STABILIZATION OF LATERITE WITH LIME AND ASPHALT FOR ROAD WORKS: A COMPARATIVE ANALYSIS.

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CERTIFICATION

This is to certify that this research work was carried out and written by ***AMU, OLUGBENGA OLUDOLAPO.***

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DEDICATION

This thesis is dedicated to my lovely and faithful wife, ***Dr. (Mrs) Eyitope Oluseyi Amu*** and our daughter, ***Inioluwa***, for their prayers, moral support , understanding and encouragement throughout the period of carrying out this research work.

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ABSTRACT

Stabilization, generally is meant to improve the strength of soils and make them perform better as highway materials. The decision as to the proper form of stabilization as well as the selection of the proper chemical stabilizer is often made without the benefit of the necessary field and laboratory testing. In this study, five lateritic soil samples were treated with lime and asphalt to stabilize them and a comparative study was performed in the laboratory to determine the effectiveness of the two stabilizers. Of the five samples, only three which showed distinct features were fully analysed and the stabilizers were added in 2,6 and 10 percentages by weight of the samples. Various laboratory procedures were used to determine the influence of the stabilizers on the samples.

The addition of chemical stabilizers to the samples was generally found to increase their strengths but the rate depends on the type of soil, the type and amount, that is, percentage of stabilizer used. Sample A when treated with lime, increased from a CBR value of 28% to 92% when the percentage stabilizer was increased from 2 to 6 %, but a drop in the CBR to 16% occurred when the percentage stabilizer was further increased to 10%. Similar trends were observed with the other samples indicating that there is an optimum percentage of stabilizer required. With the other samples, however, the strength increase was more significant with sample D, being the most granular. A CBR value of 142% was

obtained when treated with lime at same 6% stabilizer level. Asphalt addition gave only slight improvements in the CBR values of all the samples. The highest strength gain corresponded to 4% addition of asphalt.

For lime, the optimum amount of stabilizer which gave a corresponding optimum strength was at 6 %. Lime showed a significant strength improvement in all the samples than asphalt due to the fact that pozzolanic cementing reactions takes place in lime stabilized soils which is absent in asphalt stabilized soil which only has a water- proofing property. Therefore lime is a better stabilizer than asphalt for the fine-grained soils (Anday, 1963).

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LIST OF ABBREVIATIONS

1.	AASHTO	-	American Association of State Highway Transportation Officials.
2.	Al_2	-	Aluminium
3.	ASTM	-	American Society For Testing and Materials
4.	B.C	-	Before Christ
5.	BS	-	British Standard
6.	Ca^{+2}	-	Calcium
7.	CBR	-	California Bearing Ratio
8.	CL	-	Low Plasticity Clay
9.	c	-	Cohesion
10.	Fe_2	-	Iron
11.	g	-	Gramme
12.	K^+	-	Potassium
13.	Kg	-	Kilogramme
14.	KN	-	Kilonewton
15.	Kpa	-	Kilopascals.
16.	L L	-	Liquid Limit
17.	m	-	Meter
18.	MDD	-	Maximum Dry Density
19.	Mg^{+2}	-	Magnesium

20.	Na^+	-	Sodium
21.	OMC	-	Optimum Moisture Content
22.	pH	-	Level of Acidity or Alkalinity
23.	P I	-	Plasticity Index
24.	P L	-	Plastic Limit
25.	ϕ	-	Internal Friction
26.	SC	-	Sandy Clay Soil
27.	Si	-	Silicon
28.	SM	-	Sand - Silt Soil
29.	TOC	-	Total Organic Content
30.	UCS	-	Unconfined Compressive Strength
31.	USCS	-	Unified Soil Classification System

CHAPTER ONE

INTRODUCTION

1.1 PREAMBLE

In many countries of Africa and Asia, lateritic soils are the traditional materials for Roads and Airfield Pavements construction. Stabilization of lateritic soils with lime and Asphalt is receiving much attention especially by Transportation and Highway Engineers, therefore a comparative study of this type will go a long way in helping Highway Engineers and contractors in the design and construction of road pavements in the future.

Naturally, one is intrigued to know the effects of stabilizers such as Lime and Asphalt on the lateritic soils of Africa and especially that of Nigeria. It is however considered very important to have a global view on the topic, so as to observe the various parameters necessary for the performances of Asphalt and Lime as stabilizers on lateritic Soils.

1.2 Problem Definition.

The concept of soil improvement or modification through stabilization with additives has been around for several thousand years. At least as early as 5000 years ago, soil was stabilized with Lime or pozzolans according to Terrel et.al. (1979). Although this process of improving the engineering properties of soils has been practiced for centuries, soil stabilization has not gained significant acceptance for road and airfield construction in Africa and especially Nigeria. Even in the

U.S, soil stabilization did not gain acceptance until after World War II.

In areas where there has been shortages of conventional aggregates, one gets concerned. In the U.S, highway construction industry consumes over half of the annual production of aggregates (Collins, 1975). Hence, this traditional use of aggregates in pavement construction has resulted in acute shortages in those areas that normally have adequate supplies. This and other factors combine to bring about an escalation of aggregate costs, with a resultant increase in highway construction and maintenance costs. Consequently, there is a great need to find more economical replacements for conventional aggregates. A natural option is that attention is being focused on substitute materials, such as stabilized soils and marginal aggregates that can be upgraded through stabilization.

Another area of concern in terms of highway construction materials has been the temporary shortage of petroleum. The trend now will be toward the use of materials which require less petroleum input in their production, handling and placement. If relatively small quantities of binders such as lime, cement, fly ash and asphalt can be used to improve pavement layers using stabilization technology, total petroleum demands may be reduced as well as costs.

1.3 Study Objectives

The objectives of this study are:

(a) To investigate the factors associated with Lime and Asphalt as binding materials for lateritic soils in road works.

(b) To compare their stabilizing properties on the basis of strength and durability of the lateritic soils.

c. To guide road builders in the choice of stabilizing agents for road works by bringing out the relative advantages of the two stabilizing agents.

As an overview, existing literature suggests that soil stabilization is a desired design alternative. Specifically, soil stabilization may provide the following engineering advantages as compared to conventional unstabilized construction materials :

(a) Function as a working platform

(b) Reduce dusting

(c) Water - proof the soil

(d) Upgrade marginal materials

(e) Improve strength

(f) Improve durability

(g) Control volume change of soils

(h) Improve workability

(i) Reduce pavement thickness requirements.

(j) Conserve aggregate materials

(k) Reduce cost

(l) Conserve energy

(m) Provide a temporary or permanent wearing surface.

1.4 Definitions

(a) Soil - sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks and which may or may not contain organic matter. (National Cooperative Highway Research Program, 1976).

(b) Soil Stabilization - Chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to improve its engineering properties.

(c) Chemical Stabilization - The altering of soil properties by use of certain chemical additives which when mixed into a soil often changes the surface molecular properties of the soil grains and in some cases, cement the grains together resulting in strength increases.

(d) Aggregate - A granular material of mineral composition such as sand, gravel, shell, slag or crushed stone, used with a cementing medium to form mortars, or cement, or alone as in base courses, railroad ballast, etc.

(e) Lime - These are oxides and hydroxides of calcium and magnesium CaO includes all classes of quicklime and hydrated lime, both calcitic (high calcium) and dolomitic. e.g Ca0, $Ca(OH)_2$, etc.

(f) Bitumen - A class of black or dark - colored (solid, semisolid or viscous) cementitious substances, natural or manufactured, composed principally of high molecular weight hydrocarbons, of which asphalts, tars, pitches and asphaltites are typical.

(g) Asphalt - A dark brown to black cementitious material in which the predominating constituents are bitumens which occur in nature or are obtained in petroleum processing.

(h) Cut-back asphalt - Petroleum residiums which have been blended with distillates.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Stabilization of subgrade soils and aggregates by mechanical or chemical means is very common. The decision as to the proper form of stabilization (mechanical or chemical) as well as the selection of the proper chemical stabilizer is often made without the benefit of extensive field and laboratory testing.

Ideally, field and laboratory tests should be performed to determine the types and characteristics of the subgrade soils as well as to define the types and properties of borrow materials available. Laboratory tests should be performed to determine the engineering properties of mechanically stabilized and chemically stabilized soils and borrow materials. Cost and energy associated with providing the pavement structures designed with these materials should be calculated, based on engineering economic principles including first costs and maintenance and rehabilitation costs over a 20 to 30 years period. Except for large projects, this desired engineering approach is rarely undertaken. Therefore, simplified guidelines need to be established to direct the engineer to those stabilization techniques which appear most suitable for the particular situation.

2.2 Laterites

A review of the available data on lateritic soils gave the impression that the red colour seems to have been accepted by most authors as the most important property by which these soils could be identified. It is noted that three major factors influence the engineering properties and field performance of lateritic soils. They are:

(a) Soil forming factors e.g. Parent rock, Climatic - vegetational condition, topography and drainage conditions.

(b) Degree of weathering (degree of laterization) and texture of soils, genetic soil type, the predominant clay mineral type and depth of sample.

(c) Pretest treatment and laboratory test procedures (as well as interpretation of test results).

Chemical analysis do not usually reveal the origin, the nature or even the composition of lateritic soils. For example, physically similar laterites may have different chemical compositions and chemically similar laterites may display different physical properties (Maignien, 1966). Maignien further confirmed under chemical, mineralogical and physico-chemical characteristics that the mineralogical composition is considered to be more important in explaining the physical properties of indurated laterites and lateritic soils. He divided the mineralogical constituents into major elements, which are essential to laterization and minor elements which do not affect the laterization process. The major constituents are oxides and hydroxides of Aluminium and Iron with clay minerals and to a lesser extent, manganese, titanium and silica.

For the physical properties such as texture, Schofield, (1957) found that wet sieving increases the silt and clay fractions from 7 - 20 percent as compared to dry sieving. Mohr and Mazhar, (1969) found that oven - dried lateritic soils give the least amount of clay fraction, as compared to air - dried or as received (natural moisture content) sample. Remillon (1967) reported that though it is an over - simplification to identify all temperate soils with dispersed structure and lateritic soils with concretionary structure, lateritic soils may be on the whole more concretionary than most temperate zone soils.

Nascimento et.al. (1959) have suggested an interesting lithological classification of lateritic soils in the following gradations:

Lateritic clays : less than 0.002mm

Lateritic silts : 0.002 - 0.06mm

Lateritic sands : 0.06 - 2mm

Lateritic gravels : 2 - 60mm

Lateritic stones : greater than 60mm.

The effects of the method of drying on the compaction characteristics have been investigated for soils varying in texture from clays (Newell, 1961 and Tateishi, 1969) to gravels and gravelly soils (Brand and Hongsnoi, 1969). It was shown that oven-drying always gives the highest maximum dry density and lowest optimum moisture content, while soils at natural moisture content give the lowest maximum dry densities and highest optimum moisture contents.

Strength tests on lateritic soils vary mainly with texture. Crushing tests, field penetration tests (Remillon, 1967) aggregate and durability tests (Ackroyd, 1960) are used for indurated laterites and concretionary gravels. The CBR, compression, shear, penetration and weathering tests are used for the gravelly and fined grained soils.

Lateritic soils have been found to have very high specific gravities of between 2.6 and3.4 (De Graft - Johnson and Bhata, 1969) and the most popular test for assessing the strength of lateritic gravels remains the CBR (Remillon, 1967).

The consolidation and permeability characteristic of tropical lateritic soils have been recently summarized (Little, 1967 and De Graft - Johnson and Bhata, 1969). The compressibilities are generally moderate. The permeabilities show wide variations, being generally higher for the residual soils and less for clayey and compacted materials.

Generally speaking, lateritic soils will usually perform satisfactorily in highways. The distribution and engineering properties as well as specifications for highway utilization of lateritic soils from Nigeria have been covered in detail (Clare and Bearen, 1962).

Stabilization of lateritic soils has been successful with both cement and lime, although some soils do not show a marked improvement. Lime has not been used as extensively as cement for stabilizing lateritic soils (Millard, 1962), although Clare and O' Reilly, (1960) considered both equally effective.

Asphalt has been utilized successfully for stabilization according to Nixon and Skipp, (1957) and with only partial success by Winterkorn and Chandrasekharam, (1951).

2.3 Criteria for Lime Stabilization

Experience has shown that lime will react with medium, moderately fine, and fine grained soils to produce decreased plasticity, increased workability, reduced swell, and increased strength (Robnett and Thompson, 1969).

Generally speaking, those soils classified by AASHTO as A - 4, A - 5, A - 6, A - 7, and some of the A - 2 - 7, and A - 2 - 6 soils are most readily susceptible to stabilization with lime. Soils classified according to the Unified system as CH, CL, MH, ML, SC, SM, GC, GM, SW - SC, SP-SC, SM-SC, GW-GC, CP-GC, or GM-GC should be considered as potentially capable of being stabilized with lime.

Robnett and Thompson (1969), based on experience gained with Illinois soils, have indicated that lime may be an effective stabilizer with clay contents ($< 2\ \mu$) as low as 7 percent and, furthermore, soils with a P.I. as low as 8 can be satisfactorily stabilized with lime (Thompson and Robnett, 1970).

Air Force Manual (1969) criteria presented in Table 1 indicates that the P.I should be greater than 12, while representatives of the National Lime Association, Kelly, (1970) indicate that a P.I greater than 10 would be a reasonable criteria to utilize.

2.4 Criteria for Bituminous Stabilization

Asphalt cement, cutback asphalt and emulsified asphalt are the most commonly used bituminous soil stabilization binders. Current design and construction trends, have indicated that stabilization of base courses with asphalt cement is by far the most popular form of bituminous stabilization (Highway Research Board, 1970). In general, those materials which are most effectively stabilized with asphalt cement have lower percentages of fines than those materials which have been successfully stabilized with cutback asphalts and emulsion.

Some of the earliest criteria for bituminous stabilization were developed by the Highway Research Board Committee on Soil - Bitumious Roads. These criteria as revised and published by Winterkorn, (1957) are summarized in

Table 1.

The American Road Builders Association, (1953) made similar recommendations and these are shown in Table 2.

The Asphalt Institute (1965) publication offered guidelines for grading and plasticity requirements for bituminous base courses as follows:

(a) Less than 25 percent passing No. 200 sieve,

(b) Sand equivalent less than 25% and

(c) Plasticity Index less than 6.

Table 1. Types of Soil Bitumen and Characteristics of Soils Empirically Found suitable for their Manufacture. (Winterkorn, 1957)

Sieve Analysis	Soil Bitumen %	Sand Bitumen %	Type of Soils.		
Passing:			A	B	C
37.50mm	-	-	100		
25.00mm	-	-	80 - 100	100	
18.75mm	-	-	65 - 85	80 - 100	100
No. 4	>50	100	40 - 65	50 - 75	80 - 100
No. 10	-	-	25 - 50	40 - 60	60 - 80
No. 40	35-100	-	15 - 30	20 - 35	30 - 50
No. 100	-	-	10 - 20	13 - 23	20 - 35
No. 200	10 - 50	<12, <25	8 - 12	10 - 16	13 - 30

Table 2. Grading and Plasticity Requirements for effective Soil - Bitumen Mixtures. The American Road Builders Association, (1953)

Sieve Size	Percent Passing
No. 40	50 - 10
No. 200	0 - 35
Atterberg Limits	
LL	< 30
PI	< 10

Dunning and Turner, (1965) of the Douglas Oil Company have presented guidelines for emulsion stabilization as shown in Table 3 below.

Table 3. Guidelines for Emulsified Asphalt Stabilization.
Dunning and Turner, (1965).

	Good	Fair	Poor
% Passing No. 200 sieve	3-20	0-3, 20-30	>30
Sand Equivalent %	>25	15-25	<15
Plasticity Index	< 5	5 - 7	> 7

2.5 Summary of Criteria for Selecting Stabilizing Agents.

Criteria have been presented which represent a wide range of opinions as to the types of soils that can be stabilized by certain stabilizers. Most published information gives reference to soils classified either by the AASHTO or Unified soil classification system (UCSC). However, a more appropriate separation of soils for stabilization can be made utilizing Atterberg Limits and sieve analysis (Terrel, et. al, 1979). It should be remembered that both Atterberg Limits and seive analysis are relatively easy tests to perform in the laboratory and both are a necessary input for the AASHTO and Unified soil systems.

Figure 1 presents guidelines that have been proposed for use by the engineer to select the most suitable stabilizer for a given soil (Air Force Manual of standard practice, 1976). Once the stabilizer is selected, detailed laboratory tests should be performed as outlined later in the study.

Once the types of stabilizers have been selected for a particular soil, the engineer should be aware of certain climatic limitations that may restrict the use of the stabilizer. Even though Figure 1 guidelines has been widely accepted by the Highway Engineers, it will still be necessary to confirm this guidelines with the lateritic soils of Nigeria and see whether the figure can be generalized even though it was developed outside Nigeria.

2.6 Lime Stabilization

2.6.1 Introduction

Lime is one of the oldest soil stabilizing agents known to man. Lime was used as a roadway stabilizer by the early Romans as well as other early civilizations. Lime has also been used as a soil stabilizer for large municipal airport runways, military runways and roads, county and municipal roads and streets, and for parking areas. Experience has shown that lime will react with medium, moderately fine and fine grained soils (Robnett and Thompson, 1969).

2.6.2 Types of Lime

In general, the term lime refers to oxides and hydroxides of calcium and magnesium, but not to carbonates. There are various types of lime commercially available. Calcitic quicklime (CaO) and dolomitic quicklime ($CaO+MgO$) are produced by calcining calcitic and dolomitic limestone, respectively. By the controlled addition of water to quicklime, three types of hydrated lime can be produced : high - Calcium, $Ca(OH)_2$; Monohydrated dolomitic, $Ca(OH)_2 + MgO$; and dehydrated dolomitic, $Ca(OH)_2 + Mg(OH)_2$. Boynton, (1966) has further

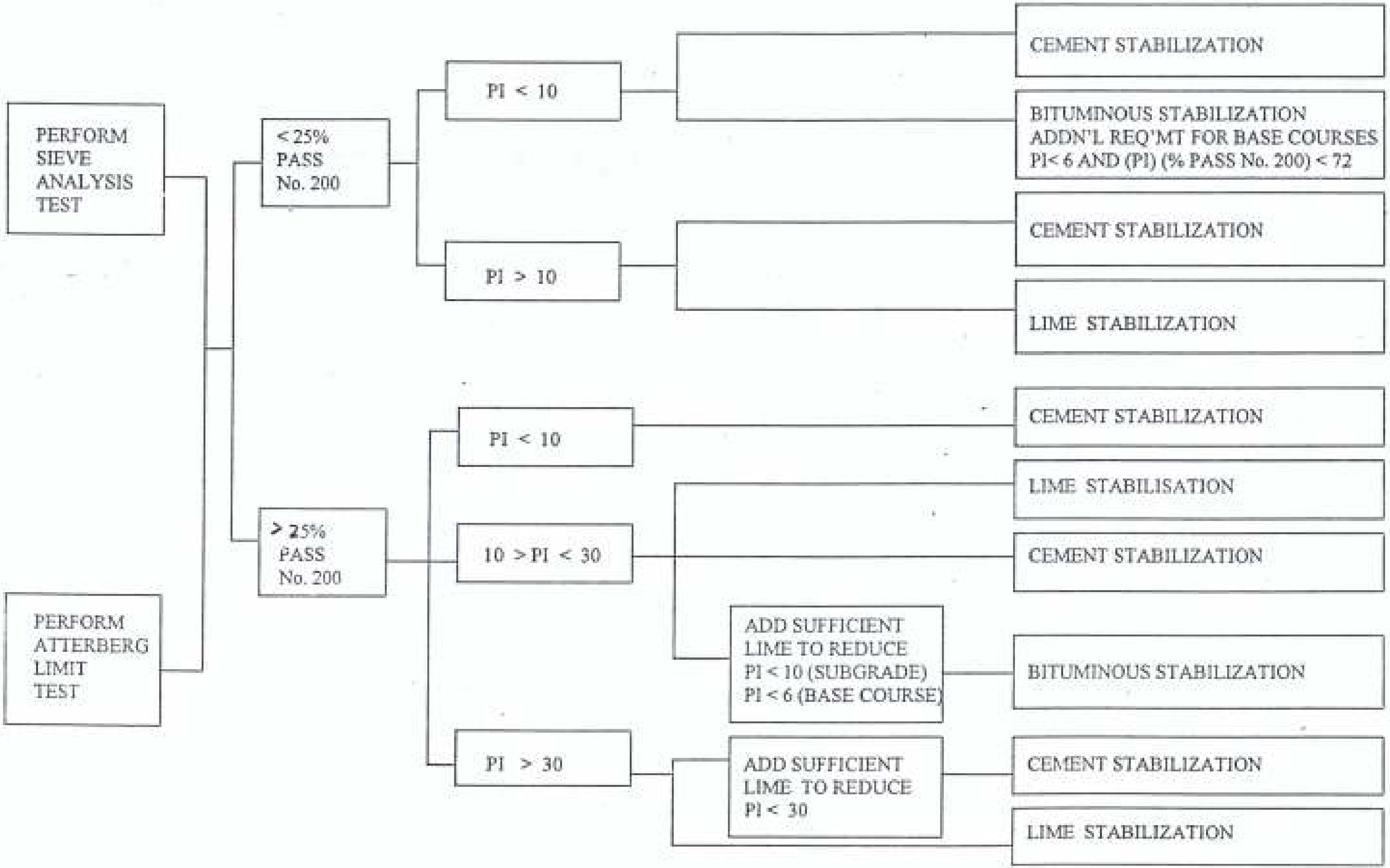


Figure 1. Selection of Stabilizer. (US AIR FORCE, 1976)

explained the types of lime and lime production processes.

Various forms of lime, including products with varying degree of purity, have been successfully utilized as a soil stabilizing agent for many years. However, the most commonly used products are hydrated high calcium lime Ca $(OH)_2$, Monohydrated dolomitic lime Ca $(OH)_2$, : MgO, Calcitic quicklime CaO and dolomitic quicklime CaO. MgO. The use of quicklime for soil stabilization has increased during the past few years in the United States, while in Europe quicklime is the major type used.

Studies by Thompson, (1967) and Nussbaum, (1975) have shown that high Calcium limes are generally more effective for modifying soil plasticity. Dolomitic limes have been found to produce higher cured strength in the Illinios study but most soils do not respond preferentially to dolomitic monohydrate or hydrated Calcitic lime stabilization for strength improvement. It can probably be concluded that either high - calcium or monohydrated dolomitic lime is, in general, satisfactory for use in soil stabilization. In most instances, considerations of local availability and cost are more significant than lime type in selecting a lime source, and this consideration at times may lead to the use of admixtures.

Lime specifications have been prepared by many groups and agencies. Chemical and Physical (primarily particle size) properties are normally the major factors considered in a lime specification. Appropriate quality control testing should be conducted during the course of a project to ensure the quality and uniformity of the lime being incorporated into the construction.

2.6.3 Soil Lime Reactions

The addition of lime to a fine - grained soil initiates several reactions. Cation exchange and flocculation - agglomeration reactions take place rapidly and produce immediate changes in soil plasticity, workability, and the immediate uncured strength and load - deformation properties.

Depending on the characteristics of the soil being stabilized, a soil - lime pozzolanic reaction may occur. The pozzolanic reaction results in the formation of various cementing agents which increase mixture strength and durability. Pozzolanic reactions are time dependent; therefore, strength development is gradual but continuous for long periods of time, amounting to several years in some instances. Temperature also affects the pozzolanic reaction. Temperatures less than 55° to 60°F (13° to 16°C) retard the reaction and higher temperatures accelerate the reaction (Anday, 1963).

2.6.3.1 Cation Exchange and Flocculation - Agglomeration

Practically, all fine - grained soils display cation exchange and flocculation - agglomeration reactions when treated with lime. The reactions occur quite rapidly when soil and lime are intimately mixed.

The general order or replaceability of the common cations associated with soils is given by the lyotropic series, $Na^{+} < K^{+} < Ca^{++} < Mg^{++}$ (Grim, 1953). Cations tend to replace cations to the left in the series and monovalent cations are usually replaceable by multivalent cations. The addition of lime to a soil in sufficient quantity supplies an excess of Ca^{++} and cation exchange will occur, with Ca^{++} replacing dissimilar cations from the exchange complex of the soil. In some cases, the exchange complex may be Ca^{++} saturated before the lime addition, and cation exchange does not take place or is minimized.

Flocculation and agglomeration produce an apparent change in texture, with the clay particles 'clumping' together into larger - sized aggregates. According to Herzog and Mitchell, (1963), the flocculation and agglomeration are caused by the increased electrolyte content of the pore water and as a result of ion exchange by the clay to the calcium form. Diamond and Kinter, (1965) suggested that the rapid formation of calcium aluminate hydrate cementing materials is significant in the development of floccualtion - agglomeration tendencies in soil - lime mixtures.

2.6.3.2 Soil Lime Pozzolanic Reactions

The reactions between lime, water and various sources of soil silica and alumina to form cementing - type materials are referred to as soil - lime pozzolanic reactions.

When a significant quantity of lime is added to a soil, there is a substantial pH increase compared to the pH of natural soils. The solubilities of silica and alimina are greatly increased at elevated pH levels (Keller, 1957). The extent to which the soil - lime pozzolanic reaction proceeds is influenced primarily by natural soil properties.

2.6.3.3 Carbonation

Lime carbonation is an undesirable reaction which may also occur in soil - lime (lime reacts with carbon dioxide to form a carbonate, $CaO + Co_2 \text{-->} CaCo_3$). Construction should be carried out in such a fashion that carbonation is minimized, because it reduces workability and plasticity of soil - lime mixtures.

Major actions to avoid are long exposure of the lime prior to mixing with the soil and long intensive mixing and processing times. It is recommended that prior to mellowing, the mixture be compacted using a rubber -tired roller.

More extensive and detailed background information on basic soil - lime reactions can be found in papers written by Diamond and Kinter, (1965) and a comprehensive publication by Stocker, (1972).

2.6.3.4 Summary

Soil - Lime reactions are complex and not completely understood at this time. However, sufficient basic understanding and successful field experience are available to provide the basis of an adequate technology for successfully utilizing soil - lime stabilization under a wide variety of conditions.

2.7 Asphalt Stabilization

2.7.1 Introduction

Asphalt is one of the oldest adhesives known to man. As early as 3800 B.C. asphalt was being used as mortar for building stones and paving blocks. Some of the reasons for utilizing asphalt stabilization are listed below:

(a) Waterproofing fine-grained subgrade soils,

(b) Construction expediency,

(c) Upgrading of marginal materials,

(d) Reduction of pavement layer thickness to conserve materials, reduce cost, and conserve energy,

(e) Provide temporary and permanent wearing surface and

(f) Reduce dusting.

2.7.2 Types of Asphalt

Asphalt most commonly used are refined from petroleum. Asphalt cement is the basic refined material and is the hard, high - molecular, weight fraction of crude oil. Asphalt cement at ambient temperature is a semi - solid. Liquid asphalt products are most often derived from asphalt cement by blending petroleum distillates to form cutbacks or by emulsifying with water to form emulsified asphalts.

2.7.2.1 Asphalt Cements

Asphalt cements are graded on the basis of consistency or viscosity. Three different techniques are utilized to grade asphalts on this basis: Penetration at 77°F (25°C) of original asphalt, viscosity at 140°F (60°C) of original asphalt, and viscosity at 140°F of laboratory - aged asphalt. Specifications have been developed by AASHTO and ASTM, (Table 4). Typical penetration grades are 40 - 50, 60-70, 85 -100, 120 - 150, and 200 - 300. Typical viscosity grades are AC-5, AC - 10, AC - 20 and AC - 40.

Asphalt cements must be heated to obtain a mixing and spraying consistency. The curing or setting time of mixture utilizing asphalt cements occurs as the heat required for mixing, lay down, and compaction dissipates.

Table 4 Asphalt Specifications. (AASHTO and ASTM)

Material		Specifications	
		AASHTO	ASTM
Asphalt cement	Penetration basis	M20	D946
	Viscocity basis	M226	D3381
Cutback	Rapid curing	M81	D2028
	Medium curing	M82	D2027
	Slow curing	M141	D2026
Emulsion	Anionic	M140	D977
	Cationic	M208	D2397

2.7.2.2 Cutback Asphalts

Cutbacks are combinations of asphalt cement and a petroleum diluent blended to provide viscosities suitable for mixing and spraying at relatively low temperatures. Cutbacks are graded based upon curing time and consistency. Curing time is varied by the solvent used in cutting back the asphalt cement, while the viscosity (consistency) is controlled by the amount of solvent. Rapid - cure cutbacks (RC) use a naphtha or gasoline type solvent, medium - cure cutbacks (MC) use kerosene - type solvents, and slow - cure cutbacks (SC) use low volatility oils or are made during the refining process.

Typical grade designations for viscosity graded RC, MC, and SC materials are as shown below, where the numbers represent the kinematic viscoucities.

(a) RC - 70, RC - 250, RC - 800, RC - 3000

(b) MC - 30, MC - 70, MC-250, MC-800, MC - 3000 and

(c) SC - 70, SC-250, SC- 800, SC - 3000

2.7.2.3 Emulsified Asphalts

Emulsified asphalts are mixtures of asphalt cement, water and an emulsifying agent. Anionic emulsions are manufactured with anionic (negatively charged) emulsifying chemicals. Cationic emulsions are manufactured with cationic (positively charged) emulsifying chemicals. The type and amount of emulsifying agent will determine to a large degree the setting characteristics of the asphalt emulsion. Rapid - setting (RS), medium - setting (MS) and slow - setting (SS), anionic and cationic emulsions are manufactured.

2.7.3 Mechanisms of Asphalt Stabilization

The mechanism involved in the stabilization of soils and aggregates with asphalt differ greatly from those involved in lime stabilization. The basic mechanism involved in asphalt stabilization of fine grained soils is a water proofing phenomenon. Soil particles or soil agglomerates are coated with asphalt, resulting in a membrane that prevents or impedes the penetration of water which, under normal conditions, would result in a decrease of shear strength, compressive strength, tensile strength, flexural strength, and elastic modulus. In addition, asphalt stabilization can improve durability characteristics. Since the soil particles or aggregates are coated with water - repelling asphalt film, the soil is resistant to the detrimental effects of water such as volume change due to alternating wet - dry and freeze - thaw cycles.

In non-cohesive materials, such as sand and gravels, crushed gravels and crushed stones, two basic mechanisms are active: waterproofing and adhesion.

2.7.4 Soils Suitable for Asphalt Stabilization

2.7.4.1 Fine Grained Soils

Fine - grained soils may be stabilized with asphalt, depending upon the plasticity characteristics of the soil and the amount of material passing the No 200 sieve. Due to the extremely high surface area of the finer soil particles, a large percent of asphalt would be required to coat all of the soil surfaces. Since this is virtually impossible, agglomerations of particles are coated with economical percentages of asphalt. The gradation of fine grained soils suitable for asphalt stabilization are shown on Table 5 (Herrin, 1960).

2.7.4.2 Coarse Grained Soils

Cohensionless soils (plasticity index less than 6) suitable for asphalt stabilization are shown on Table 5 and identified as sand - bitumen and sand - gravel - bitumen. In addition, cohensionless soils identified as suitable for hot mix asphalt concrete by AASHTO, ASTM, and Some States in the USA are in general acceptable. Asphalt - stabilized materials made with well - or dense - graded aggregates have higher strength, e.t.c. than the more one - sized sand - asphalt mixtures.

Table 5. Engineering Properties of Materials Suitable for Asphalt Stabilization. (AASHTO and ASTM)

% Passing sieve	Sand-Asphalt	Soil - Asphalt	Sand-Gravel Asphalt
37.50mm			100
25.00mm	100		
18.75mm			60 - 100
No 4	50 - 100	50 - 100	35 - 100
No 10	40 - 100		
No 40		35 - 100	13 - 50
No 100			8 - 35
No 200	5 - 12	Good 3 - 20 Fair 0-3 & 20-30 Poor > 30	
Liquid Limit		Good < 20 Fair 20 - 30 Poor 30 - 40 Unstable >40	
Plasticity Index	10	Good < 5 Fair 5 - 9 Poor 9 - 15 Unstable >12-15	10

* Note: Table 5 deals with asphalt (black cementitious materials predominantly bitumen) while Tables 1 and 2 based their properties on bitumen (natural or manufactured high molecular weight hydrocarbons).

CHAPTER THREE

METHODOLOGY

3.1 Preambles

Stabilization has been receiving much attention by Transportation and Highway Engineers. It is therefore necessary to know the effects of stabilizers such as lime and asphalt on the lateritic Soils of Nigeria and observe the various parameters necessary for their performances and effectivenesses as stabilizers.

Asphalt has been utilized successfully for stabilization according to Nixon and Skipp, (1957) and with only partial success by Winterkorn and Chandrasekharam, (1951). This study is designed to confirm the state of the earth findings. In order to have a wide coverage of the lateritic soil samples for the project, five samples named A - E were taken and their properties are given below, showing the point of collection, the colour, the classification, specific gravity and the Optimum moisture content.

3.2 Materials

(a) Lateritic Soil Samples

(i) Sample A

Fine clay from the borrow pit of Federal Polytechnic, Ado-Ekiti.

This is a yellowish-brown soil classified by USCS as (CL) and AASHTO as (A-6) with a specific gravity of 2.57 and OMC of 13%.

(ii) **Sample B**

Coarse clay from the borrow pit of Federal Polytechnic, Ado-Ekiti. This is a brownish soil classified by USCS as (SM) and AASHTO as (A-2-6) with a specific gravity of 2.59 and OMC of 11%.

(iii) **Sample C**

This is a brownish-red soil from Ilawe-Ado road, classified by USCS as (SC) and AASHTO as (A-6) with a specific gravity of 2.35 and OMC of 16%.

(iv) **Sample D**

This is a brownish-red soil from Akure-Ado road, classified by USCS as (SM) and AASHTO as (A-2-6) with a specific gravity of 2.46 and OMC of 11%. This has a similar property with Sample B.

(v) **Sample E**

This is a brownish-red soil from OSUA-Ado road, classified by USCS as (SC) and AASHTO as (A-6) with a specific gravity of 2.56 and OMC of 16%. This is also similar with sample C.

3.2.1 Basis for Selection of Samples.

It can be observed that sample B and D are closely identical sand-silt (SM) soil, while samples C and D presented identical Sand-clay (SC) soil. We therefore selected three samples A, C and D because of their distinct features, that is, samples A is a low compressibility clay (CL), sample C is a Sand-clay and Sample D is a sand -silt.

(b) Cutback asphalt, rapid curing (RC)

This was obtained locally from a road contractor's drum at the Federal Polytechnic, Ado-Ekiti. Petrol was used as the thinner, the asphalt is a rapid caring (RC) cutback, M81 (AASHTO) or D2028 (ASTM).

(c) Lime.

This was obtained as calcitic quicklime (CaO).

3.3 Testing Procedures

To achieve the objectives of this project the procedures followed are:

(a) Five different soil samples named A - E were obtained in lateritic soil deposits in Ado - Ekiti area, three of these samples, A,C, and D having distinct features were fully analysed.

(b) Cutback asphalt obtained was thinned with petrol and added as stabilizer in 2, 6, and 10% content by weight of the samples and laboratory tests were carried out on the resulting mixtures.

(c) Identification, classification and mineralogical tests were performed on the samples, and they were then classified based on AASHTO and USCS classification systems. Compaction, strength and penetration tests were also performed.

(d) From the results, the relationships between the following properties when treated with the asphalt admixtures were graphically determined:

i. Particle size distribution

ii. Variation of dry densities with moisture,

iii. Variation of unconfined compressive strength and curing period in days.

iv. Variation of maximum dry densities (MDD) optimum moisture contents (OMC), california bearing ratio (CBR) and unconfined compressive strength (UCS) with asphalt contents.

(e) Stress - strain curves for the treated samples.

(e) The various results and graphs were then analysed and compared with a similar analysis with lime available from another study.

(f) Appropriate conclusions and recommendations were then drawn.

3.4 Pre-Treatment of Samples

The following precautions were observed in testing the samples.

(a) The samples were stored in bags prior to their use.

(b) The samples were air-dried, broken down with mortar and pestle and all large particles and impurities removed.

(c) Mixing was done mechanically at the OMC of the natural soil with varying proportions of additives.

(d) For the asphalt samples, of the resulting mixture, mixing was done mechanically after thinning with petrol.

(e) Moulding of test- specimens was started as soon as possible after completion of mixing. Specimens were moulded in CBR mould using the modified AASHTO compaction method. The unconfined compression test mould was used for compressive strength determination.

(f) Testing of specimens for relevant characteristics were then carried out using the following standard tests:

(i) Compaction : AASHTO T180 - 70 method.

(ii) CBR : AASHTO T193 - 63 method.

(iii) UCS : BS 1377 method.

(g) The normal procedures for lime is assumed to have been followed in the study covering lime.

CHAPTER FOUR

LABORATORY RESULTS AND

ANALYSIS OF TREATED SOIL SAMPLES

4.1 Identification and Classification Tests

The particle size distributions of the samples are as shown in Table 6 while the results of the physical properties are shown in Table 7.

Figure 2 is the particle size distribution curve.

A full description of the samples based on USCS and AASHTO classifications are shown in Tables 8 and 9. In order to establish correlation between composition and chemical stabilization, the chemical and mineralogical composition of the samples were determined using the procedures given in BS 1377. The chemical analysis results are presented in Table 10.

Table 6. Particle Size Distribution of Samples A, C and D.

Sample A

Sieve Opening	Wt. Retained (g)	% Retained	% Passing
9.51	2	0.4	99.6
4.76	4	0.8	98.8
3.35	7	1.4	97.4
2.36	6	1.2	96.2
1.18	28	5.6	90.6
0.600	40	8.0	82.6
0.425	23	4.6	78.0
0.300	27	5.4	72.8
0.150	42	8.4	64.2
0.075	29	5.8	58.4

Sample C			
9.51	31	6.2	93.8
4.76	40	8.0	85.8
3.35			
2.36	58	11.6	74.2
1.18	30	6.0	68.2
0.600	38	7.6	60.6
0.425	19	3.8	56.8
0.300	15	3.0	53.8
0.150	21	4.2	49.6
0.075	11	2.2	47.4
Sample D			
9.51	41	8.2	91.8
4.76	94	18.8	73.0
3.35			
2.36	101	20.2	52.8
1.18	31	6.2	46.6
0.600	24	4.8	41.8
0.425	12	2.4	39.4
0.300	8	1.6	37.8
0.150	14	2.8	35.0
0.075	9	1.8	33.2

From table 6, it can be noticed that the weight retained on Sample A is 2g, on Sample C it is 31g, on Sample D it is 41g, all at the largest sieve opening of 9.51mm. This indicates that sample A is the finest and Sample D is the most granular.

Samples A, C, D

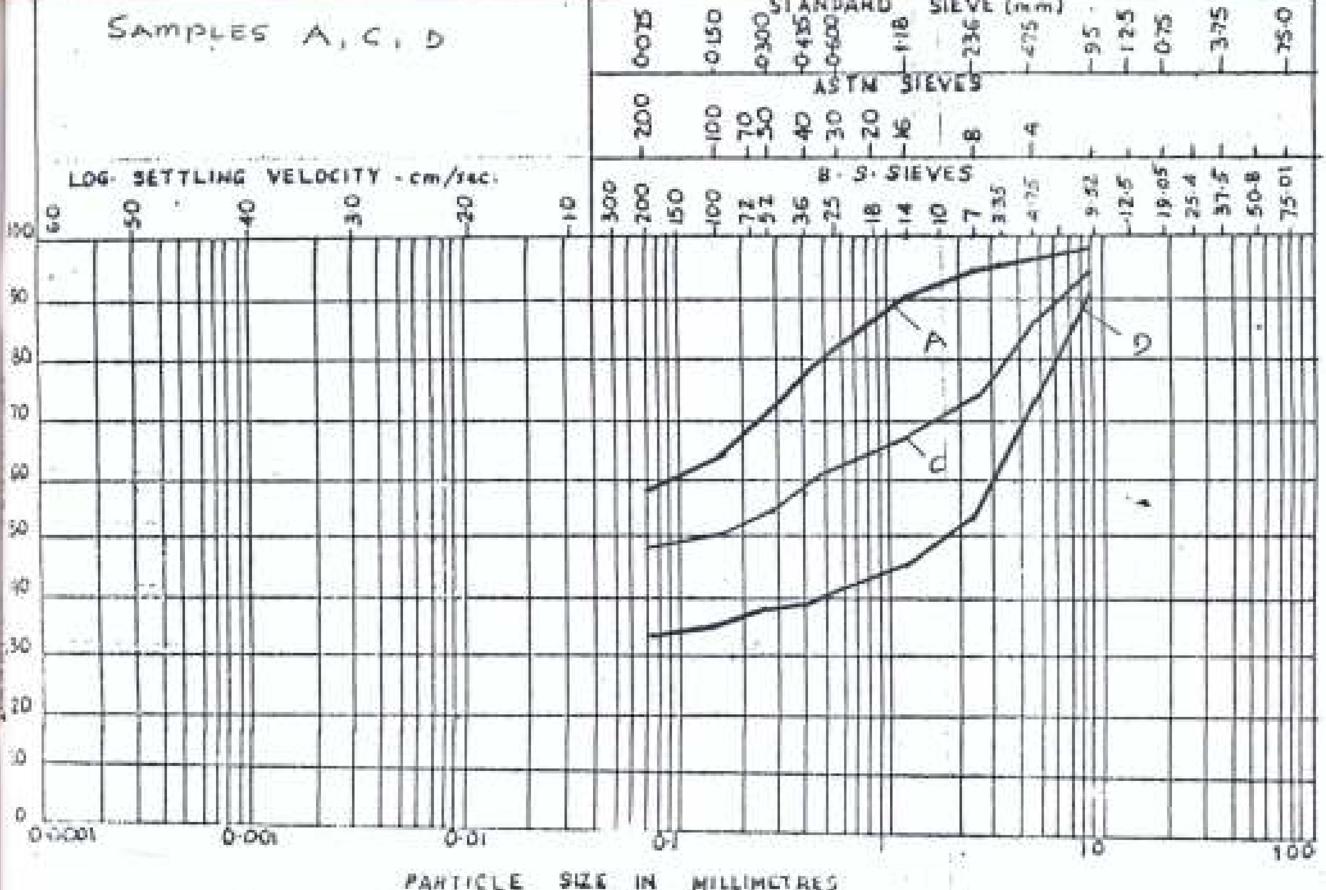


Fig 2 Particle Size Distribution Curves.

Table 7. Results of The Physical Properties of The Samples.

Property	Sample A	Sample B	Sample C	Sample D	Sample E
Colour	Yellowish Brown	Brownish	Brownish Red	Brownish Red	Brownish Red
G	2.57	2.59	2.35	2.46	2.56
OMC%	13	11	16	11	16
LL %	39	28	40	39	40
PL %	22	16	27	24	26
PI %	17	12	13	15	14
% Passing #200 B.S sieve	58.4	26.0	47.4	33.2	42.2
MDD (mg/m^3)	1798	2175	1767	1958	1820
GI	8	0	4	1	3
Total Org. Content. %	48.22	48.20	48.40	41.01	49.07
USCS Class.	CL	SM	SC	SM	SC
AASHTOClass.	A-6	A-2-6	A-6	A-2-6	A - 6

Table 7 also confirms Sample A as the finest because 58.4 percentage passes #200 sieve while 47.4 percentage passes in Sample C and 33.2 percentage passes in Sample D reflecting that it is the most granular of the Samples.

The distinct features of the Samples are also seen. Samples B and D are similar and Samples C and E are also similar.

Table 8. Classification of Samples (USCS Method)

Sample	Classification	Description
A	CL	Fine grained soil, inorganic clay of low to medium plasticity, silty clay.
B	SM	Coarsed grained, silty sand, (Sand - silt mixture)
C	SC	Sand - clay mixture
D	SM	Silty sand
E	SC	Sand - clay mixture.

Table 9. Classification of Samples (AASHTO Method)

Sample	Classification	Rating As Subgrade mat.	Description
A	A - 6	Fair - Poor	Clayey soil
B	A - 2 A - 2 -6	Excellent - Good	Granular silty or clayey gravel or sand.
C	A - 6	Fair - Poor	Clayey soil
D	A - 2 A - 2 -6	Excellent - Good	Granular silty or clayey gravel or sand.
E	A - 6	Fair - Poor	Clayed soil.

Table 10. Chemical Analysis Result

Sample	(1) S_iO_2	(2) Al_2O_3	(3) Fe_2O_3	LOI	pH	Undet-ermined	$\frac{S_iO_2\ (1)}{(2)+(3)}$
A	45.10	38.00	1.53	12.17	5.8	3.20	1.14
B	52.43	34.60	1.67	9.78	6.5	1.52	1.45
C	51.00	37.15	0.85	8.15	6.1	2.85	1.34
D	54.40	34.10	0.50	10.71	5.7	0.29	1.57
E	55.03	31.04	0.64	11.97	5.8	1.32	1.74

LOI - Loss on Ignition

4.1.1 Chemical And Mineralogical Composition

For all the samples, the silica - alumina ratio is less than 2, but the samples are still lateritic soils with no presence of sulphates and carbonates. The pH test results also show that the soil samples are acidic. By using the clay identification chart, all the samples consist mainly of kaolinite (Oluyemi, 1999). The S_iO_2 is a minor element that affects the physical properties of the Samples. The S_iO_2 in Sample D is 54.40% signifying a more granular Sample than A and C. The major elements affecting laterization, that is, the physical properties of the Samples are the Al_2O_3 and Fe_2O_3 contents. The iron content is more in Sample A because it is a low compressibility clay (CL) and smaller in the others, being SC and SM.

4.2 Characteristics of Treated Samples.

The samples were treated with both lime and asphalt acting as stabilizers. Varying percentages (2, 6 and 10) were added to the samples by weight. For proper investigation and comparison of results, the following tests were carried out on the treated samples :

(a) Compaction test

(b) Unconfined compressive strength test

(c) California Bearing Ratio (C.B.R) test

(d) Deformation test.

4.2.1 Compaction Characteristics

The results of compaction using lime and asphalt are summarized in Table 11. They are to determine the moisture - density relations of the samples when treated and stabilized with lime and asphalt. The variations of dry density and moisture contents are shown in Figures 3 - 4.

For a given compactive effort, soil - lime mixtures have a lower maximum dry density and a higher optimum moisture content than the untreated soil (Terrel, 1979). It was observed in Table 11, that lime reduces the maximum dry density (MDD) but increases the Optimum Moisture Content (OMC), and this continues as the percentages of lime increases. On the other hand, as percentages of asphalt increases the optimum moisture content (OMC) drops and maximum dry density (MDD) increases.

Table 11. Summary of OMC and MDD.

		Sample Locations					
		A		C		D	
%	Stabilizer	% OMC	kg/m^3 MDD	% OMC	kg/m^3 MDD	% OMC	kg/m^3 MDD
2	Asp.	15.2	1690	13.4	1735	12.0	1840
	Lime	14.0	1675	11.6	1687	12.5	1640
6	Asp.	11.8	1725	14.2	1692	11.4	1750
	Lime	17.0	1700	16.2	1710	15.0	1790
10	Asp.	7.6	1830	11.5	1645	11.0	1610
	Lime	22.5	1567	23.4	1615	16.8	1780

* Note that the reference point is lime and not the natural soil, therefore 0% stabilizer is not included.

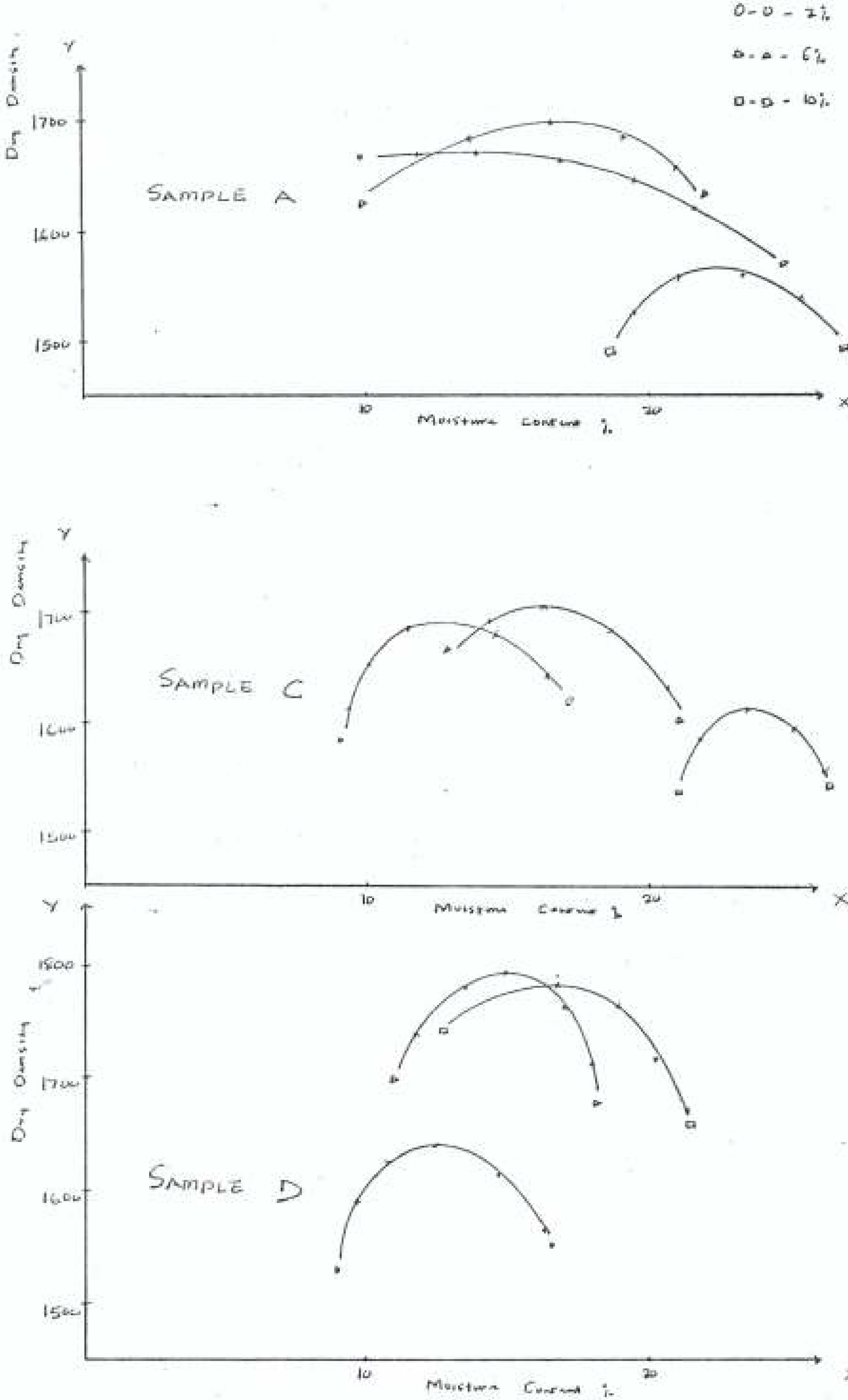


Fig. 3 Variation of Dry Densities and Moisture Contents with Lime.

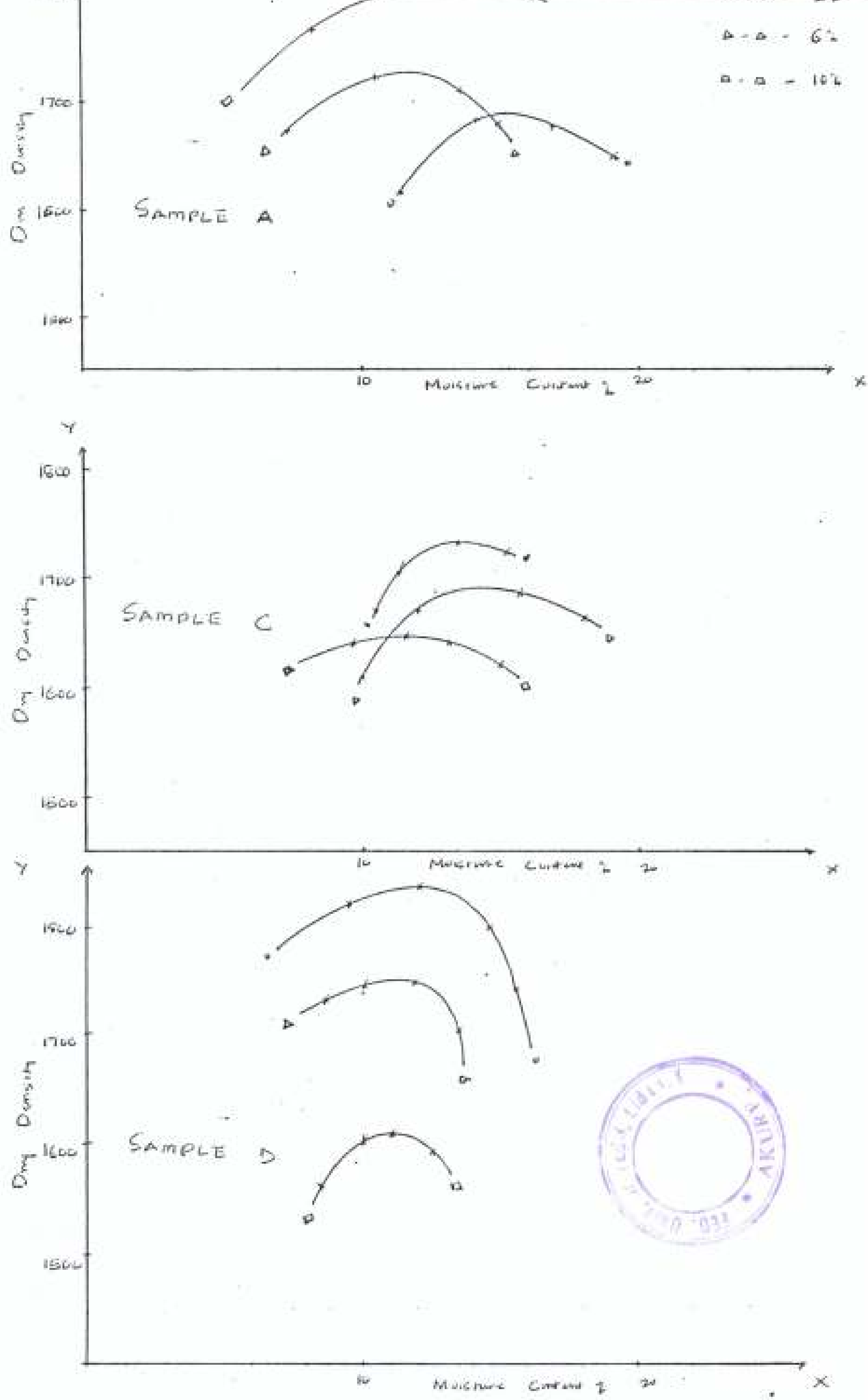


Fig. 4 Variation of Dry Densities and Moisture Contents with Asphalt.

4.2.2 Unconfined Compressive Strength (UCS)

The results of the unconfined compressive strength of the samples when treated with both lime and asphalt are presented in Table 12. The variations of UCS with curing periods for the samples, when treated with lime and asphalt contents are shown in Figures 5 and 6, while Figures 7 - 12 show the variations of MDD, OMC, CBR and UCS for the samples when treated with lime and asphalt.

The percentages of admixtures used are 2, 6 and 10 percent by weights of the samples. Tests were run on both the soaked and unsoaked samples and results were obtained on 7, 21, and 28 days.

The unconfined compression test is a simple and effective test for evaluation of the properties of treated soils.

Soil - lime mixture strength is measured by cured mixture strength minus strength of natural soils, and varies substantially (Thompson, 1967, 1966). Soil - lime mixture strength increases with many soils displaying increases greater than (690 Kpa). Field data indicate that with some soil - lime mixtures, strength continues to increase with time up to ten years or more.

The difference between the compressive strength of the natural and lime-treated soils is an indication of the degree to which the soil - lime - pozzolanic reaction has proceeded (Thompson, 1966). Substantial strength increase indicates that the soil is reactive with lime and can be stabilized to produce a quality paving material. Figures 5 and 6 shows the variations of UCS and curing periods for samples A, C and D when treated with lime and asphalt.

The addition of lime at 6 percent and for 28 days gave a UCS of above 60 KN/m^2 while the addition of asphalt at the percentage and curing days gave a UCS of above 35 KN/m^2. On sample D (granular) the addition of lime at 10 percent gave a constant UCS of 100 KN/m^2 for the curing days while the addition of asphalt on the same sample D at same percentage only produced a maximum UCS of about 18 KN/m^2 at 28 days.

In Figures 11 and 12, when asphalt was added to Sample D at 10 percent, the CBR was just about 4%, whereas when lime was added to the same sample D at same percent, the CBR was above 200%. These observations shows that lime performed better on the samples both for UCS and CBR values.

Table 12. Results of Unconfined Compression Strengths.

2 % Stabilizer Content.

Sample	Age (Days)	Stabilizer	Unsoaked compressive Strength kN/m²	Soaked Compressive Strength kN/m²
A	7	Asph.	38.00	-
		Lime	36.00	32.00
	21	Asph.	36.00	-
		Lime	36.00	34.00
	28	Asph	44.00	-
		Lime	34.00	28.00
C	7	Asph.	20.00	-
		Lime	38.00	46.00
	21	Asph.	12.00	-
		Lime	70.00	60.00
	28	Asph	16.00	-
		Lime	55.00	48.00
D	7	Asph.	18.00	-
		Lime	75.00	40.00
	21	Asph.	32.00	-
		Lime	80.00	46.00
	28	Asph	22.00	-
		Lime	90.00	48.00

6 % Stabilizer Content.

Sample	Age (Days)	Stabilizer	Unsoaked Compressive Strength. KN/m²	Soaked Compressive Strength. KN/m²
A	7	Asph.	28.00	-
		Lime	40.00	36.00
	21	Asph.	26.00	-
		Lime	55.00	46.00
	28	Asph	36.00	-
		Lime	60.00	50.00
C	7	Asph.	14.00	-
		Lime	75.00	48.00
	21	Asph.	10.00	-
		Lime	60.00	50.00
	28	Asph	14.00	-
		Lime	80.00	75.00
D	7	Asph.	10.00	-
		Lime	90.00	46.00
	21	Asph.	20.00	-
		Lime	100.00	50.00
	28	Asph	18.00	-
		Lime	100.00	48.00

10 % Stabilizer Content.

Sample	Age (Days)	Stabilizer	Unsoaked Compressive Strength kN/m^2	Soaked Compressive Strength kN/m^2
	7	Asph.	12.00	-
		Lime	26.00	26.00
A	21	Asph.	18.00	-
		Lime	32.00	38.00
	28	Asph	24.00	-
		Lime	32.00	60.00
	7	Asph.	10.00	-
		Lime	75.00	65.00
C	21	Asph.	8.00	-
		Lime	80.00	70.00
	28	Asph	8.00	-
		Lime	80.00	85.00
	7	Asph.	4.00	-
		Lime	100.00	60.00
D	21	Asph.	8.00	-
		Lime	100.00	65.00
	28	Asph	18.00	-
		Lime	100.00	70.00

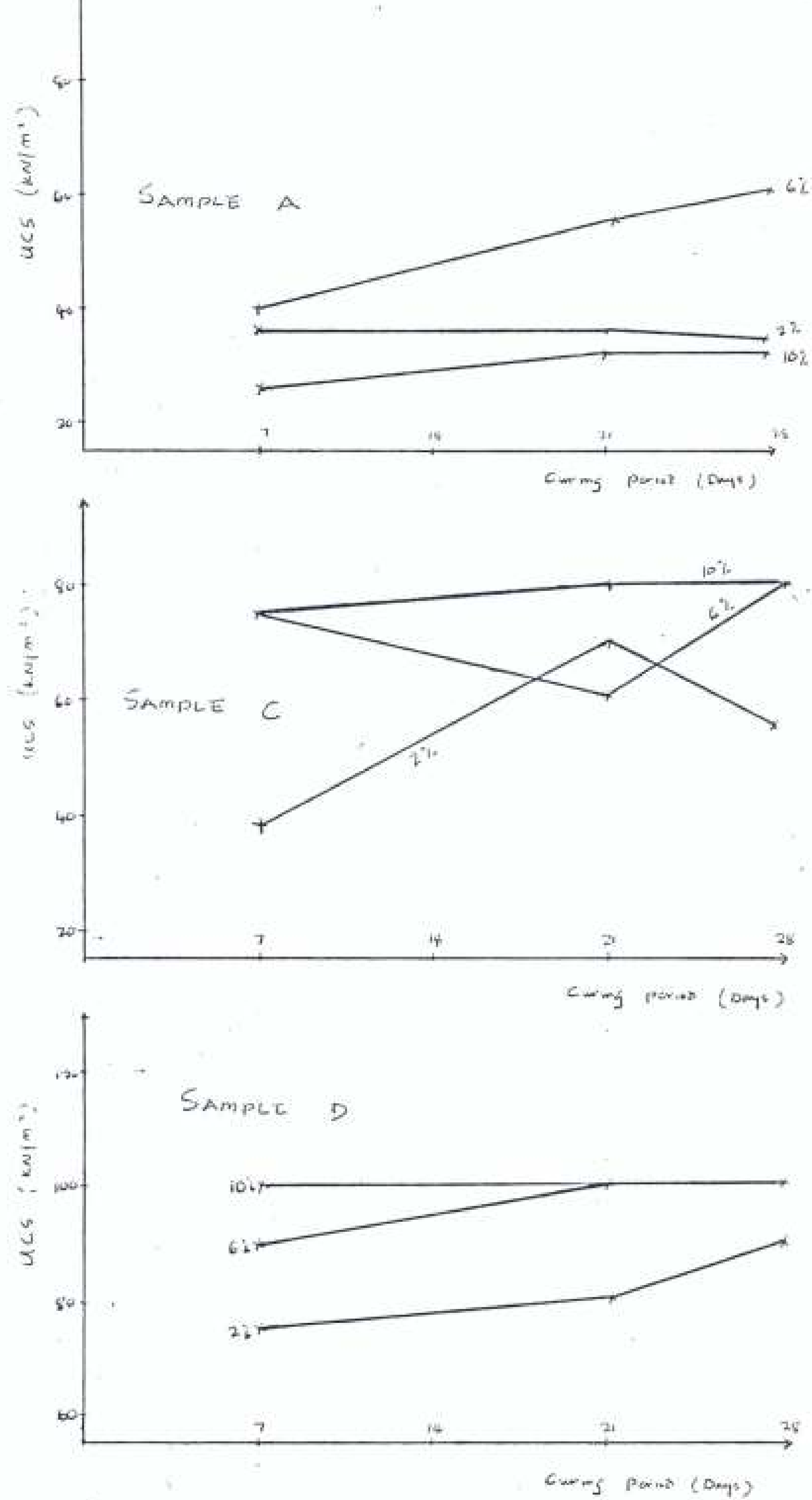


Fig 5. Variation of UCS and Curing periods with Lime Contents

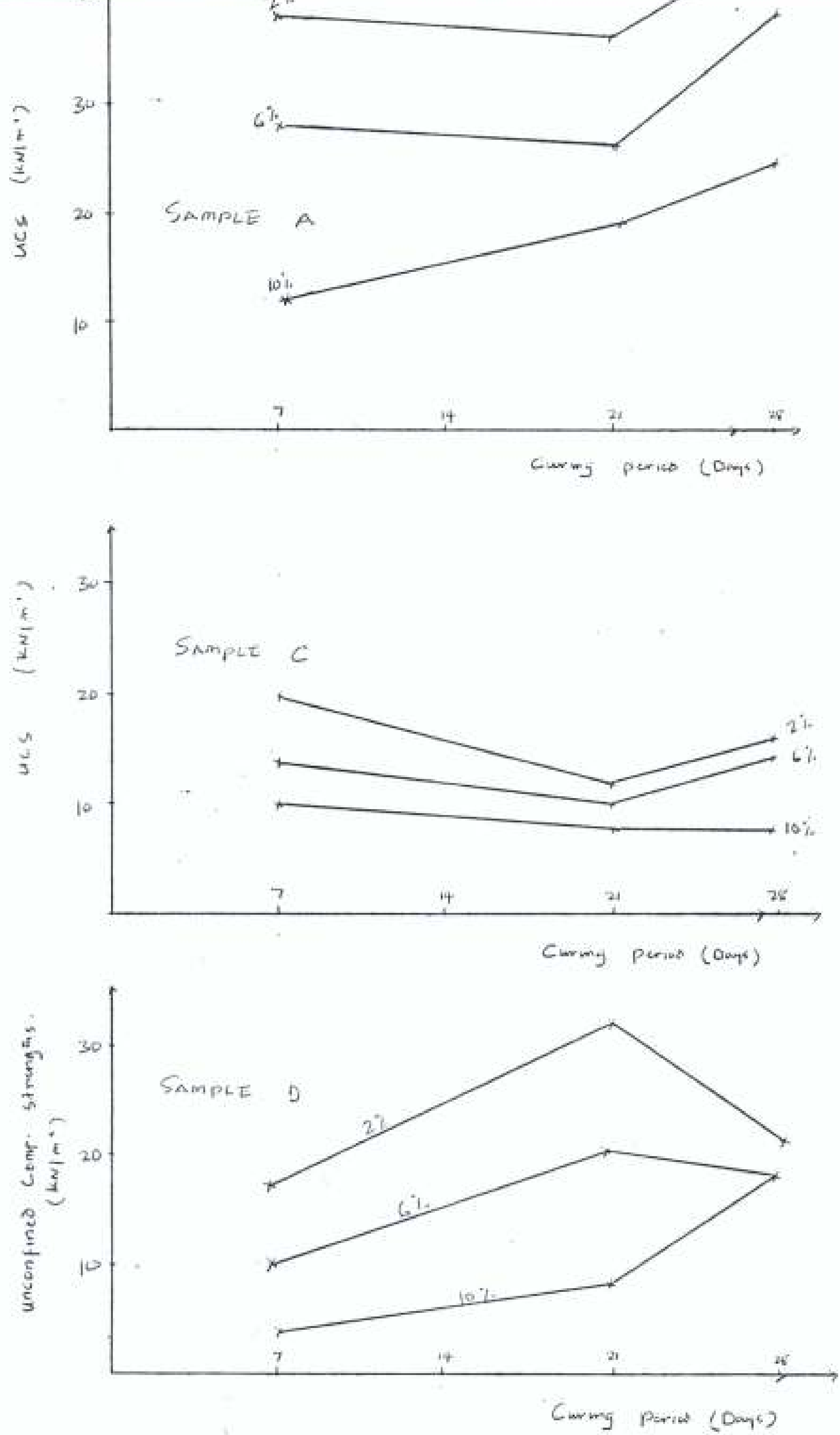


Fig. 6. Variation of UCS and curing periods w.r.t Asphalt Contents

4.2.3 California Bearing Ratio (CBR)

The results of the CBR for the treated samples are presented in Table 13 and Figures 7 - 12. The percentages of lime and asphalt stabilizers used are 2, 6 and 10 percent by weights of the samples. Results were obtained at 7, 21 and 28 days. Few results were obtained for the soaked samples due to time constraint, the samples were many and the container holding the soaked samples was limited in volume since curing was done by soaking samples in a water filled tank which is the common method of curing, hence some missing data in Table 13 - 15.

CBR testing procedures have been extensively used to evaluate the strength of lime stabilized soils. Many agencies have arbitrarily adopted this technique because of their familiarity with the test. In reality, CBR is not appropriate for characterizing the strength of soil - lime mixtures under all conditions.

The CBR values for cured soil - lime mixtures are quite large and definitely indicate the extensive development of pozzolanic cementing agents. For those mixtures that display CBR values of 100 or more, it is quite apparent that CBR test results have little practical significance. If extensive pozzolanic cementing action has not developed due to either lack of curing time or non-reactivity of the treated soil, the CBR value may serve as a general strength indicator.

From Table 13 it could be observed that lime treated samples have greater CBR values than asphalt treated samples. The CBR value increases as curing day and percentage of stabilizer increases.

Table 13. California Bearing Ratio Results.

			C. B. R			
Sample	% Stabi-lizer	Days	Asph.	Lime.	Asph.	Lime
			UNSOAKED		SOAKED	
SAMPLE A (Poly fine)	2	7 21 28	3.80 3.90 4.00	28.00 37.00 42.00	2.50 2.55 2.00	2.40 2.00 2.00
	6	7 21 28	3.50 4.00 3.50	92.00 103.00 101.00	1.50 2.00 1.80	
	10	7 21 28	2.00 2.00 2.00	16.00 23.00 15.00	- - -	
SAMPLE C (Ilawe)	2	7 21 28	2.50 3.00 3.50	- - -	2.50 2.00 1.50	
	6	7 21 28	2.50 2.00 2.50	- - -	- - -	
	10	7 21 28	2.00 2.00 2.00	37.00 78.00 52.00	- - -	
SAMPLE D (Akure)	2	7 21 28	4.00 4.50 4.00	- - -	2.00 2.50 2.00	
	6	7 21 28	2.00 2.50 2.60	13.00 100.00 142.00	3.00 3.00 3.00	10.00 44.00 90.00
	10	7 21 28	3.00 4.00 4.00	221.00 224.00 253.00	- - -	- - -

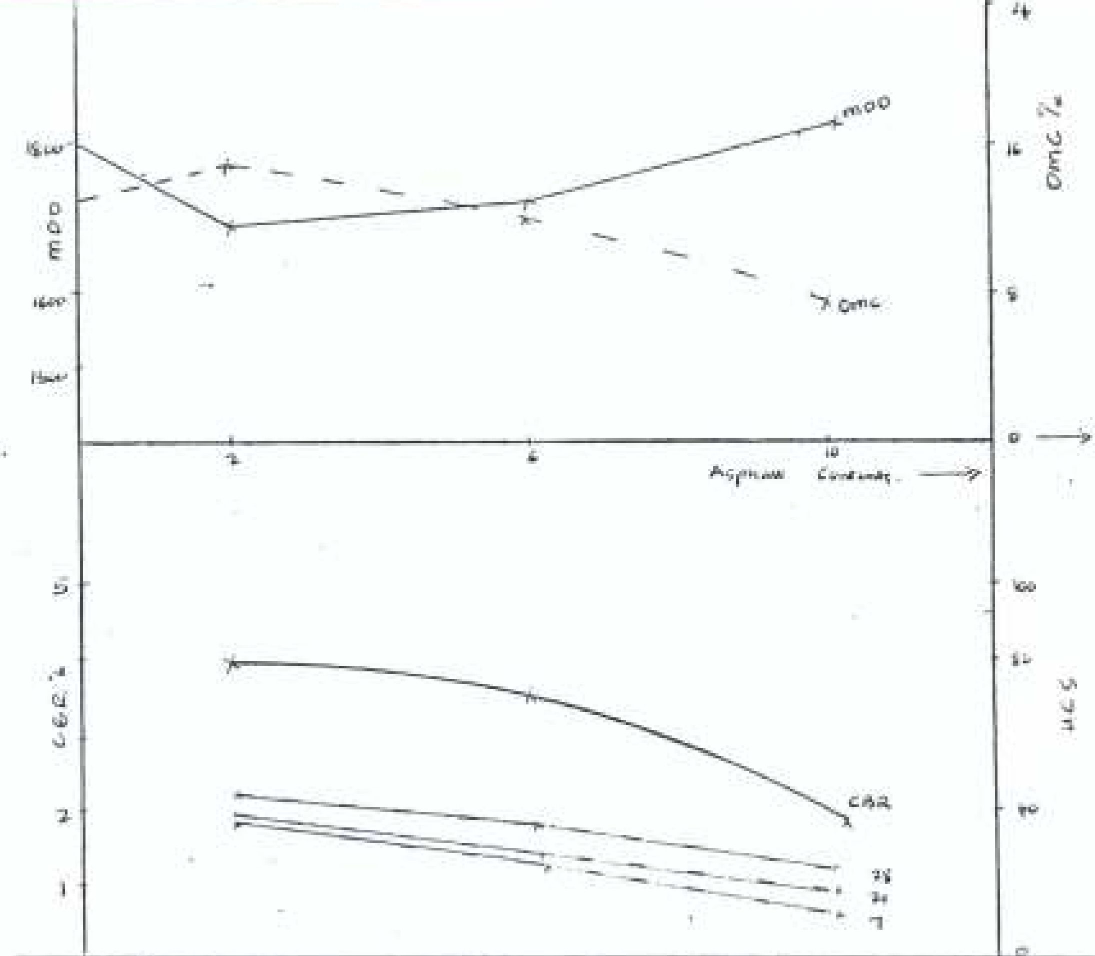


Fig. 7. Variation of MDD, OMC, CBR, and UCS with Asphalt Content (Sample A)

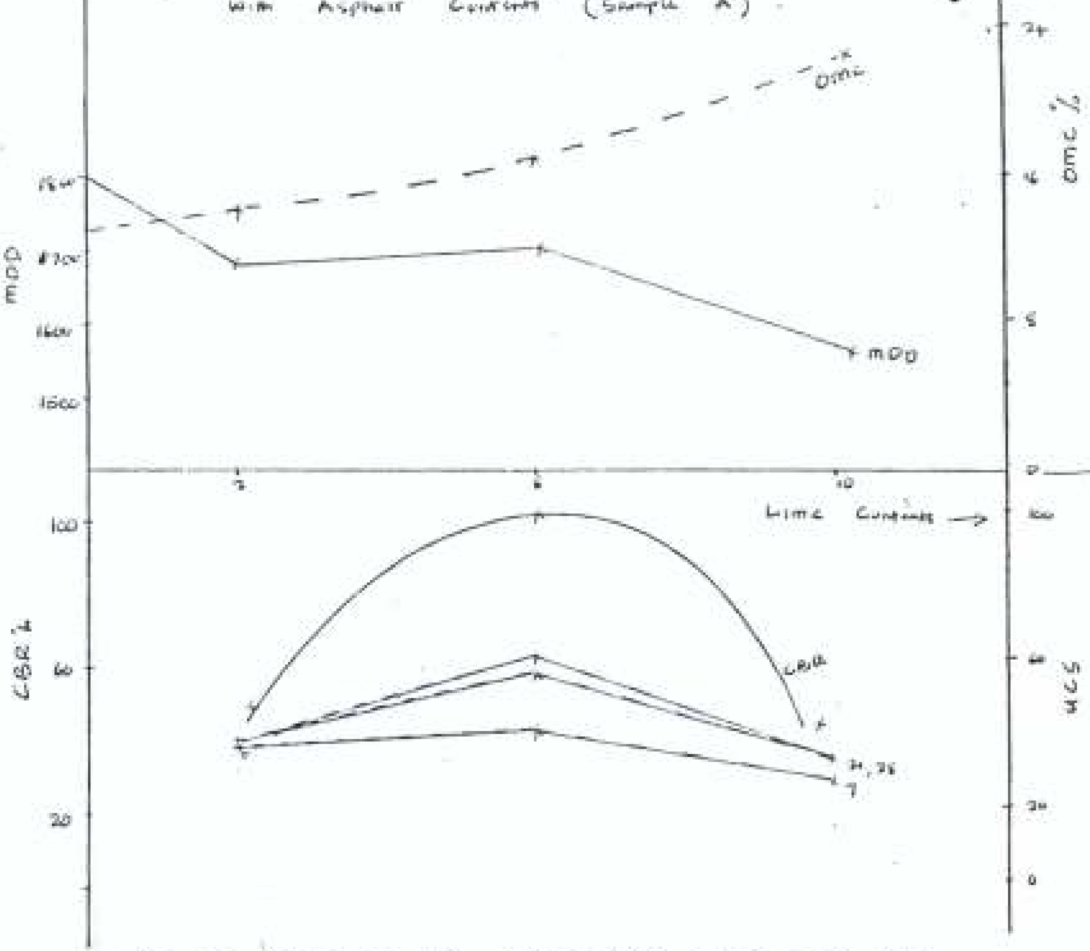


Fig. 8. Variation of MDD, OMC, CBR and UCS with Lime Content. (Sample A)

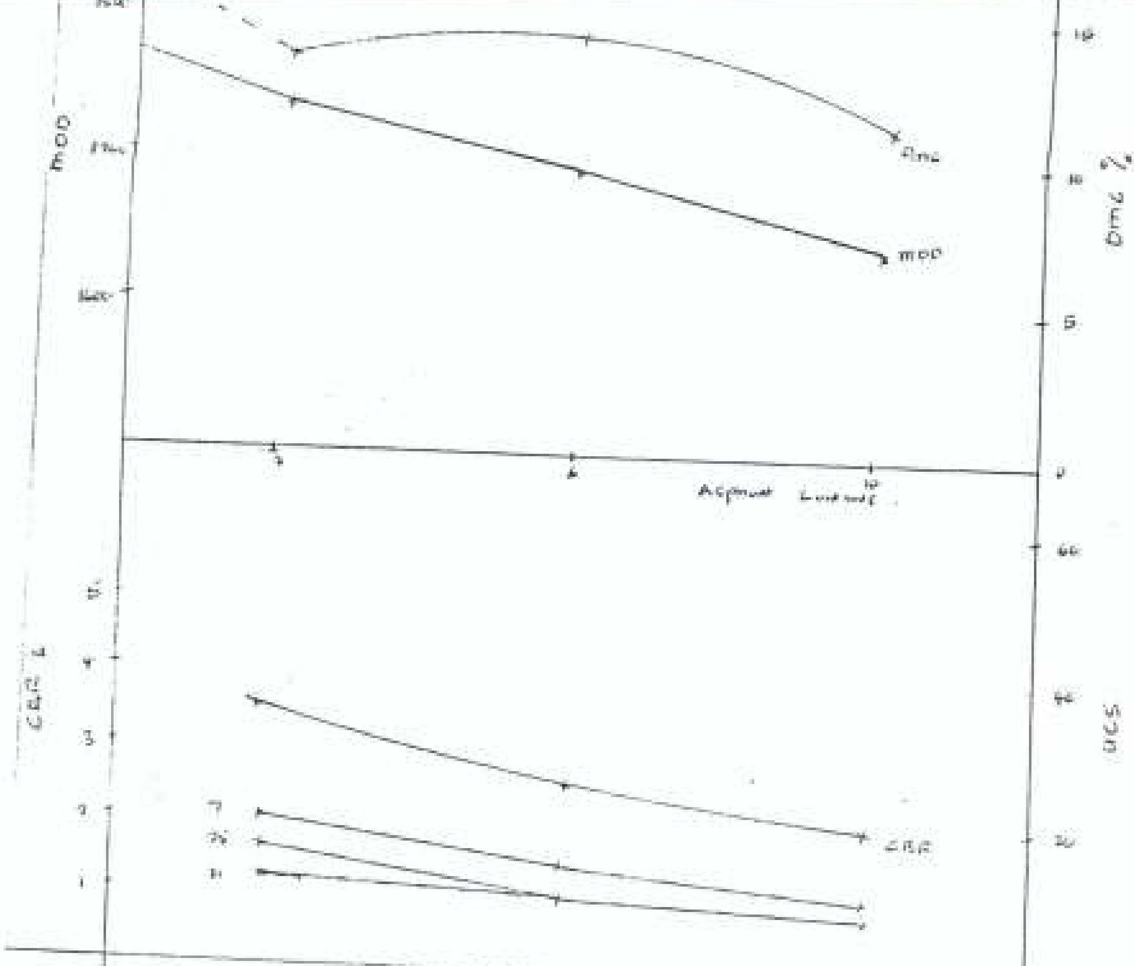


Fig 9. Variation of MDD, OMC, CBR and UCS with Asphalt Content (Sample C)

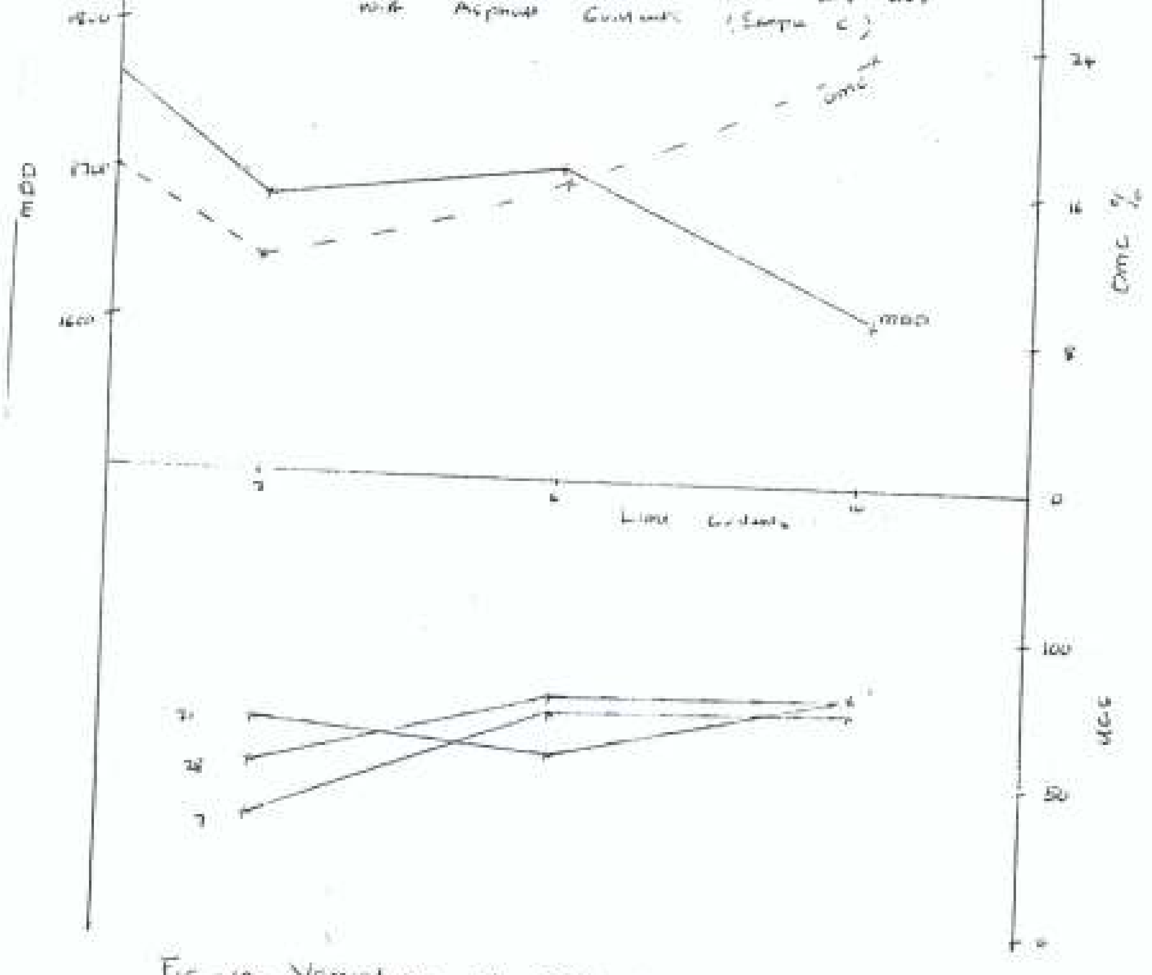


Fig 10. Variation of MDD, OMC, CBR and UCS with Lime Content (Sample C)

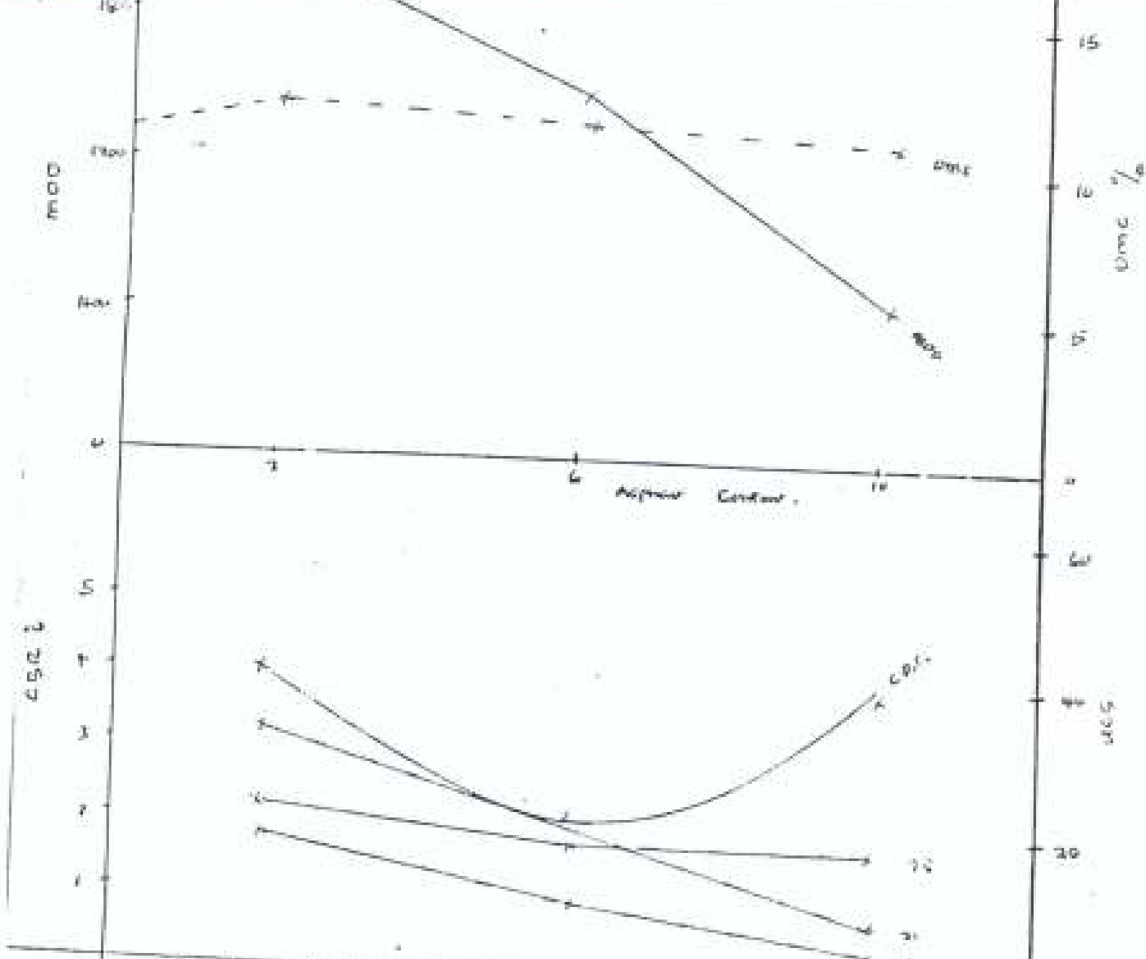


Fig. 11. Variation with Asphalt Content (Sample D)

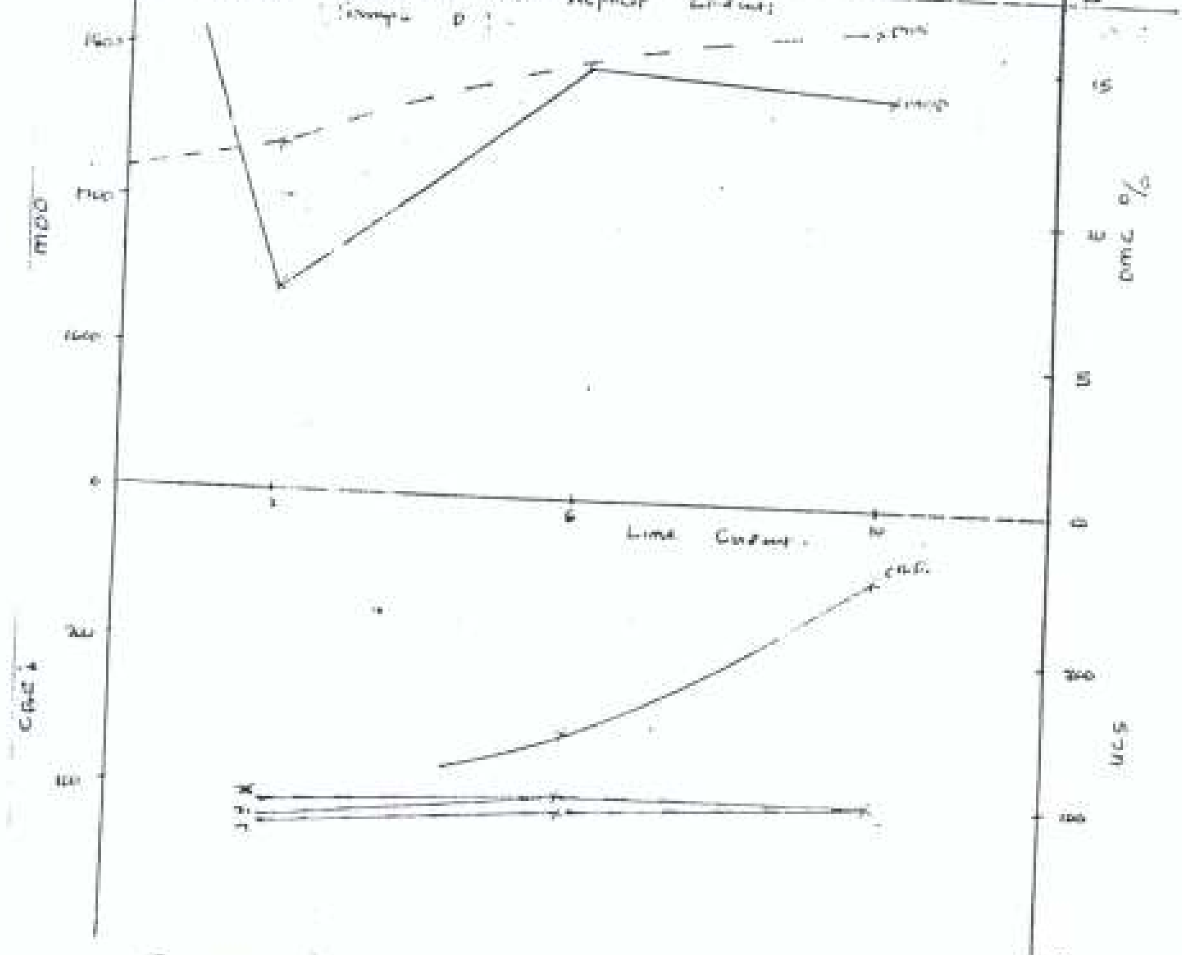


Fig. 12. Variation of MDD, OMC, CBR and UCS with Lime Content (Sample D)

4.2.4 Deformation Characteristics

The results of the Triaxial compressive strengths on the treated samples are presented in Tables 14 and 15. The percentages of lime and asphalt admixtures used, the days involved are the same for the previous tests.

Figures 13 and 14 are the stress - strain curves for the samples.

Stress - strain properties are essential for properly analysing the behaviour of a pavement structure containing a soil - lime mixture structural layer. The failure stress is increased and the ultimate strain is decreased for soil - lime mixtures to the natural soil. Soil - lime mixtures tested in triaxial compression are strain sensitive and the ultimate strain (for maximum compressive stress) is approximately 1 percent, regardless of the soil type or curing period (Thompson, 1966 b).

From Table 14, cohesion is greater when lime is used as stabilizer on fine grains. The cohesion however is greater when asphalt is used as stabilizer on sample D, being more granular. The more the cohesion the smaller the internal frictions both for the lime and asphalt. In Figures 13 and 14, stress increases with percentage increase of stabilizer; and curing generally increases the stresses.

Table 14. Results of Triaxial Compression.
Cohesion (c) KN/m^2

Sample		2 %			6 %			10 %		
		7	21	28	7	21	28	7	21	28
A	U.A	68	48	32	78	30	65	140	92	5
	U.L	120	-	-	80	-	-	80	240	250
	C.A	-	-	-	-	-	-	-	-	-
	C.L	117	60	-	82	130	158	125	205	130
C	U.A	55	33	73	-	-	-	-	-	-
	U.L	-	150	60	63	-	135	42	-	-
	C.A	-	-	-	-	-	-	-	-	-
	C.L	-	50	-	73	-	-	78	-	-
D	U.A	70	20	-	33	3	18	30	7	105
	U.L	35	-	-	57	-	-	200	-	-
	C.A	-	-	-	-	-	-	-	-	-
	C.L	-	-	-	60	-	-	45	-	-

Table 15. Results of Triaxial Compression.
Angle of Internal Friction (ϕ^o)

Type of Soil		2 %			6 %			10 %		
		7	21	28	7	21	28	7	21	28
A	U.A	14	18	18	9	18	9	-	10	27
	U.L	3	-	-	40	-	-	25	29	18
	C.A	-	-	-	-	-	-	-	-	-
	C.L	3	37	-	11	7	16	20	8	35
C	U.A	9	-	-	-	-	-	-	-	-
	U.L	-	10	40	30	-	34	32	-	-
	C.A	-	-	-	-	-	-	-	-	-
	C.L	-	29	-	12	-	-	90	-	-
D	U.A	14	23	-	9	9	-	9	9	-
	U.L	23	-	-	20	-	-	25	-	-
	C.A	-	-	-	-	-	-	-	-	-
	C.L	-	-	-	20	-	-	18	-	-

Symbols. L - Lime. C - Cured.
A - Asphalt. U - Uncured

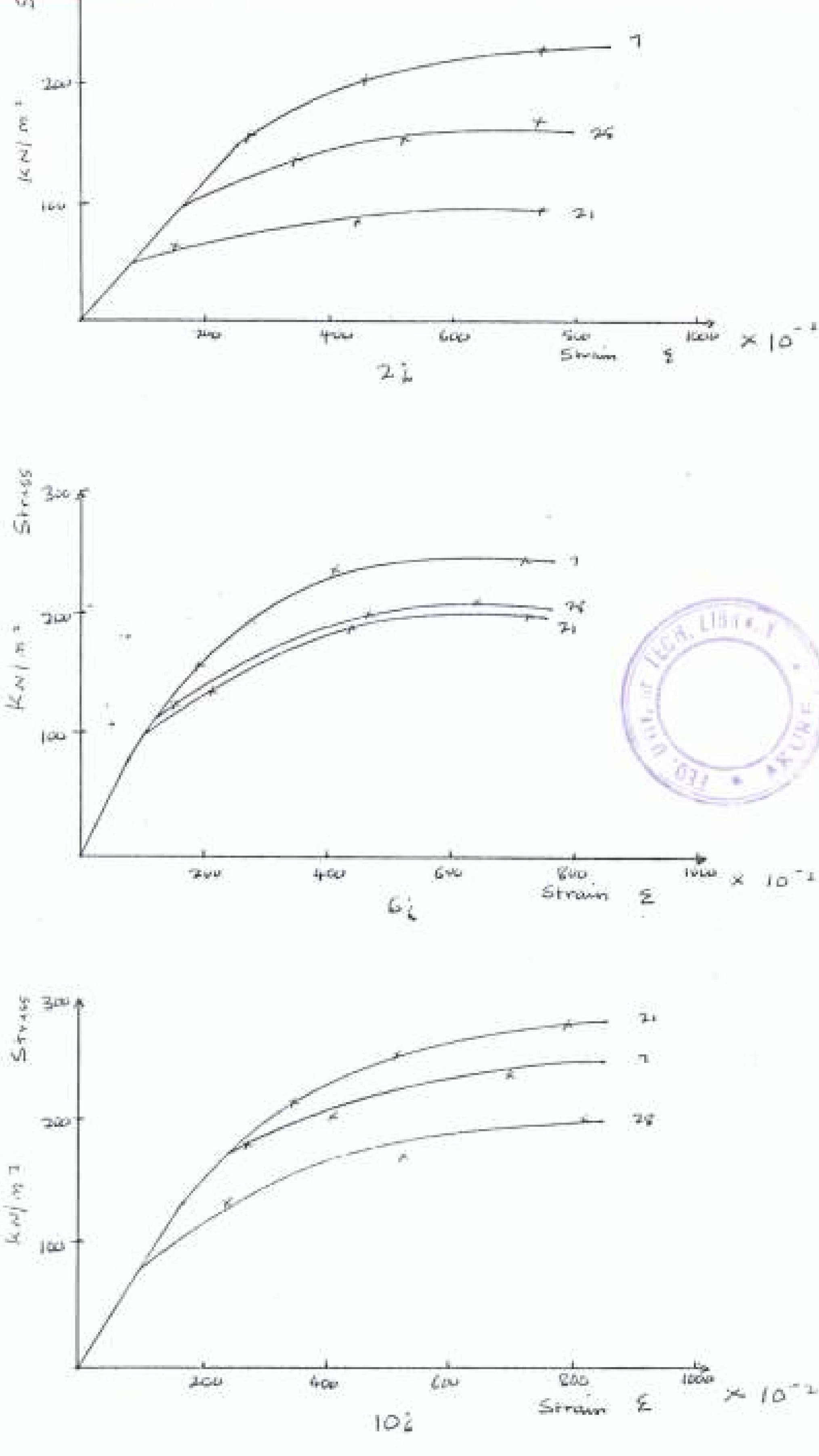
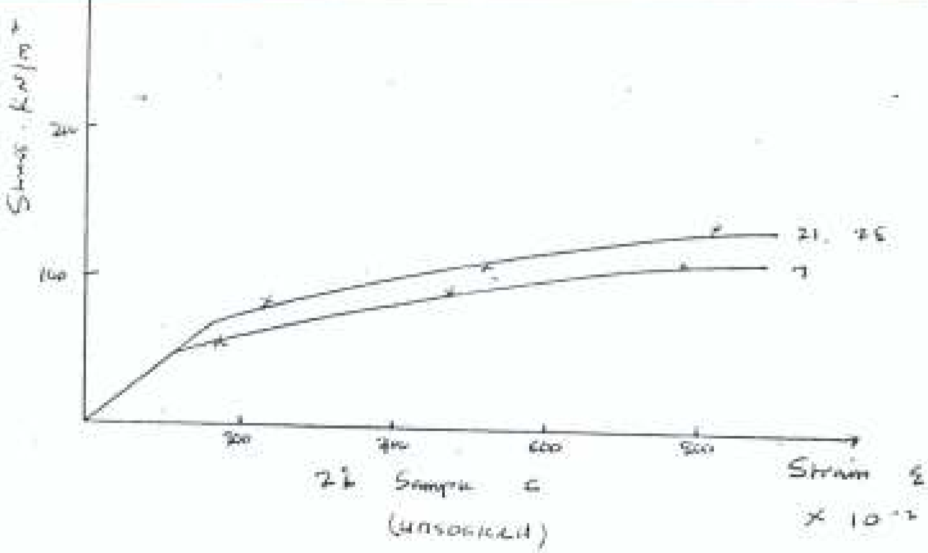
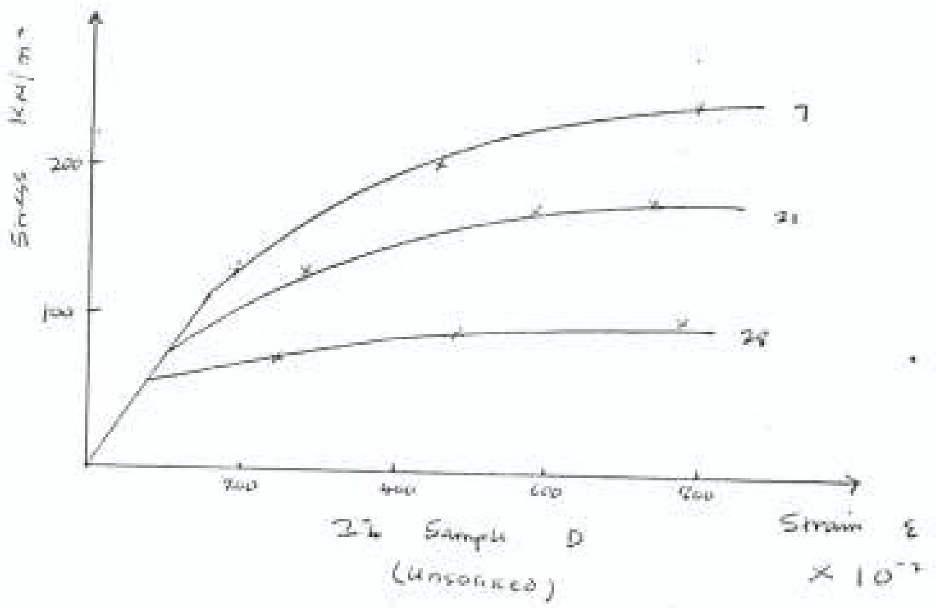


Fig. 13. Stress - Strain Curves of Sample A





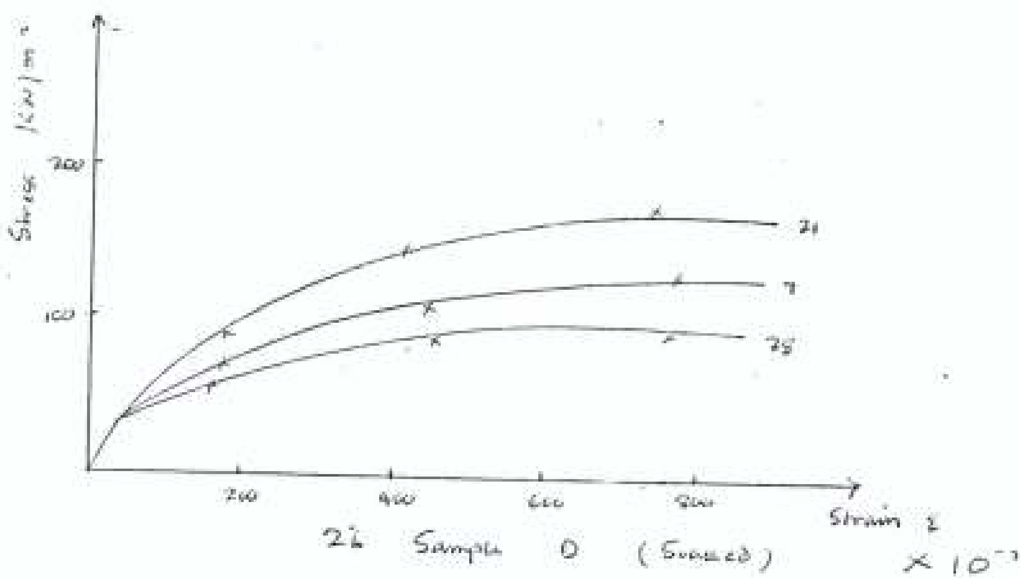


Fig. 14. Stress - Strain Curves of Samples C and D.

4.3 Observations on the Results

On the compacted characteristics of the treated samples, it was observed that soil - asphalt produced an higher dry density in most of the compacted samples than soil - lime. This might be as a result of the slowness with which soil - lime gains strength, which is an obstacle especially in cold climates (Baker, 1975).

In the use of cutback asphalt, Singh (1983) reported that maximum stability in dry density corresponds to between 4 - 6 percent content by weight of dry soil. In this study, it was observed that at around 6 percent addition of stabilizer (asphalt and lime) there is a drop in the dry density for Samples C and D but not for Sample A as shown in Table 11 and Figure 4.

Previous work had indicted that with some soil - lime mixtures, unconfined compressive strength continues to increase with time (Terrel, et.al, 1979). In this study, when samples were treated with lime, the more granular samples showed a significant increase in UCS more than the other samples. The UCS increased until around 6 percent stabilization and started to decrease in value showing that the optimum percentage has been reached. On the other hand, all the samples when treated with asphalt showed a reduction in their UCS values, this might be due to the fact that the type of pozzolanic cementing action is absent because asphalt only forms a waterproofing none cementing coat around the soil grains of the samples.

The addition of asphalt caused a negligible increase in CBR values of the treated samples, while the addition of lime caused a significant improvement in the CBR values of all the samples. This agrees with the works of Paquette and

McGee, (19). Maximum CBR values for the treated samples were observed at around 6 percent of stabilizer addition (Table 13).

Coyne, (1976) in the asphalt stabilization mix design reported at the Transportation Research Board Meeting in San Francisco California that small percentage (2-3 percent) of stabilizer will provide cohesion to sands and granular bases. This trend was affirmed by the results obtained,where it was observed that cohesion properties are generally better with lime than asphalt, especially with cohesive samples (A and C).

The addition of lime to samples A and C produced an increase in cohesion values with increased periods of curing and as the percentage admixture increases. When asphalt is used, there is a drop in value at 21 days and also as the percentage of stabilizer increases from 2 percent, especially on sample D (granular sample). The treated samples were mixed with a mechanical mixer,so mixing difficulty was eliminated.

The angle of internal friction is more with sample D when treated with asphalt. This trend continues and also as the number of days increases in both asphalt and lime mixture which confirms the work of Yoder and Witczak, (1975).

The effects of lime and asphalt on the compressive stress - strain properties of the samples are shown in Figures 13 - 14. Terrel et.al, (1979) states that the failure stress is increased and the ultimate strain is decreased for soil - lime mixtures to the natural soil. Similarly, it was observed in the stress - strain analysis obtained in this study that failure stress increases with percentage increase in the stabilizer and also that soaking increases the stresses on samples.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

From the results and findings of this work, the following conclusions could be drawn:

The stabilization of lateritic soil of Nigeria is possible by using lime and asphalt as stabilizer. This improvement on the lateritic soil is more significant when lime and not asphalt is used and the granular samples will perform better with the stabilizers because of the greater surface areas.

There is an optimum amount of stabilizer which gives a corresponding optimum strength and the point is 6 percent in this study for both lime and asphalt stabilizers. When the treated samples are cured generally there is a strength increase. On the whole, lime is a better stabilizer than asphalt for lateritic soils and this is better with granular materials, so gradation factor is very important in stabilization as well as the percentage stabilizer content and the type of exposure of samples.

5.2 RECOMMENDATIONS

This study is a cross - sectional study which has considered five samples A-E of lateritic soil sample in Ado-Ekiti of which only three, that is, A, C and D were fully analysed.

To make more generalized conclusion therefore one needs to examine samples from other areas and to cover wider and interior areas of the country.

There is room for expansion in order to confirm or contradict the findings of this study. But for the present situation and circumstances one can say the best as been presented.

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