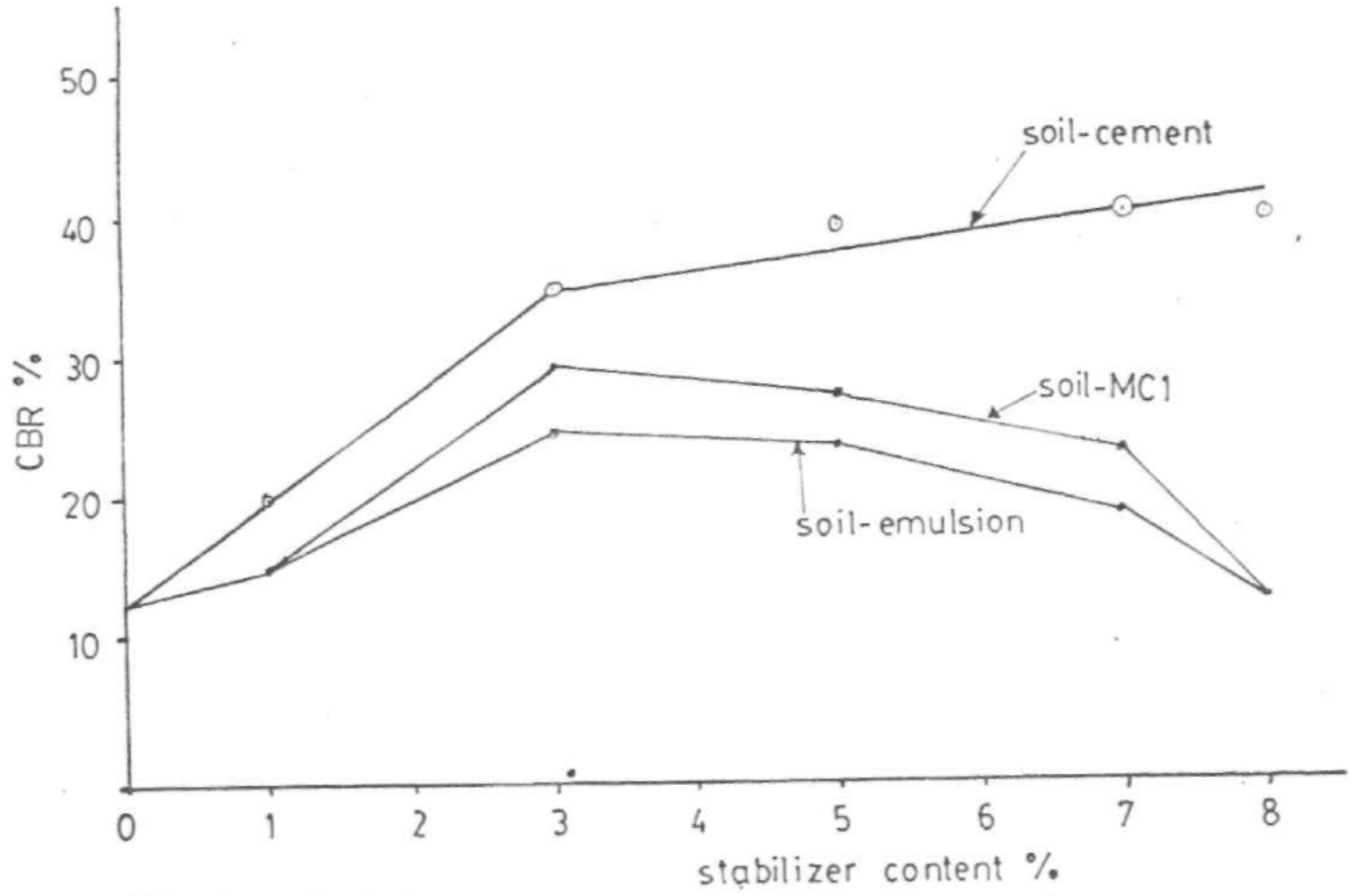


FIG. 3.4: Variation of CBR with stabilizer content(specimen oven-dried for 1 day).

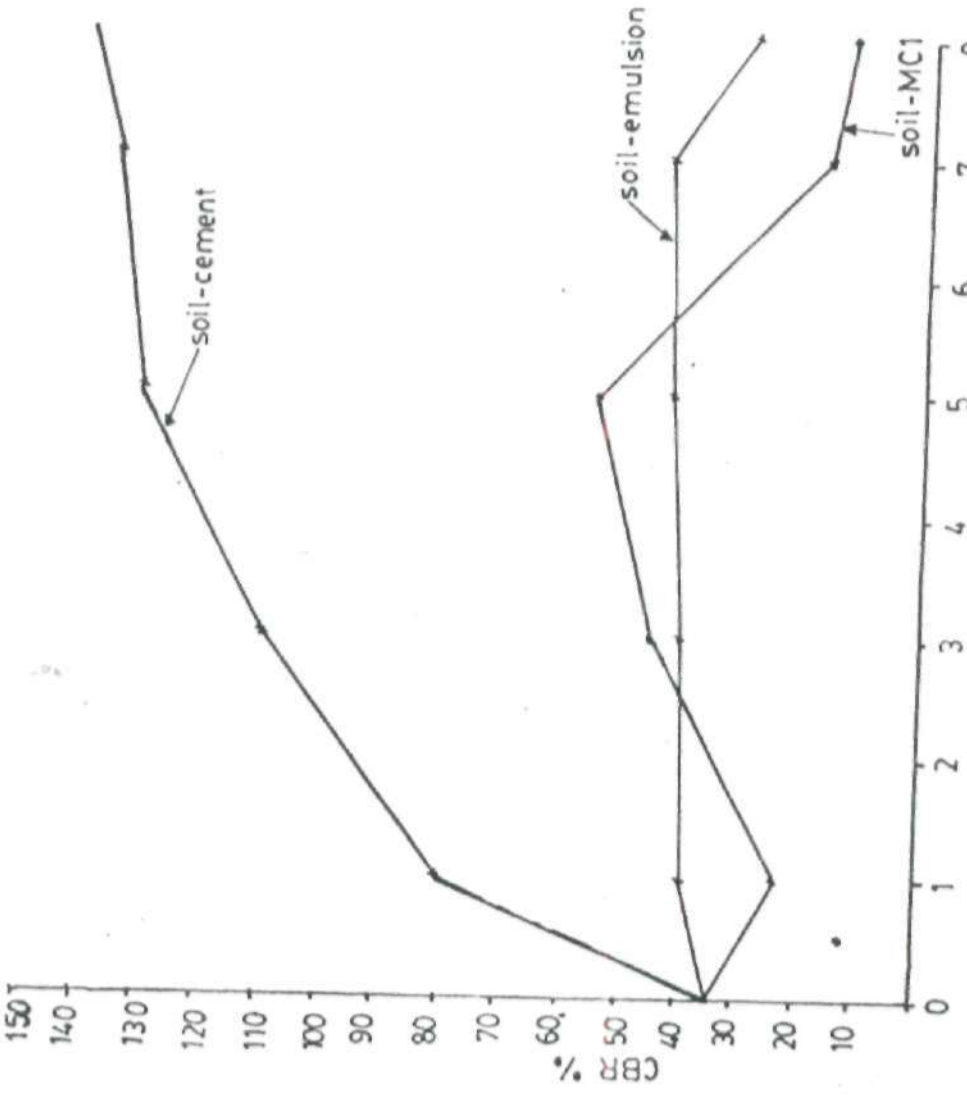


FIG. 3.5: Variation of CBR with stabilizer content(specimen oven-dried for 7 days).

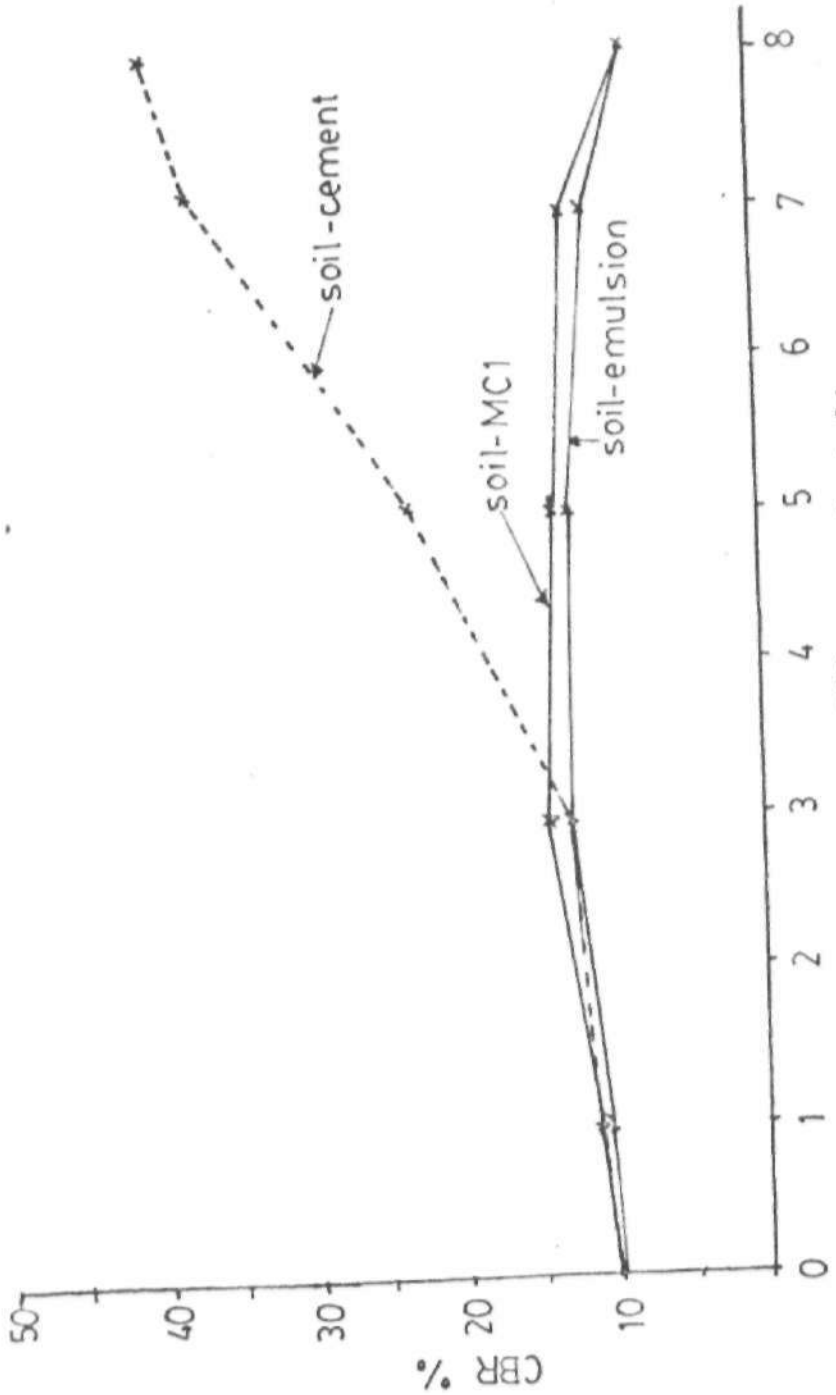


FIG. 3.6: Variation of CBR with stabilizer content
(specimen soaked for 7 days).

Table 3.9: Results of the average CBR (%) Oven-dried for 7 days)

Stabilizer content (%)	Emulsion	MC1	Cement
0	34	34	34
1	39	23	80
3	40	45	110
5	42	55	130
7	43	16	135
8	29	13	140

Table 3.10: Results of the average CBR (%)(Soaked for 7 days)

Stabilizer content (%)	Emulsion	MC1	Cement
0	10	10	10
1	11	12	12
3	13	14	13
5	12	13	23
7	10	12	38
8	9	9	41

Table 3.11: Tests on the soil and some recommended values from Specifications:

Test carried out	Results	Sub-base specifications	Base Specification
Specific gravity	2.44	≤ 3	≤ 3
Liquid limit (%)	42.0	≤ 30	≤ 30
Plasticity Index (%)	17.1	≤ 12	≤ 12
CBR (unsoaked) (%)	34.0	≥ 30	≥ 80
CBR (soaked) (%)	10.0	25-35	≥ 75

3.8: The unfoncined compression test:

The unconfined compression test was carried out on the soil samples, and for details of the procedures, see BS 1924: Part 1: 1990.

The change in specimen length, L, can be expressed as:

$$L = a-b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.16$$

where

a= movement of the top of the proving ring,

= K(in mm per revolution) X no of revolutions.

K is a property of the gear box, and is found by caliberation (Bishop and Henkel, 1982).

b= compression of the proving ring

= proving ring dial reading (mm)- initial proving ring dial reading.

The axial strain, E, can be expressed as:

$$E = \frac{l}{l_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3.17$$

where l_o = initial specimen length

The average cross-sectional area, A, at any point in time is:

$$A = \frac{A_o}{1-E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.18$$

Where

A_o = the initial cross-sectional area of specimen with a diameter of 38.1mm.

$$A_o = \frac{\pi (38.1)^2}{4(10)^6} = 1.14 \times 10^{-3} m^2$$

The applied stress= $\frac{P}{A}$

where

P = applied force

= (proving ring reading-initial proving ring reading) x ring calibration factor.

The lab results of the unconfined compression tests are presented in Appendix D. A summary of the results is given in tables 3.12 and 3.13 for specimen wax-cured for 7 days and also soaked for 7 days.

Table 3.12: Results of unconfined compression test (specimen wax-cured for 7 days)

Stabilizer content %	Emulsion (MS2) Compressive strength (KN/m^2)	MC1 Compressive Strength (KN/m^2)	Cement compressive Strength (KN/m^2)
0	330	330	330
1	420	430	690
3	450	450	900
5	500	510	1690
7	390	380	2080
8	350	340	2380

Table 3. 13: Results of unconfined compression test (specimen soaked for 7 days)

Stabilizer Content %	Emulsion (MS2) Compressive Strength (KN/m^2)	MC1 Compressive Strength (KN/m^2)	Cement Compressive Strength (KN/m^2)
0	60	60	60
1	70	80	340
3	140	150	500
5	210	220	590
7	240	240	690
8	260	260	770

The plots of compressive strength versus stabilizer content are shown in figures 3.7 and 3.8.

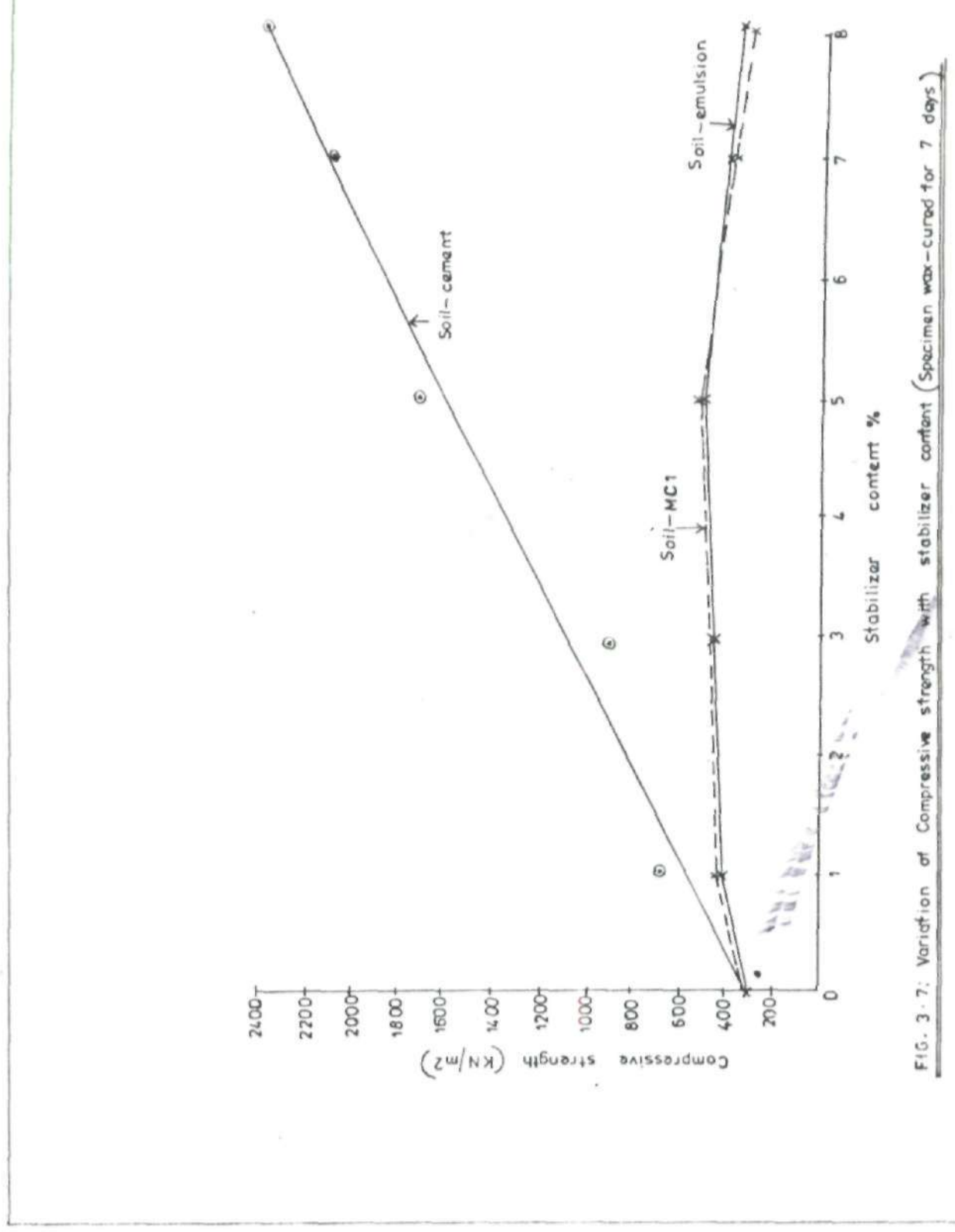


FIG. 3·7: Variation of Compressive strength with stabilizer content (Specimen wax-cured for 7 days)

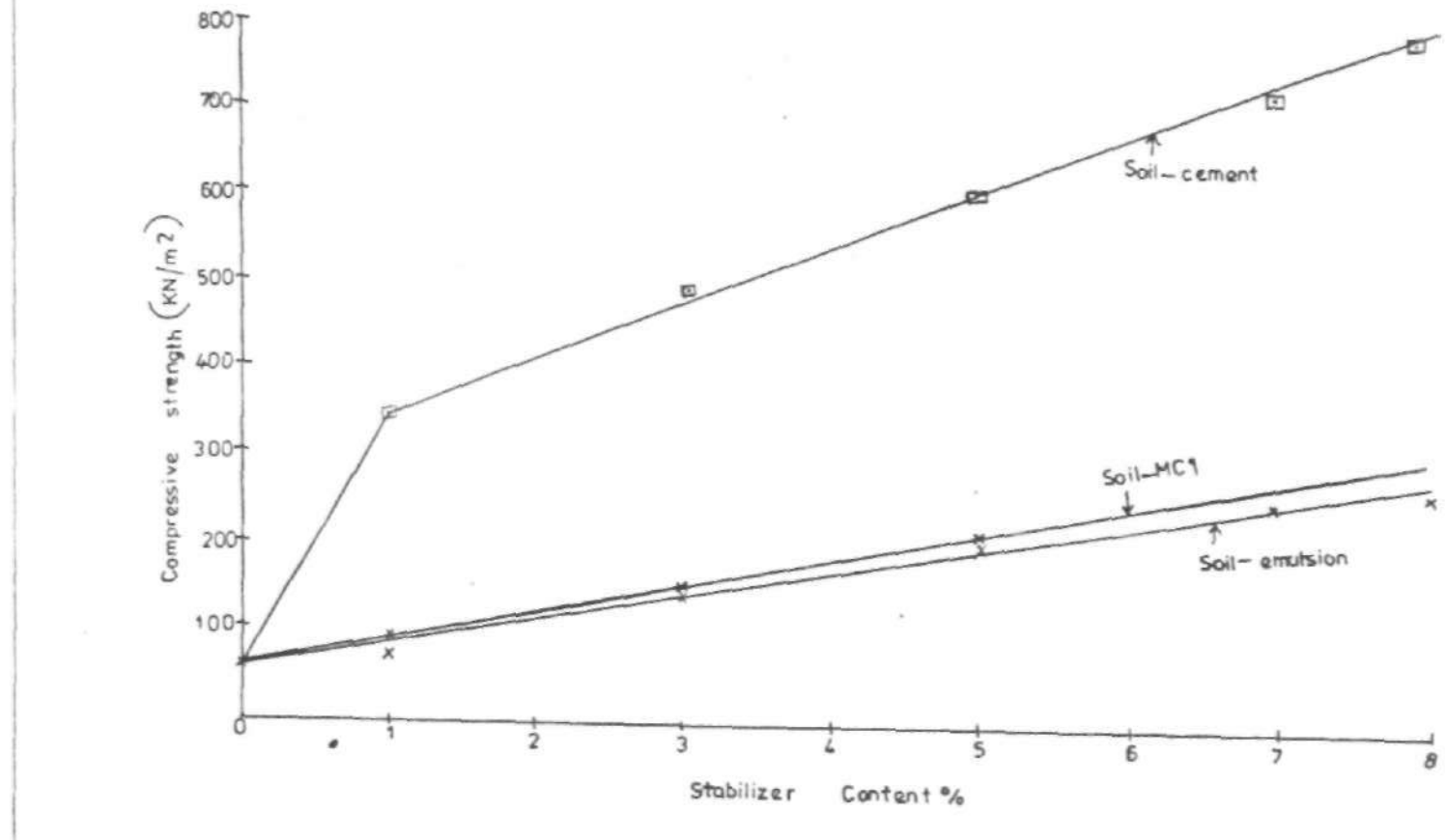


FIG 3·8: Variation of compressive strength with stabilizer content (Specimen soaked for 7 days)

3.9 Capillary water absorbtion test:

This test was carried out to determine the efficiency of the stabilizers as waterproofing agents. It was performed in accordance with BS 1924: Part 1 1990. The summary of the results is presented in table 3.14 and 3.15, while the graphical plot is shown in figure 3.9. For the lab data, see Appendix E.

Table 3.14: Results of water absorption test (using emulsion stabilizer)

Emulsion content (%)	Water absorbed as percentage of oven-dried weight		
	1 day	3 days	7 days
0	3.1	3.4	3.6
1	2.7	2.9	3.2
3	1.7	2.2	2.4
5	1.2	1.4	1.8
7	0.5	0.7	1.2
8	0.3	0.5	0.7

Table 3.15: Results of water absorption tests (using MC1 stabilizer)

MC1 content (%)	Water absorbed as percentage of oven-dried weight.		
	1 day	3 days	7 days
0	3.2	3.4	3.6
1	2.6	3.0	3.2
3	1.6	2.2	2.4
5	1.0	1.4	1.7
7	0.4	0.6	1.0
8	0.2	0.4	0.6

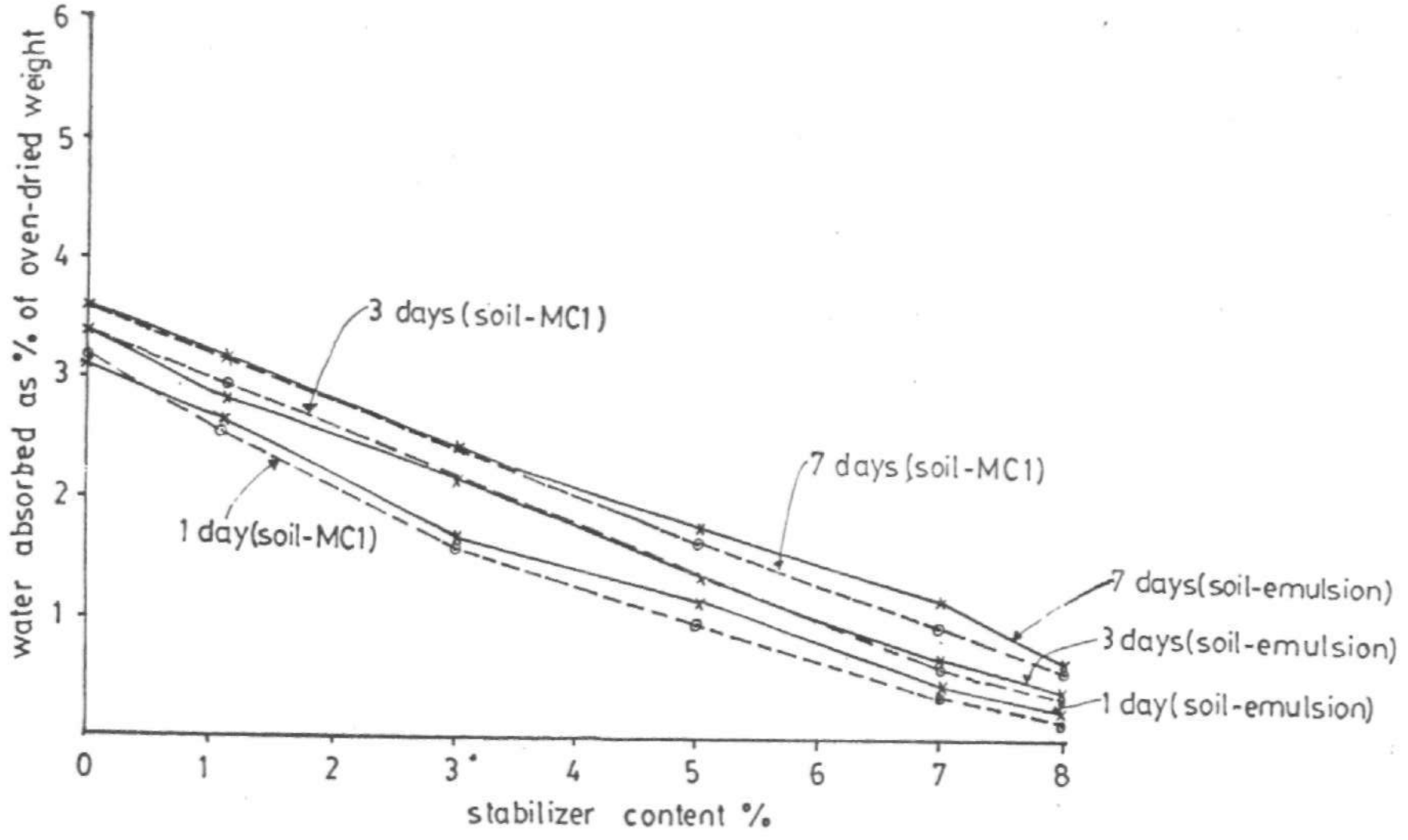


FIG. 3.9: Variation of water absorbed with stabilizer content.

CHAPTER 4

ANALYSIS OF RESULTS

4.0 Summary of the preliminary results and comparison with Recommended values:

The summary of the results compared with some recommended values for subgrade, sub-base and base specifications is as shown in table 3.11. From the table, the unstabilized soil is generally not suitable for sub-base or base material on account of the CBR result for the soaked specimen.

The results of the stabilized samples are discussed below:

4.1 Compaction tests:

The compaction characteristics of the stabilized samples in terms of the variation of the maximum dry density versus the stabilizer content, and also the optimum moisture content are presented in tables 3.4 - 3.6 and figures 3.2 and 3.3. The effects of the three stabilizers used are discussed in the sections that follow:

4.1.1 Maximum Dry Density:

From figure 3.2 the cement stabilized soil gave the highest maximum dry density as from 5% cement content while at a lower content the other two stabilizers (MS2 and MC1) had higher maximum dry density. Generally the maximum dry density is observed to decrease with increase in the stabilizer content up to 8%. This compaction characteristics occur owing to the effect of both the grain size distribution and the specific gravities of the soil and stabilizer. Initially the stabilizers coat the soil to form large aggregates which then occupy larger spaces. This consequently leads to decrease in the maximum dry density as the content of the stabilizer is increased. However, at high content, only cement, owing to its high specific gravity, is able to compensate for the larger spaces to give high values of maximum dry density.

4.1.2 Optimum Moisture Content:

The variation of the optimum moisture content with the stabilizer content (figure 3.3) shows that the optimum moisture content increases with increase in cement content while it decreases with increase in emulsion or MC1. These results are so because of the hydration of cement which normally requires more water with increase in cement while less water is required in the case of the other two stabilizers as bitumen does not react with water.

4.2 California Bearing Ratio (CBR) Tests:

The results of the CBR tests are presented in tables 3.7 - 3.9, and the graphical plots in figures 3.4 - 3.6. In both cases of specimen oven-dried for seven days and those soaked for seven days, the cement stabilizer generally gave the highest values of CBR while the emulsion and MC1 stabilizers had similar characteristics in the soaked condition. These results show that about 7% of emulsion is required for a maximum CBR of 43% in oven-dried condition while 3% of emulsion is needed for a maximum CBR of 13% in soaked condition. Also, 5% of MC1 is required for a maximum CBR of 55% in oven-dried condition while 3% of MC1 is needed for a maximum CBR of 14% in soaked condition. As the CBR results for oven-dried condition are quite higher than those of the soaked condition, the importance of humidity of the environment in bitumen stabilization is clearly shown. Furthermore, if a CBR of about 80% (standard Proctor compaction) is required for a base material, only the cement-stabilized soil will be accepted.

4.3 Unconfined Compression tests:

From figure 3.7, showing the variation of compressive strength with stabilizer content for the wax-cured condition, samples stabilized with emulsion and MC1 have similar characteristics. A content of 5% emulsion or MC1 is required for a maximum compressive strength of about 500 KN/m^2 in this case. The soil-cement gave a linear relationship with much higher values of compressive strength than the other two stabilizers.

If the minimum permissible compressive strength for base materials is taken as 1725 KN/m^2 (for standard Proctor compaction), it is obvious that only the cement-stabilized soil will be acceptable in this case. The compressive strength values in the soaked condition (fig 3.8) are quite lower than in the wax-cured condition. The soil-emulsion and soil-MC1 in soaked condition gave linear characteristics. Furthermore, at a high stabilizer content, the compressive strength values for the wax-cured condition and soaked condition tend to the same result. This can be attributed to the waterproofing effect of the bitumen which prevents some water from penetrating to produce a further decrease in the soil strength.

4.4 Water Absorption tests:

The results of the water absorption test are presented in table 3.12 and 3.15 while the graphical plot is shown in figure 3.9. For both the emulsion and MC1-stabilized samples, the percentage of water absorbed decreased with increase in the stabilizer content. Also the longer the duration of the absorption test the higher the percentage of water absorbed. These results show the waterproofing effect of both emulsion and MC1 as soil stabilizers. From figure 3.9, MC1 is better as a waterproffing agent than emulsion (MS2).

4.6 Soil Stabilization Mechanisms:

Both cement stabilization and bituminous stabilization mechanisms utilized in this work are discussed as follows:

4.6.1 Cement Stabilization mechanism:

When water is added to cement, hydration results. This produces cementitious compounds such as calcium silicate hydrates, calcium aluminate hydrates and hydrated lime(Moh, 1962). The first two products constitute the major cementitious materials while lime is deposited as a crystal. The increase in strength of the stabilized samples is as a result of the development of cementitious linkage between these hydration products and soil particles. The lime produced during the hydration of cement may react with any pozzolanic material such as clay in the soil to produce a secondary cementitious material which also contributes to inter-particles bonding.

4.6.2. Bituminous Stabilization Mechanism:

Bitumen stabilizes soils in two ways. The first one is a cementation process brought about by small percentages of bitumen aimed at increasing the cohesion in the soil without preventing mineral-to-mineral contacts being formed between grains (Rahaman, 1968). When the bitumen content exceeds the optimum, the bitumen then acts as a lubricant separating the soil particles; this is the second mechanism having mainly a waterproofing property which is most effective in cohesive soils. Waterproofing preserves the natural stability which these soils have in a dry and well-compacted condition. In A-7-6 laterite, bitumen tends to coat the soil in small aggregates thereby plugging voids and acting as a water-proofer (Rahaman, 1968).

CHAPTER 5

5.0 DEDUCTION, CONCLUSION AND RECOMMENDATION

From the analysis of the results, some brief deductions are made in this capter followed by the conclusion and recommendation.

5.1 DEDUCTION:

The following deductions are made:

(a) From figure 3.4, the CBR for samples oven-dried for 1 day shows that 3% content of emulsion and also MC1 are needed to give a maximum CBR of 20% and 24% respectively. However, the CBR for the cement-stabilized specimen increased with an increase in the content of the cement.

(b) For samples oven-dried for 7 days (fig.3.5), 7% emulsion is needed to give a maximum CBR value of 43%; and 5% MC1 is required to give a maximum value of 45%. Again, the CBR value for the cement increases with an increase in cement content. Comparing these values with the recommendad CBR (unsoaked) values in table 3.11 (30% for sub-base; 80% for base), the soils stabilized with emulsion and MC1 meet the specifications for the sub-base while the cement-stabilized soil meets the specifications for the sub-base and base course.

(c) For samples soaked for 7 days (fig.3.6), 3% of both emulsion and MC1 are required to give a maximum CBR value of 13% and 14% respectively. These values are lower than those recommended for soaked conditions (25 - 35% for sub-base; 75% for base). The CBR value for the cement increases with an increase in the cement content.

(d) For soils wax-cured for 7 ys (fig.3.7), 5% emulsion or MC1 is required to give a maximum compressive strength of about 500KN/m^2. The compressive strength for the cement-stabilized soil increases with increase in the cement content.

(e) In the case of soils soaked for 7 days, both emulsion-stabilized and MC1-stabilized soils show that the compressive strength increases with increase in the stabilizer content. However, they do not meet the specifications for sub-base or base.

(f) The results of the water absorption test (fig.3.9) show that about 3% emulsion and also MC1 is effective to be used as a waterproofing agent for the soil.

5.2 CONCLUSION:

Based on the results presented, the stabilized soil using emulsion (MS2) or cutback bitumen (MC1) meet the specifications for mainly sub-base in dry conditions only. Therefore, emulsion stabilization may be employed to improve A-7-6 laterite as only fill or sub-base material. However, on very lightly trafficked roads with restrictions on the wheel loads, bituminous stabilization may be used to upgrade A-7-6 laterite for base course. In the case of cement stabilization, it can easily be used to upgrade the soil to meet the base course specifications provided an adequate percentage of cement is utilized.

5.3 RECOMMENDATION:

It is recommended that between 3 - 5% of emulsion (MS2) or cutback bitumen (MC1) should be used to stabilize A-7-6 laterite for fill or sub-base material. A maximum content of 3% is recommended for cement. A high percentage of cement should be avoided because it can lead to cracking and deformation of the stabilized soil.

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APPENDIX A

LABORATORY RESULTS OF THE PRELIMINARY TESTS ON THE SOIL

DATA SHEET 1

DETERMINATION OF THE NATURAL MOISTURE CONTENT OF THE SOIL

Container No.	133	180	193	Average
Wt of container + wet soil, M_1 (gm)	254.80	261.20	270.70	
" " " + dry " M_2 (gm)	41.00	41.40	42.20	
" " " M_3 (gm)	8.60	8.60	8.60	
Dry wt of soil $M_2 - M_3$ (gm)	32.40	32.80	33.60	
Wt of moisture, $M_1 - M_2$ (gm)	213.80	219.80	228.50	
Moisture content, $m = \frac{M_1-M_2}{M_2-M_3}$ %	6.60	6.70	6.80	6.70

DATA SHEET 2

DETERMINATION OF THE SPECIFIC GRAVITY OF THE SOIL

TEMP = 23°C

Bottle No	1	2	3
Wt of bottle + water (full), W_4 (gm)	683.9	346.5	352.4
" " " " soil + water, W_3 (gm)	717.1	369.7	385.7
" " " " soil, W_2 (gm)	241.3	137.0	160.0
" " " W_1 (gm)	184.2	98.1	103.8
" " water, $(W_4 - W_1)$ gm	499.7	248.4	248.6
" " " added to soil, $(W_3 - W_2)$ gm	475.8	232.7	225.7
" " " displaced by soil, $W = (W_4 - W_1) - (W_3 - W_2)$ gm	23.9	15.7	22.9
Specific gravity of the soil, $(W_2 - W_1)/W$	2.39	2.48	2.45

Average specific gravity = 2.44

DATA SHEET 3

SIEVE ANALYSIS OF THE SOIL

Gross mass of Soil, M_o = 200.00 gm

Mass Passing 2 mm sieve, M_1 = 195.00 gm

Mass retained on 2mm sieve = 5.00 gm

Mass used for sedimentation analysis, M_2 = 171.50gm

Sieve No	Opening size (mm)	Mass of soil retained (gm)	Fraction retained	Cumulative fraction retained	Fraction Finer
7	2.411	0.00	0.00	0.00	1.00
14	1.204	4.00	0.08	0.08	0.92
25	0.700	4.00	0.08	0.16	0.84
36	0.422	3.00	0.06	0.22	0.78
52	0.295	2.50	0.05	0.27	0.73
72	0.210	2.50	0.05	0.32	0.68
100	0.149	2.50	0.05	0.37	0.63
200	0.075	5.00	0.10	0.47	0.53

DATA SHEET 4

HYDROMETER (SEDIMENTATION) ANALYSIS

Gross mass of soil, M_o = 200.00 gm

mass passing 2mm sieve, M_1 = 195.00gm

mass used in sedimentation analysis = 171.50gm

Elapsed Time	Hydrometer Reading	Temp (°C)	Effective depth L (cm)	Fully correct Reading (Rc)	Particle diameter (mm)	Fraction finer
20 secs	28.8	23	9.4	27.1	0.075	0.55
40 "	26.8	"	10.2	25.1	0.055	0.51
1 min	25.4	"	10.8	23.7	0.046	0.48
1 min 20 s.	23.9	"	11.3	22.2	0.041	0.45
1 min 40 "	22.4	"	11.9	20.7	0.037	0.42
2 min	20.9	"	12.5	19.2	0.035	0.39
2 min 30 s	19.0	"	13.3	17.3	0.032	0.35
5 mins	17.5	"	13.8	15.8	0.023	0.32
10 mins	16.5	"	14.2	14.8	0.018	0.30
20 mins	15.5	"	14.6	13.8	0.012	0.28
40 mins	15.0	"	14.8	13.3	0.009	0.27
80 mins	15.0	"	14.8	13.3	0.006	0.27
160 mins	14.5	"	15.0	12.8	0.004	0.26
320 mins	14.5	"	15.0	12.8	0.003	0.26
640 mins	14.5	"	15.0	12.8	0.002	0.26

DATA SHEET 5

COMBINED PARTICLE SIZE ANALYSIS

Gross mass of soil, M_o = 200.00gm

Mass passing 2mm sieve, M_1 = 195.00gm

Mass used in sedimentation analysis, M_2 = 171.50gm

Particle Size (mm)	Percentage smaller			Total (%)
	Sieve Analysis		Hydrometer Analysis	
	Out of M_o-M_1	Out of M_1	Out of M_1	
2.00	100	100		100
2.00		100		100
1.20		92		92
0.70		84		84
0.42		78		78
0.30		73		73
0.21		68		68
0.15		63		63
0.075			55	55
0.075		53		53
0.055			51	51
0.046			48	48
0.041			45	45
0.037			42	42
0.035			39	39
0.032			35	35
0.023			32	32
0.018			30	30
0.012			28	28
0.009			27	27
0.006			27	27
0.004			26	26
0.003			26	26
0.002			26	26

DATA SHEET 6

DETERMINATION OF THE ATTERBERG LIMITS

PLASTIC LIMIT TEST

Determination No	1	2	3
Container No	22	58	0
Wt of container + wet soil, W_3 (gm)	16.0	15.4	14.4
" " " " dry soil, W_2 (gm)	14.6	14.1	13.3
" " W_1 (gm)	8.9	8.9	8.9
" " water, W_w, (W_3-W_2) gm	1.4	1.3	1.1
" " dry soil, W_s, (W_2-W_1) gm	5.7	5.2	4.4
Water content, w%, (W_w/W_s) x 100	24.6	25.0	25.0

Average Plastic limit = 24.9%

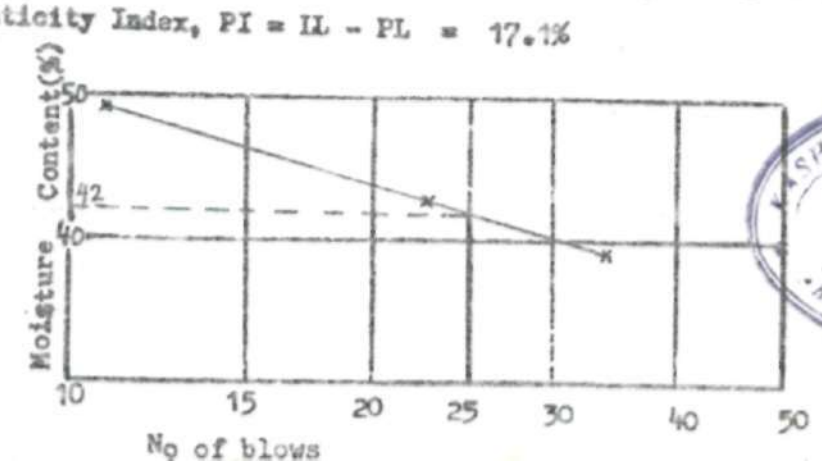
LIQUID LIMIT TEST

Determination No	1	2	3
No of Blows	11	23	34
Container No	193	179	150
Wt of container + wet soil, W_3 (gm)	28.2	21.0	21.1
" " " + dry soil, W_2 (gm)	21.7	17.2	17.6
" " " W_1 (gm)	8.5	8.4	8.7
" " water, W_w, (W_3-W_2) gm	6.5	3.8	3.5
" " dry soil, W_s, (W_2-W_1) gm	13.2	8.8	8.9
Water content, w%, (Ww/Ws) x 100	49.2	43.2	39.3

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Liquid limit = 42.0% (moisture content corresponding to 25 blows)

Plasticity Index, PI = LL - PL = 17.1%



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DATA SHEET 7

DETERMINATION OF THE SPECIFIC GRAVITY OF MC1

Wt of bottle + water (full), W_4 = 1595.5gm

" " " " bitumen + water, W_3 = 1596.7gm

" " " " bitumen, W_2 = 681.7gm

" " " W_1 = 557.0gm

" " water, $(W_4 - W_1)$ = 1038.5gm

" " " added to bitumen, $(W_3 - W_2)$ = 915.0gm

" " bitumen, $(W_2 - W_1)$ = 124.7gm

" " water displaced by bitumen, $(W_4 - W_1) - (W_3 - W_2)$ = 123.5gm = W

Specific gravity of bitumen, $(W_2 - W_1)/W$ = 1.01

APPENDIX B

TYPICAL LABORATORY RESULTS OF THE COMPACTION TESTS

Data Sheet

DETERMINATION OF THE MOISTURE/DENSITY RELATION OF SOIL
USING STANDARD* ~~AASHTO~~ * COMPACTION

Job Emulsion Stabilization (1%) Sample A-7 Laterite Operator.....

Site.....Depth.....1m.....Date.....

Amount retained on ¾" B.S. Sieve.....g..... Total weight of sample.....g.....

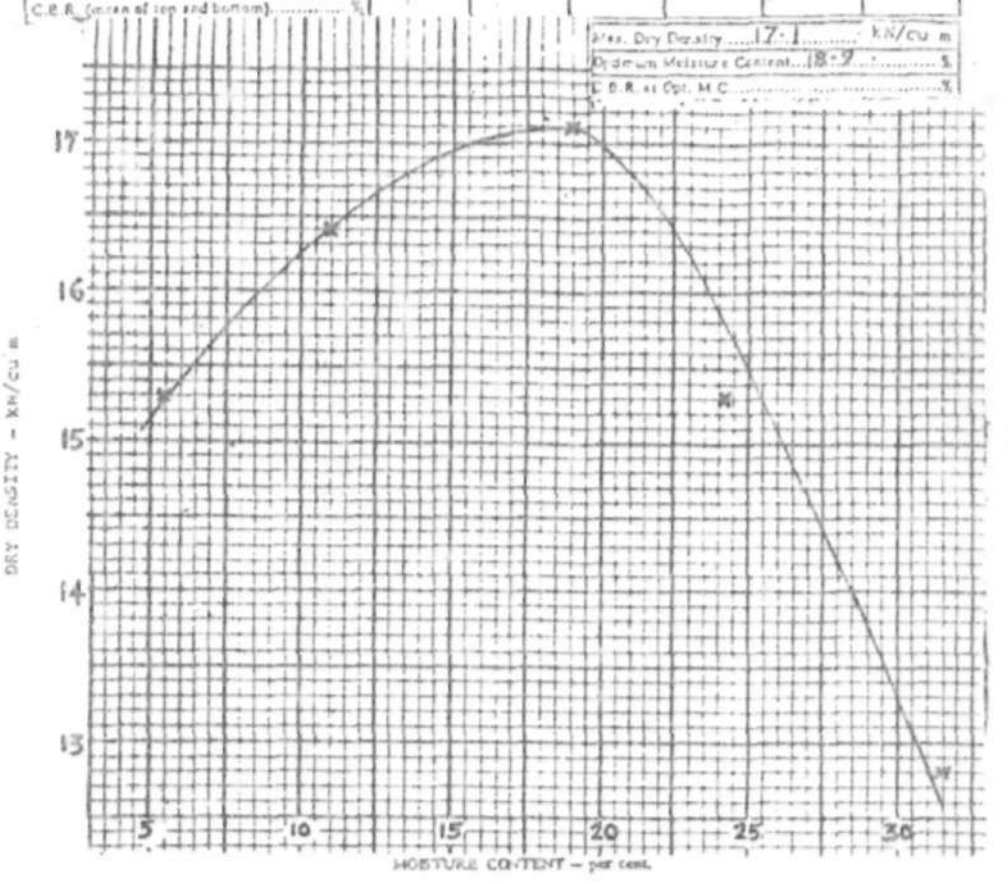
B.S.*/~~G.B.R.~~* Mould .....*Delete the inappropriate word

Wt. of mould and wet soil.....(W2).....g	6648	6843	7062	6953	6705			
Wt. of mould.....(W1).....g	5007	5007	5007	5007	5007			
Wt. of wet soil.....(W2 - W1).....g	1641	1836	2055	1946	1698			
Bulk density $\gamma = \frac{W_2 - W_1}{x}$ .....(kN/cu m)	16·2	18·2	20·3	19·2	16·8			

For B.S. Mould x = 101.1

MOISTURE CONTENT DETERMINATIONS

Container No.	175	128	150	177	133	132	179	140	172	180		
Wt. of wet soil and container.....g	40·6	40·3	38·8	39·7	42·1	35·3	41·6	39·9	41·3	41·1		
Wt. of dry soil and container.....g	39·1	38·2	35·6	36·8	37·2	30·1	35·2	33·8	33·5	33·4		
Wt. of container.....g	8·7	8·6	8·8	8·6	8·5	5·0	8·8	8·7	8·8	8·9		
Wt. of dry soil (Ws).....g	30·4	29·6	26·8	28·2	28·7	25·1	26·4	25·1	24·7	24·5		
Wt. of moisture (Ww).....g	1·5	2·1	3·2	2·9	4·9	5·2	6·4	6·1	7·8	7·7		
Moisture content 100 Ww/Ws.....%	4·9	7·1	11·9	10·3	17·1	20·7	24·2	24·3	31·6	31·4		
Average moisture content (m).....%	6·0		11·1		18·9		24·3		31·5			
Dry density $\gamma_d = 100\gamma/(100+m)$.....kN/cu m	15·3		16·4		17·1		15·3		12·8			
C.B.R. (mean of top and bottom).....%												



Data Sheet

DETERMINATION OF THE MOISTURE/DENSITY RELATION OF SOIL

USING STANDARD* /~~HEAVY~~* COMPACTION

Job MC1 Stabilization (1%) Sample A-7 Laterite Operator .....

Site ..... Depth ..... m Date .....

Amount retained on 3/4" B.S. Sieve ..... g Total weight of sample ..... g

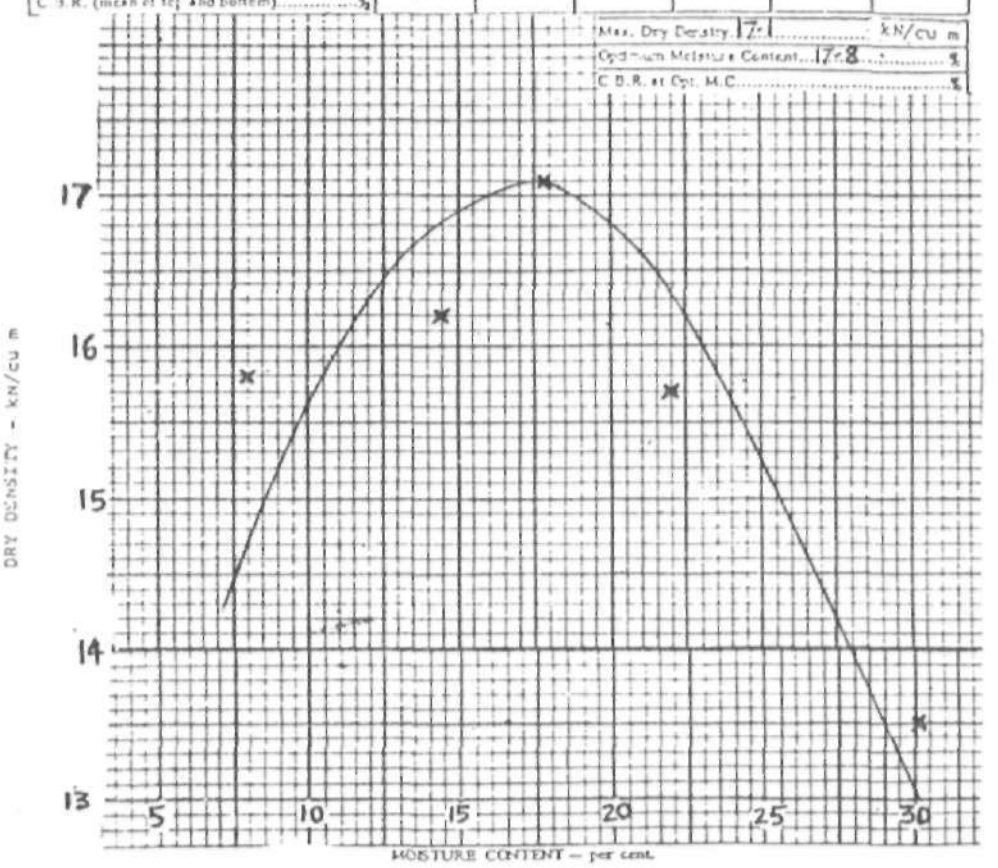
B.S.* /~~C.B.R.~~* Mould ..... *Delete the inappropriate word

Wt. of mould and wet soil ..... (W1) ..... g	6723	6882	7039	6942	6821		
Wt. of mould ..... (W1) ..... g	5007	5007	5007	5007	5007		
Wt. of wet soil ..... (W2 - W1) ..... g	1716	1875	2032	1935	1814		
Bulk density $\gamma = \frac{W_2 - W_1}{X}$ ..... (kN/cu m)	17.0	18.5	20.1	19.1	17.9		

For B.S. Mould X = 101.1

MOISTURE CONTENT DETERMINATIONS

Container No.	158	133	178	175	163	149	193	143	177	145		
Wt. of wet soil and container ..... g	28.7	34.3	23.9	29.1	32.8	33.9	32.5	33.2	34.1	35.0		
Wt. of dry soil and container ..... g	27.2	32.5	21.8	26.8	29.3	30.0	28.2	28.9	28.1	29.0		
Wt. of container ..... g	8.8	8.6	8.8	8.7	8.8	8.5	8.7	9.0	8.5	8.8		
Wt. of dry soil (W_s) ..... g	18.4	23.9	13.0	18.1	20.5	21.2	19.5	19.9	19.6	20.2		
Wt. of moisture (W_w) ..... g	1.5	1.8	2.1	2.3	3.5	3.9	4.3	4.3	6.0	6.0		
Moisture content $100 W_w/W_s$ ..... %	8.2	7.5	16.2	12.7	17.1	18.4	22.1	21.6	30.6	29.7		
Average moisture content (m) ..... %	7.9		14.5		17.8		21.9		30.2			
Dry density $\gamma d = 100\gamma/100+m$ ..... kN/cu m	15.8		16.2		17.1		15.7		13.7			
C.B.R. (mean of top and bottom) ..... %												



Data Sheet

DETERMINATION OF THE MOISTURE/DENSITY RELATION OF SOIL

USING STANDARD* ~~HEAVY*~~ COMPACTION

Job Cement stabilization (1%) Sample A-7 Laterite Operator.....

Site.....Depth.....Jar.....Date.....

Amount retained on 3/4" B.S. Sieve.....g.....Total weight of sample.....g.....

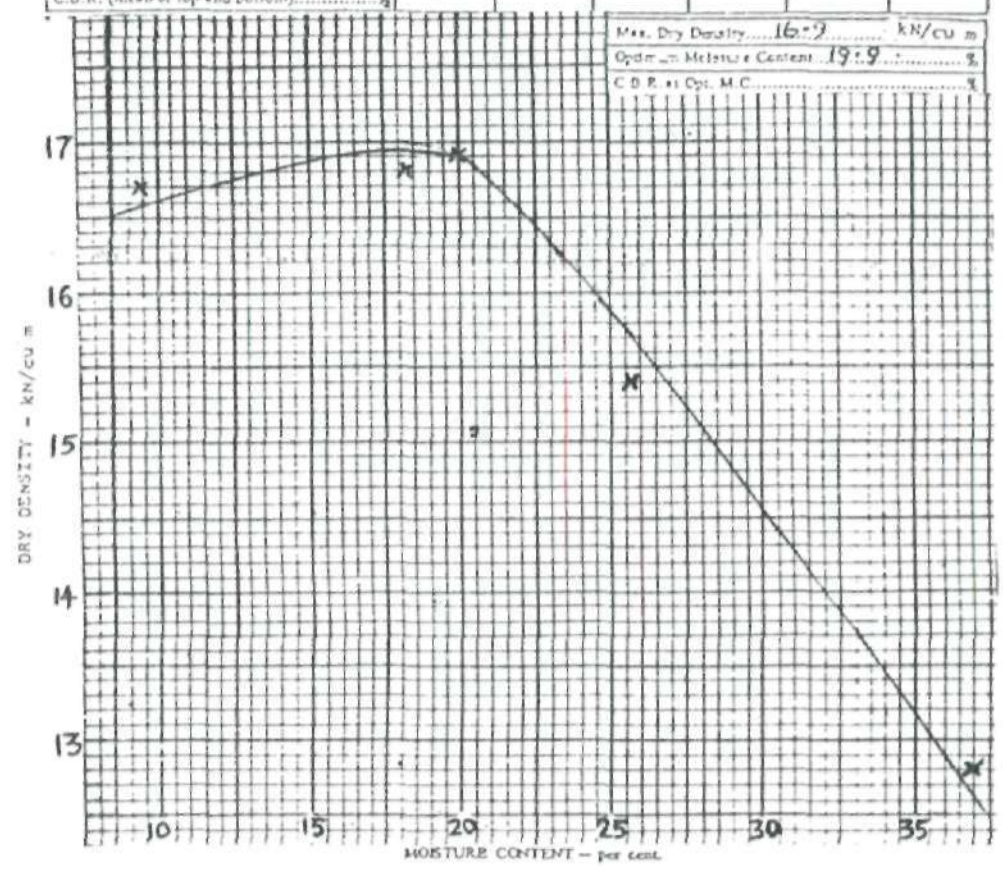
B.S.*/~~C.B.R.*~~ Mould.....*Delete the inapplicable word

Wt. of mould and wet soil.....(W2).....g	6848	7016	7056	6960	6780		
Wt. of mould.....(W1).....g	5007	5007	5007	5007	5007		
Wt. of wet soil.....(W2 - W1).....g	1841	2009	2049	1953	1773		
Bulk density $\gamma = \frac{W_2 - W_1}{X}$ (kN/cu m)	18.2	19.9	20.3	19.3	17.5		

For B.S. Mould x = 101.1

MOISTURE CONTENT DETERMINATIONS

Container No.	173	164	158	133	177	130	58	150	132	193		
Wt. of wet soil and container.....g	51.6	43.0	27.8	35.2	27.8	37.3	39.7	39.5	42.8	56.9		
Wt. of dry soil and container.....g	48.2	39.8	24.8	31.1	24.8	32.3	33.4	35.2	32.6	46.0		
Wt. of container.....g	8.5	8.5	8.5	8.6	8.6	8.8	8.9	8.7	4.9	8.7		
Wt. of dry soil (Ws).....g	39.7	31.3	16.3	22.5	16.2	23.5	24.5	24.5	27.7	37.3		
Wt. of moisture (Ww).....g	3.4	3.2	3.0	4.1	3.0	5.0	6.3	6.3	10.2	10.9		
Moisture content 100 Ww/Ws.....%	8.6	10.2	18.4	18.2	18.5	21.3	25.7	25.7	36.8	37.2		
Average moisture content (m).....%	9.4		18.3		19.9		25.7		37.0			
Dry density $\gamma_d = 100\gamma/100+m$..... kN/cu m	16.7		16.8		16.9		15.4		12.8			
C.B.R. (mean of top and bottom).....%												



APPENDIX C

TYPICAL LABORATORY RESULTS OF THE CBR TESTS

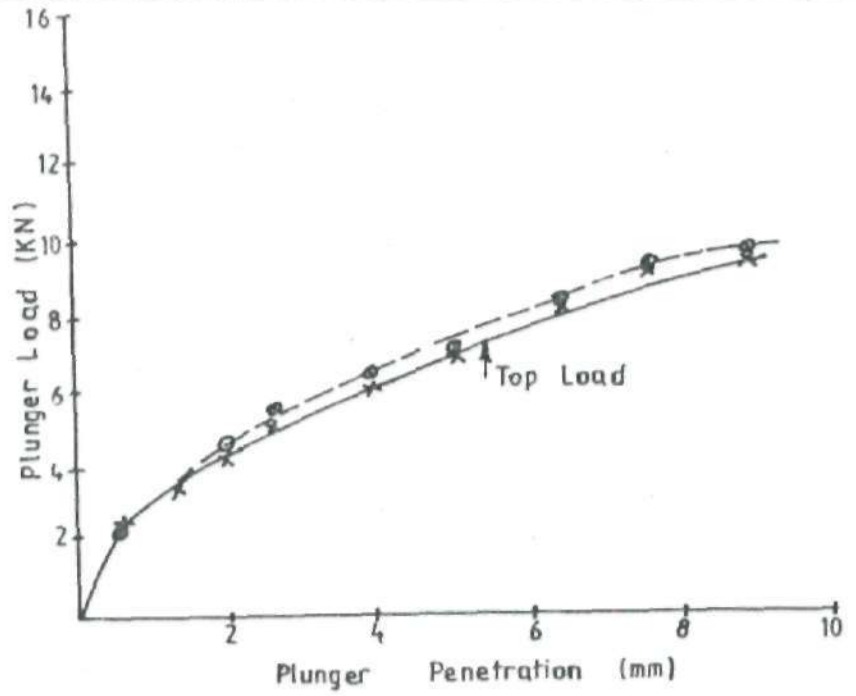
CBR TEST

Sample Oven-dried for 7 days

1% Emulsion

Machine Calibration factor = 0.01286KN/Div

Penetration (mm)	Standard Load	TOP Load indicator dial	TOP Load (KN)	TOP CBR (%)	BOTTOM Load indicator dial	BOTTOM Load (KN)	BOTTOM CBR (%)	A
0		0	0		0	0		
0.6		178	2.3		166	2.1		
1.3		267	3.4		270	3.5		
1.9		330	4.2		338	4.3		
2.5	13.24	391	5.0	38	412	5.3	40	
3.8		483	6.2		490	6.3		
5.0	19.96	559	7.2	36	574	7.4	37	
6.4		631	8.1		641	8.2		
7.6		698	9.0		706	9.1		
8.9		738	9.5		744	9.6		
10.2		800	10.3		810	10.4		



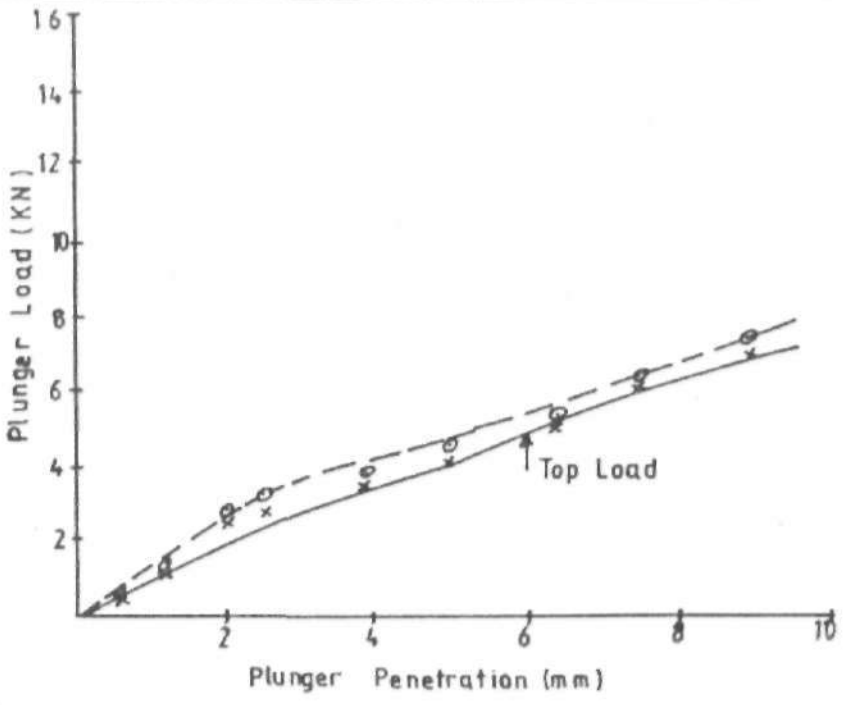
CBR TEST

Sample Over-dried for 7 days

1% MC1

Machine Calibration Factor = 0.01286KN/Div

Penetration (mm)	Standard (KN)	TOP Load indicator dial	TOP Load (KN)	TOP CBR (%)	BOTTOM Load indicator dial	BOTTOM Load (KN)	BOTTOM CBR (%)	Average
0		0	0		0			
0.6		45	0.6		55	0.7		
1.3		107	1.4		115	1.5		
1.9		190	2.4		201	2.6		
2.5	13.24	216	2.8	21	257	3.3	25	
3.8		278	3.6		288	3.7		23
5.0	19.96	310	4.0	20	341	4.4	22	
6.4		397	5.1		401	5.2		
7.6		480	6.2		500	6.4		
8.9		530	6.8		572	7.4		
10.2		600	7.7		620	8.0		
11.4		680	8.7		692	8.9		



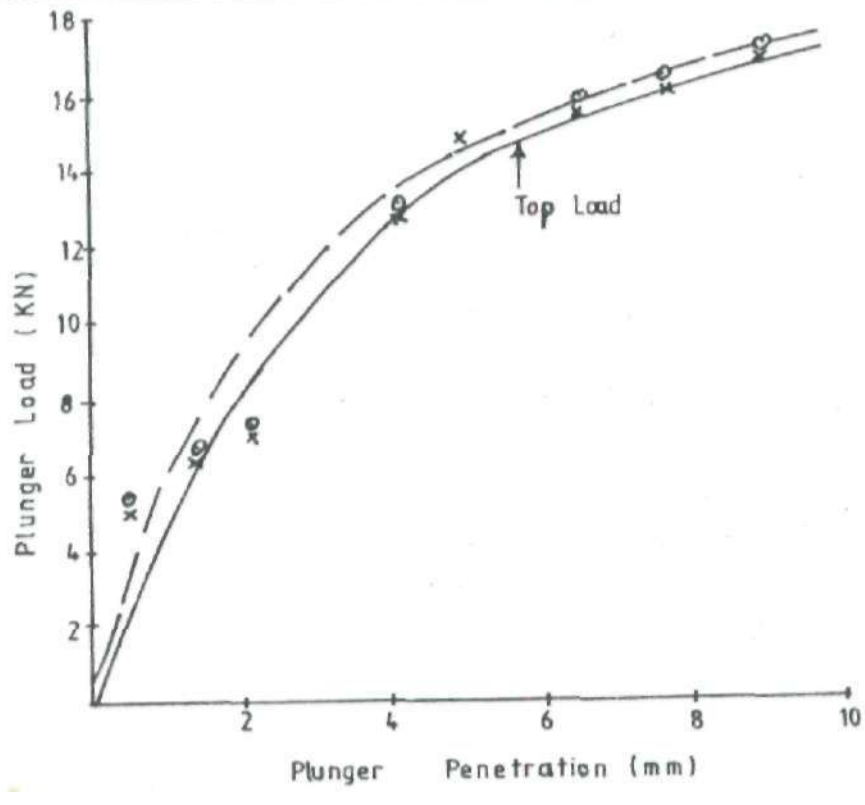
CBR TEST

Sample Oven-dried for 7 days

1% Cement

Machine Calibration Factor = 0.01286KN/Div

Penetration (mm)	Standard Load (KN)	TOP Load indicator Dial	TOP Load (KN)	TOP CBR (%)	BOTTOM Load indicator Dial	BOTTOM Load (KN)	BOTTOM CBR (%)	Average
0		0	0		0	0		
0.6		388	5.0		396	5.1		
1.3		490	6.3		498	6.4		
1.9		610	7.8		615	7.9		
2.5	13.24	813	10.5	79	813	10.5	79	
3.8		991	12.7		999	12.8		80
5.0	19.96	1180	15.2	76	1257	15.2	81	
6.4		1200	15.4		1208	15.5		
7.6		1250	16.1		1259	16.2		
8.9		1300	16.7		1313	16.9		
10.2		1340	17.2		1350	17.4		



APPENDIX D

TYPICAL LABORATORY RESULTS OF THE UNCONFINED COMPRESSION TESTS

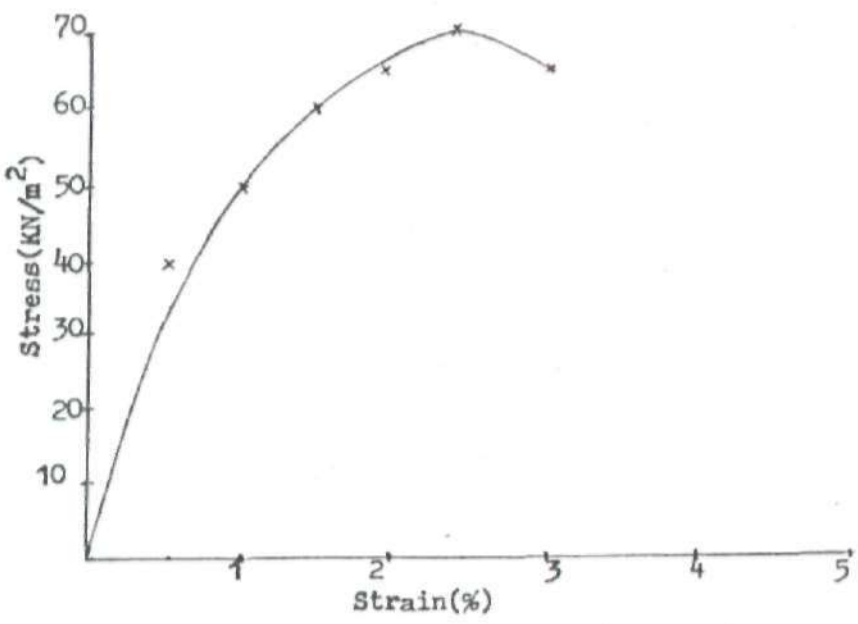
Unconfined Compression Test

SAMPLE SOAKED FOR 7 Days

1% Emulsion

Machine Calibration factor = 5.3×10^{-3} KN/Div

Strain Dial Gauge	Area x 10^{-3} (m^2)	Proving ring Dial	Compressive Strength (KN/m^2)	Strain ($\frac{L}{Lo} \times 100$
0	1.14	0	0	0
0.38	1.15	9	40	0.5
0.76	1.16	11	50	1.0
1.14	1.18	13	60	1.5
1.52	1.19	15	65	2.0
1.91	1.20	16	70	2.5
2.28	1.21	15	65	3.0
2.67	1.23	14	60	3.5
3.05	1.24	13	55	4.5
3.81	1.25			5.0
4.19	1.28			5.5
4.57	1.30			6.0
5.33	1.31			7.0
6.10	1.33			8.0
6.86	1.34			9.0
7.62	1.36			10.0



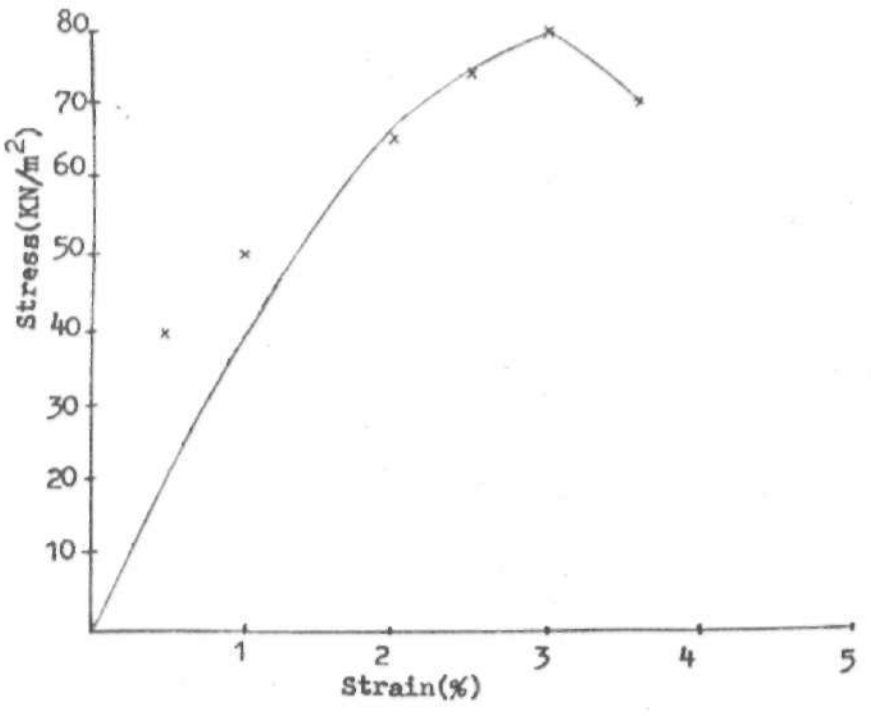
Unconfined Compression Test

Sample Soaked For 7 Days

1% MC1

Machine Calibration Factor = 5.3×10^{-3} KN/Div

Strain Dial Gauge (mm)	Area $\times 10^{-3}$ (m^2)	Proving ring Dial	Compressive Strength (KN/m^2)	Strain ($\frac{L}{Lo} \times 1$
0	1.14	0	0	0
0.38	1.15	9	40	0.5
0.76	1.16	11	50	1.0
1.14	1.18	12	55	1.5
1.52	1.19	15	65	2.0
1.91	1.20	17	75	2.5
2.28	1.21	18	80	3.0
2.67	1.23	16	70	3.5
3.05	1.24	15	65	4.0
3.43	1.25			4.5
3.81	1.27			5.0
4.19	1.28			5.5
4.57	1.30			6.0



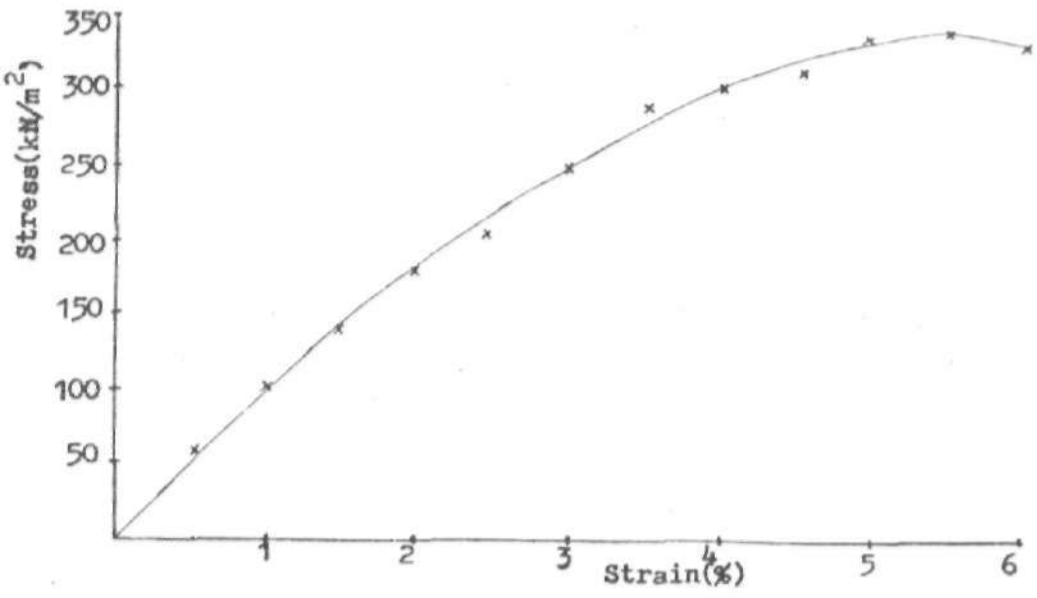
Unconfined Compression Test

Sample Soaked For 7 Days

1% Cement

Machine Calibration Factor = 5.3×10^{-3} KN/Div

Strain Dial Gauge (mm)	Area $\times 10^{-3}$ (m^2)	Proving Ring Dial	Compressive Strength	Strain ($\frac{L}{Lo} \times$
0	1.14	0	0	0
0.38	1.15	13	60	0.5
0.76	1.16	22	100	1.0
1.14	1.18	31	140	1.5
1.52	1.19	40	180	2.0
1.91	1.20	47	210	2.5
2.28	1.21	57	250	3.0
2.67	1.23	65	280	3.5
3.05	1.24	70	300	4.0
3.43	1.25	75	320	4.5
3.81	1.27	79	330	5.0
4.19	1.28	82	340	5.5
4.57	1.30	81	330	6.0
5.33	1.31	79	320	7.0
6.10	1.33			8.0
6.86	1.34			9.0
7.62	1.36			10.0



APPENDIX E

LABORATORY RESULTS OF THE WATER ABSORPTION TESTS

RESULTS OF WATER ABSORPTION TEST

Weight in gms

Emulsion %	Initial wt	1 day	3 day	7 days
0	0-a 2100.0	2166.0	2170.0	2176.5
	0-b 2100.0	2166.5	2171.5	2175.5
1	1-a 2108.0	2155.5	2161.5	2169.0
	1-b 2092.0	2156.0	2160.0	2167.0
3	3-a 2112.0	2135.0	2147.0	2152.0
	3-b 2088.0	2134.5	2146.5	2150.5
5	5-a 2106.0	2125.0	2130.0	2138.5
	5-b 2094.0	2125.0	2130.0	2137.0
7	7-a 2102.0	2110.0	2115.5	2124.0
	7-b 2098.0	2111.5	2114.0	2125.0
8	8-a 2105.0	2106.0	2109.0	2115.5
	8-b 2095.0	2105.5	2110.0	2115.0

Weight in gms

MC1 %	Initial wt		1 day	3 days	7 days
0	0-a	2020.0	2084.5	2087.5	2092.0
	0-b	2020.0	2084.5	2089.5	2094.0
1	1-a	2021.0	2073.5	2081.5	2085.5
	1-b	2019.0	2071.5	2079.5	2083.5
3	3-a	2021.5	2054.0	2065.0	2069.5
	3-b	2019.5	2052.0	2063.0	2067.5
5	5-a	2020.4	2039.0	2048.0	2053.0
	5-b	2019.6	2041.0	2048.0	2055.0
7	7-a	2020.0	2029.5	2034.0	2041.0
	7-b	2020.0	2027.5	2032.0	2039.0
8	8-a	2021.0	2025.0	2029.0	2033.0
	8-b	2019.0	2023.0	2027.0	2031.0