CEMENT STABILIZATION USING HYDRATED LIME AS ADMIXTUBE

BY

DOUGLAS ODEMWINGIE AIYEVBEKPEN OSULA

A thesis submitted to the Postgraduate School, Ahmadu Bello University, Zaria, in partial fulfillment of the requirements for the degree of Master of Engineering

Department of civil Engineering
Faculty of Engineering
Ahmadu Bello University, Zaria

September, 1984

DEDICATION

TO MY MOTHER

DECLARATION

I hereby declare that this dissertation has been prepared by myself and that it is a record of my own research work. It has not been accepted in any previous application for a higher degree. All sources of information are specifically acknowledged by means of references.

DOUGLAS OSULA
SEPT. 1984

ACKNOWLEDGEMENT

I wish to express my sincere gratitude to all those who in their different ways helped in making this research work a success.

My special thanks go to Dr. Olusegun Adebisi and Dr. Nosa Legemah for their motivation and moral support. I cannot forget Mr. Joseph Ehigiamusoe who was always around to help with the collection of samples. I am very grateful to him.

I am also grateful to my supervisor, Dr. L.C. De Castro.

ABSTRACT

An investigation into the potentials of modifying problem laterite using hydrated lime was carried out. The possibility of stabilizing the soil with ordinary portland cement using hydrated lime as admixture was also investigated.

It was found that problem laterite classified as A-7-6 soil can be modified with 3% lime using the grain-size and plasticity criteria of the Highway Research Board (HRB) limits for soils that can be stabilized with portland cement. Results also show that using only the plasticity criteria, the soil can also be modified with 2% lime.

Further investigations show that the problem laterite cannot be economically stabilized using cement as a single stabilizer or with lime as an admixture using the usual 1.73N/mm^2 Unconfined Compressive Strength (UCS) and the Nigerian General Specification's (Roads and Bridges) 180% soaked California bearing ratio CBR values as evaluation criteria. This is due to the high clay content of the soil which stood at 44% unmodified and 22% modified. Lower values of UCS and CBR, 1.38 N/mm^2 (200lb/in^2) and 90% respectively are recommended as evaluation criteria for soil-lime-cement mixtures because of the better gain in strength with longer period, of the order of years, usually characteristic of stabilized soils containing lime.

TABLE OF CONTENTS

	PAGE
TITLE PAGE	i
DEDICATION	ii
DECLARATION	iii
ACKNOWLEDGEMENTS	iv
ABSTRACT	v
TABLE OF CONTENTS	vi
LIST OF TABLES	viii
LIST OF FIGURES	ix
ABBREVIATIONS AND SYMBOLS	xi
CHAPTER ONE: INTRODUCTION	1
1.1. Statement of problem and objectives	1
1.2. Scope	4
1.3 Location of sample and geology of area	6
CHAPTER TWO: LITERATURE REVIEW	10
2.1. Laterite soil	10
2.1.1 Description	10
2.1.2 Location and formation	13
2.1.3 Mode of occurrence in Nigeria	16
2.1.3.1 Ferruginous soils	17
2.1.3.2 Ferralitic soils	18
2.1.4. Classification of laterites	19
2.1.5 Problem laterites (as distinct from normal laterites)	22
2.1.5.1 Characteristics of problem laterite	23
2.1.5.2. Geotechnical identification of problem laterite	23

PAGE

2.2. Cement stabilization 27

2.2.1 Properties of cement-stabilized soils 29

2.2.2 Mechanism of reaction 30

2.2.3 Evaluation criteria 32

2.2.4 Factors affecting the strength and durability of cement-stabilized soils 34

2.2.5 The effect of lime as an admixture 39

2.2.5.1 Effect of lime on soil-properties 39

2.2.5.2 Mechanism of soil-lime reaction 41

2.2.5.3 Effect of lime on strength development 42

2.2.5.4 Mechanism of reaction in cement stabilization using lime as an admixture 46

CHAPTER THREE: LABORATORY INVESTIGATIONS AND TESTS 48

3.1 Tests 48

3.2 Preparation of samples and specimens 49

3.3 Compaction and curing procedure 53

3.4 Testing equipments 54

CHAPTER FOUR: TEST RESULTS AND DISCUSSIONS 56

4.1 Preliminary tests 56

4.2 Modification tests 63

4.3 Stabilization tests 71

CHAPTER FIVE: CONCLUSIONS 92

REFERENCES 96

LIST OF TABLES

		PAGE
TABLE "2.1	A form of geotechnical identification of problem laterite soils	25
TABLE 4. 1	Properties of soil before modification	57
TABLE 4. 2	Properties of soil after modification	68
TABLE 4. 3	Effect of lime content on Atterberg limits	69
TABLE 4. 4	Variation of OMC and MDD with lime and cement contents	70
TABLE 4. 5	Effect of cement content on UCS for soil-lime-cement mixtures	71
TABLE 4. 6	Effect of immersion in water on UCS for soil-lime-cement mixtures	71
TABLE 4.7	CBR values for soil-lime-cement mixtures	72

LIST OF FIGURES

		PAGE
Fig. 1.1	Geological Map of Nigeria	8
Fig. 1.2	Soil Map of Nigeria	9
Fig. 2.1	Schematic plan of engineering classification of laterites and lateritic soils	21
Fig. 4.1	Grain size distribution for soil-lime mixtures	59
Fig. 4.2	X-ray diffractograph for soil fraction passing B.S. No 200 sieve	60
Fig. 4.3	Differential thermogram for soil fraction passing B.S. No 200 sieve	61
Fig. 4.4	Variation of dry density with moisture content for soil-lime mixtures	62
Fig. 4.5	Variation of OMC, MDD with lime content for soil-lime mixtures	64
Fig. 4.6	Variation of LL, PL, Pi with lime content	67
Fig. 4.7	Variation of linear shrinkage with lime content	67
Fig. 4.8	Variation of Atterberg limits with time for soil-lime mixtures	68
Fig. 4.9	Variation of linear shrinkage with time for soil-lime mixtures	69
Fig. 4.10	Variation of shear stress with normal stress for soil-lime mixtures	73
Fig. 4.11	Stress-strain curve for soil-lime mixtures	74
Fig. 4.12	Variation of C and ϕ with lime for soil-lime mixtures	75
Fig. 4.13	Variation of E with lime for soil-lime mixtures	75
Fig. 4.14	Variation of dry density with moisture content for soil-cement (i.e. 0% lime)	77

PAGE

Fig. 4.15 Variation of dry density with moisture content for soil-lime-cement mixtures - 2% lime 78

Fig. 4.16 Variation of dry density with moisture content for soil-lime-cement mixtures - 3% lime 79

Fig. 4.17 Variation of OMC with cement content for soil-lime-cement mixtures 80

Fig. 4.18 Variation of MDD with cement content for soil-lime-cement mixtures 80

Fig. 4.19 Variation of CBR with cement content for soil-lime-cement mixtures. 84

Fig. 4.20 Variation of UCS (7 and 28 days) with cement content for soil-lime-cement mixtures 85

Fig. 4.21 Variation of UCS (7days) with CBR for soil-lime-cement mixtures 86

Fig. 4.22 Variation of UCS with MDD for soil-lime-cement mixtures 87

ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
B.S	British Standard
C	Cohesion
CBR	California bearing ratio
CC	Cement content
CEC	Cation exchange capacity
E	Modulus of elasticity
G_s	Specific gravity
HRB	Highway Research Board
LC	Lime content
LL	Liquid limit
LS	Linear shrinkage
MDD	Maximum dry density
OMC	Optimum moisture content
PCA	Portland Cement Association
pH	Hydrogen ion concentration
PI	Plasticity index
PL	Plastic limit
Ri	Resistance to loss of strength
TRRL	Transport and Road Research Laboratory (formerly Road Research Laboratory, RRL)
UCS	Unconfined compressive strength
ϕ	Angle of internal friction

kN/m^2	Kilonewtons per square metre
psi	Pounds per square inch
meq/100g	Millie-equivalent per 100 grammes
N/m^2	Newtons per square millimetre

CHAPTER ONE

1. INTRODUCTION

1.1 Statement of Problem and Objectives of Study

Soil-cement is essentially a low-cost pavement material, which is a shade better in quality than a naturally occurring suitable soil material for bases and subbases of roads designed to carry light to medium traffic. It is usually recommended for pavements which are designed to carry a cummulative standard axle not exceeding 1.5 million over the design life (TRRL Road Note 29, 1970). Millard (1960) has pointed out the need for soil stabilization in the tropics basing his argument on the premise of reduction in the frequency of pavement maintenance and the advantage of building a pavement strong enough to withstand the effect of traffic growth as well as the base or subbase so constructed providing a sound foundation for a future pavement that will be required to carry heavy traffic. Obi (1974) also highlighted the magnitude of savings that can be made in terms of money on the construction of a soil-cement as part of a pavement structure. Another need for stabilization is that most laterite soils in their natural state are at best suitable for subbase and not base construction (Arulanandan and Tunbridge, 1969). The use of soil-cement will, therefore, continue for a long time especially where new rural roads, which will carry light to medium traffic volume, will be

constructed to, among other things, provide intercity road transport and boost intercity trade and evacuation of foodcrops from the rural areas.

For areas with the problem of scarcity of economically stabilizable soil materials, there will be the need to import suitable materials. This process will obviously increase the initial investment on the road by way of increased costs of borrow and haulage. The Highway Research Board (1963) has set limits of 50% passing B.S. No 200 sieve, 40% liquid limit and 18% plasticity index to be met by soils that can be economically stabilized with Portland cement because of the difficulty in working the soils and the high proportions of cement required to stabilize the soils that do not meet the requirements.

Soils which do not meet this requirements can be found scattered all over Nigeria in small pockets and in some cases extending over large areas (Otteh-Iro and Partners, 1970-1982). Some laterites and the black cotton soils of Northeastern Nigeria belong to this group of economically unstabilizable soils. Laterites that belong to this group of soils have been dubbed "PROBLEM LATERITES" (Gidigasu, 1976) because of their reputation as very troublesome soils encountered in road construction.

For this problem laterite, the next reasonable alternative to running it to spoil and importing a stabilizable

soil is to use lime as an admixture in the process of cement stabilization. The purpose of lime is to modify the soil properties and render it workable for cement stabilization. Proportions of hydrated lime in the order of 2 to 3 percent have been recommended for soil modification (Ingles and Metcalf; 1972, Wesli, 1979). Appreciable improvements in soil properties (decreased plasticity and increased shear strength) had earlier been made on American Soils (Thompson, 1966) and confirmed later on Nigerian laterites (Ola, 1977b). Ola went further to recommend it for improvement of soils worse than those in the A-2 subgroups. (Problem laterite obviously belongs to the groups of soils worse than A-2 subgroups). Similar results were recently obtained from tests performed on synthesized kaolinite and montmorillonite soils from which 2 percent lime modification optimum was obtained for the kaolinite soil using C - ϕ criterion alone (Lees et al, 1982). Successes in field trials have also been made with the use of lime to aid pulverization and reclaim unstable subgrades (Perman, 1963; Sherban and Siembieda, 1972). Apart from direct modification, there is also the additional advantage of adding lime to a soil to counteract the retardation or prevention of proper hydration of cement caused by the presence of organic matter. This retardation which is due to the organic matter preferentially absorbing calcium ions of the cement hydration, is counteracted by the

supply of a ready source of calcium from hydrated lime to satisfy the absorptive capacity of the organic matter for calcium thereby rendering the soil stabilizable by permitting the calcium from the cement to complete its reaction with the other components in a normal manner (Maclean 1953; Sherwood and Covell, 1959).

The primary objective of this research project is to explore the potentials of cement stabilization using lime as an admixture and estimate lime and cement proportions for soil-lime-cement mixtures using a problem laterite. Hence the study is designed to determine the potentials of using hydrated lime to modify problem laterite and the possibility of stabilizing the soil with Portland cement using lime as an admixture.

1.2 Scope

The research investigations are carried out to determine the lime proportions to modify the problem laterite to attain a stabilizable state using the Highway Research Board (HRB) requirements and thereafter determine the Portland cement proportions to economically stabilize the lime-treated laterite. This is carried out against the background that the Nigerian General Specification for Roads and Bridges (1970) considers cement proportions above 7 percent of the soil by weight to be uneconomical. The particular form of problem laterite focussed on is that classified as A-7-6.

Modifications in the problem laterite are evaluated on the bases of grain size distribution and plasticity indices test results. The soil is modified with different amounts of hydrated lime. Stabilization is evaluated on the basis of results of tests conducted to determine the unconfined compressive strengths (UCS) and the California Bearing Ratio (CBR) of specimens made with different proportions of lime and cement. Of relevance to the evaluation is the requirement that CBR meet the 160% value specified in the Nigerian General Specification (Clause 6227). A value for the UCS is however not given in the Specification but the usual value of 1.73 N/mm^2 (250 lb/in^2) for soil-cement is adopted in the first instance.

Tests are performed in accordance with the following standards and specifications:

(a) B.S. 1377: 1975 "Method of Testing Soils for Civil Engineering Purposes."

(b) ASTM. STP 479: 1970 "Special Procedures for Testing soil and Rock for Engineering Purposes."

(c) B.S. 1 924: 1975 "Methods of Test for Stabilized Soils."

Modifications are made where necessary to reflect Nigerian conditions in accordance with the Nigerian General Specification for Roads and Bridges.

1.3 Location of Sample and Geology of Area

The soil samples have been collected at Agbede about 20Km. south of Auchi in Bendel State of Nigeria. Pavement failures are very noticeable features in this area along the Benin-Auchi-Okene Road. Some of the failures reflect on the asphalt surfacing in less severe cases in the form of alligator cracks and longitudinal cracks. Alligator cracks are caused by fatigue due to the swelling and contraction of the underlying base and subbase from alternate wetting and drying, this can only happen to a soil material of appreciable plasticity or clay content. Longitudinal cracks which could extend upto 10m, are caused by volume changes in the subgrade in the form of vertical movements of settlement, expansion and contraction. Severe cases of failure can be seen as shifting of the pavement and subsequent spalling and peeling of the surfacing due to plastic flow of the subbase and/or base in the presence of water and heavy axle loads transmitted through the wheels to the pavement. The soil, in this case, moves because it no longer has enough shear strength to withstand the traffic load.

The Agbede laterite, because of its clayey nature, finds use in burnt brick making in the area, a situation akin to when laterite was first discovered in 1807 by Buchanan in Malabar, India.

A study of the geological and soil maps of Nigeria which are shown in Figures 1.1 and 1.2 reveals that the soil belongs to the group of **ferrallitic tropical soils** derived from the cainozoic (tertiary) sedimentary rocks. According to D'Hoore (1964), ferrallitic soils are old, deep, highly weathered red soils of the humid tropics, strongly leached and possessing an oxic B horizon (a mixture of hydrated oxides of aluminium). These soils, whose clay mineral is the Kaolinite type, are highly deficient in weatherable mineral reserves because of the high intensity of weathering and leaching to which they have been subjected (Areola, 1982). This suggests that the soils have reached a most advanced stage of laterization. Halloysite is fairly common and to a lesser extent allophanes (Melomo, 1983).

The particular type of ferralitic soils to which the Agbede laterite belongs is the Benin sands of Western Nigeria (Ackroyd, 1967). Ackroyd described these soils more fully to be red and reddish brown, non-concretionary, gravel-free and well-graded acidic sand-clays. In addition to Kaolinite, traces of illite are to be found. Oxides are hematite geothite and traces of gibbsite.

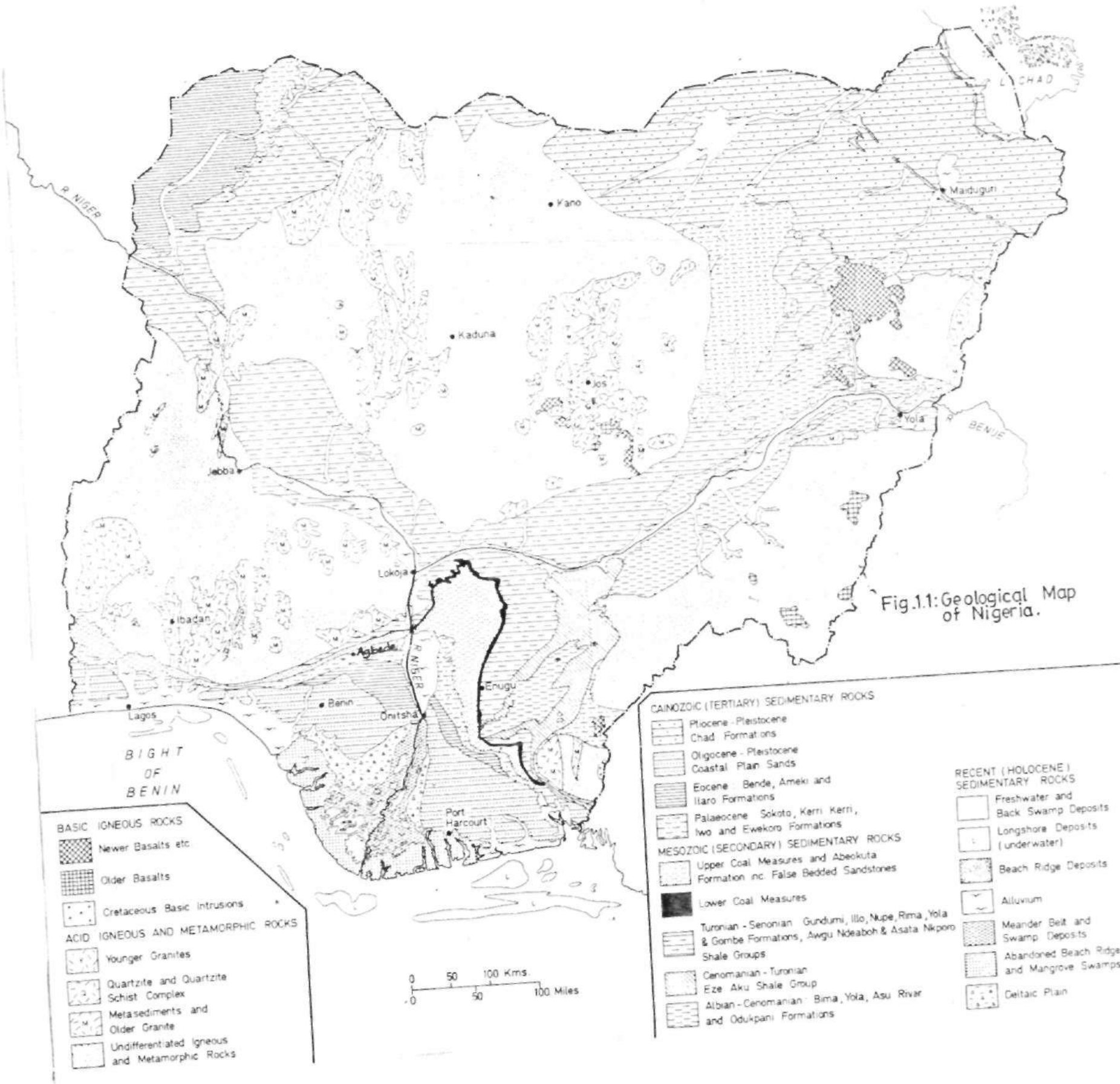


Fig.1.1: Geological Map of Nigeria.



Mainly ferruginous tropical soils

Ferralsols

Ferrisols

Weakly developed soils

Hydromorphic and alluvial soils

Fig.1.2: Soil Map of Nigeria.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Laterite Soil

2.1.1 Description

Laterite, since its discovery by Buchanan in 1807 in Malabar, India, has been given many definitions and descriptions. Buchanan originally described it as a ferruginious, vesicular, unstratified and porous material with yellow ochres due to high iron content. The Chamber's dictionary of Technology gives a rather pedogenic cum geologic description of it as a "a residual clay formed under tropical climatic conditions by the weathering of igneous rocks, usually a basic composition, consisting chiefly of hydroxides of iron and aluminum, grading through increase of the latter into bauxite." A recent and similar definition of it is given by Smith (1978) as a residual soil formed from limestones after the leaching out of soluble rock material by rain water to leave behind the insoluble hydroxides of iron and aluminum." Yet another definition for laterite was proposed by Little (1960) as "Igneous rock tropically weathered in-situ which has decomposed partially or totally with a concentration of iron or aluminium sesquioxides at the expense of silica."

Of rather academic and little practical significance is the categorization by Joachim and Kandiah (1941) into

laterite, lateritic and non-laterite soils from a knowledge of the silica-sesquioxide ratios SiO_2/ReO_3 ($Re = Al_2 + Fe_2O_3$). Ratios less than 1.33 indicate laterites, between 1.33 and 2.00 indicate lateritic soils and above 2.00 indicate non-laterites which have also been tropically weathered. Madu (1977), while agreeing with the residual nature of laterites, used this silica-sesquioxide ratio to divide Eastern Nigerian laterites into two main sub-genetic groups of sandstone laterites and lateritic shales and recorded low iron-oxide contents for lateritic shales and comparatively high contents in the sandstone laterites. He explained this phenomenon to be due to the methods of formation of laterites.

A rather critical description is that given by Rastall (1968). In his own words

> "It should first be mentioned that some of the red material so commonly used for road-surfacing in tropical lands under the name laterite is not really such at all, but simply iron-ore of various stages. True laterite is a red, yellowish or dark brown material with a curious cellular or vesicular structure. Often so strongly marked that in earlier days some of the darker varieties were mistaken for lava. One notable property is that underground it is often quite soft, and can be cut into blocks with a spade; it hardens rapidly on exposure to air and forms a good building material; hence the name (later-brick)."

Reviewing laterite definitions in his treatise on laterite soil, Gidigasu (1976) grouped the various definitions according to the soils hardening properties, chemistry and

morphology. All in all, there however seems to be a wide acceptance of the all encompassing definition due to Alexander and Cady (1962) which states that "laterite is a highly weathered material, rich in secondary oxides of iron, aluminium or both. It is really void of bases and primary silicates, but it may contain large amounts of quartz and kaolinite. It is either hard or capable of hardening on exposure to wetting and drying."

From the ongoing definitions and descriptions and according to Maignien (1966) it is clear that the origin of laterite cannot be due to a single process. Several phenomena are involved, not all operating at the same level of differentiation.

Some of these and most other definitions ignore the hardening property of laterite as well as suggest that laterite can only be formed from igneous rock or limestones. There is also an inference, latent in some definitions but vividly revealed by Bastall (1968) that there are some laterite-like materials which actually are not laterites. With recourse to this and in order to remove the confusion characteristic of having too many definitions, Lohnes and Demirel (1973) attempted to separate the true laterites from soils with mineralogic characteristic similar to laterite but neither hard nor possessing the capacity to harden due simply to alternate periods of wetting and drying which they termed "lateritic soils". They accepted the

definition of Alexander and Cady for laterite with the modification that hardness means there is sufficient induration of the soil that it cannot be readily excavated by a shovel or spade.

2.1.2 Location and formation of laterite

Laterites are widely distributed throughout the world but especially in the intertropical regions of Central and South America, West and Central Africa, India, Malaya, the East Indies and Northern Australia (Maignien 1966).

Of more interest, however, to the Civil Engineer is the presentation of Woodtorton (1975) that laterites are most commonly found in leached soils and are formed by the laterization process of weathering systems, the two main characteristics of which are (1) the decomposition of ferro-alumino silicate minerals and the permanent deposition of sesquioxides to form a laterite horizon within the profile and (2) the leaching, in the tropics, of silica from the soil by an effectively alkaline soil solution.

The first formation process is a normal chemical decomposition unassociated with leaching while the second is basically physico-chemical process, the physical aspect akin to the erosion of erodible materials while the chemical aspect is due to the alkaline nature of the soil solution.

Part of the silica dissociated in the process of leaching may react, in the presence of the alkaline solution, with the

sesquioxide to form concretionary laterite. The remaining silica may be completely removed by drainage if there is no impediment or form secondary clay minerals. "If this leaching of silica is minimal or does not take place, as in the formation of the soils generally referred to as podsols, then the process of laterization may be considered to occur under climates other than tropical" (Woodtorton, 1975).

These two processes of laterite formation form two of the three stages of laterization identified by Maignien (1966) and Gidigasu (1976) namely - decomposition of primary minerals and release of constituent materials; the leaching, under appropriate drainage conditions, of silica and bases and the relative accumulation or enrichment from outside sources of sesquioxides; and partial dehydration or desiccation (sometimes involving hardening) of the sesquioxide-rich materials and secondary minerals.

According to Ole (1978 b), the decomposition under temperate conditions of low chemical and soil-forming activity does not continue past the clay-forming stage unlike in the tropics where, due to high temperature and rainfall conditions, the first stage of laterization continues into the formation of various forms of aluminous and iron oxides from the clay minerals in relation to the nature of the weathering condition. Lohnes and Demirel (1973) have explained this weathering process to include the transformation of feldspars to kaolinite

and an indicate to gradibisite accompanied by formation of iron oxides from iron-rich primary minerals. The hardness, according to them, is the result of loss of kaolinite, great enrichment of oxides, dehydration and crystallization of iron oxide minerals into continuous aggregates or networks.

The factors affecting the formation of laterite are:

1. a basic or intermediate parent rock material rich in iron in the form of ferro-alumina silicates.;
2. vegetation - amount and kind and the associated organic matter and humic residue;
3. a permeable profile to permit leaching;
4. climate - heavy rainfall (preferably of a monsoon type) high humidity, high temperature and fluctuating water table;
5. topography - flatish and of savanna type;
6. a combination of alternate leaching by an effectively acid and effectively alkaline soil solution (silica is removed by an alkaline medium and the sesquioxides are mobilized by an acid medium).

The many forms of lateritic materials found in the various parts of the world are as a result of the differing variations of the applicability of these factors, as caused by the involvement of different pavement materials and differing external and internal climates (Woolterton, 1975).

While laterites form in the tropics as a result of the above mentioned factors, Little (1969) reported that most of these laterite formation factors also apply to many temperate sites yet laterization does not take place. In his explanation of this, he opined that the bacterial metabolism necessary to produce laterization effects can only be sufficiently stimulated in the high temperatures of the tropics.

Laterites exist in relatively thin layers on the parent rocks and in some cases in deep layers. Laterite occurring in considerable thickness is the result of laterization of the soil to its full extent, followed by effective dessication in proper sequence and extent to harden the materials somewhat uniformly (Persons, 1970).

2.1.3 Mode of Occurrence of Laterite in Nigeria

Laterites occur in Nigeria in the form of ferruginous and ferrallitic soils. (D'Hoore 1964, Ackroyd 1967). These two soils are distinguishable on the basis of physical and engineering properties (Lyons Associates 1971).

There are however a less developed type of soil known as ferrisols (D'Hoore, 1964). They developed on the savana zone (Areola, 1982). These are transitional soils resembling the ferrallitic soils but with less well developed profiles, a higher nutrient status, a better structure and higher biological activity (Maignian 1966). Annual rainfall

in their places of occurrence is low and this causes reduced leaching and profile development. Lateritic iron caps are to be found developed on the tops of flat-topped ridges in their location of occurrence in Nigeria (Areola, 1962).

According to D'Hoore (1964) ferrisols must be regarded as very similar to ferrallitic soils but have properties between ferrallitic and ferruginous soils. The silica-alumina ratio is close to 2.0. It has a clay fraction made up almost entirely of kaolinite, free iron oxides and, amorphous gels, sometimes with small quantities of 2/1 clays and gibbsite.

2.1.3.1 Ferruginous soils

As can be seen from Figure 1.2 these soil cover the greatest part of the country, occuring in the subhumid to dry parts of Middle and Northern Nigeria.

They are derived mainly from the Basement complex and old sedimentary rocks. According to Ackroyd (1967) "under subhumid climatic conditions with annual rain less than 1500mm - 2000mm, iron is mobilized in solution, transferred from a horizon of gravel accumulation and immobilized in the form of ferruginous concretion, leaving a more plastic clay complex."

As described by D'Hoore (1964), they are soils with A/C profile with some having an A_2 horizon and a textural B ni horizon showing nuciform or weakly prismatic structure. Marked separation of free oxides are frequent in the profile. They may either be leached out or precipitated in the form of spots or concretions in the profile. There is often an

appreciable reserve of weatherable minerals. The silt/clay ratio (20/2 microns) is generally above 0.15. They contain kaolinite clay minerals, residual quartz in addition to the free iron oxide. Small quantities of illite may however be present (Ackroyd, 1967). The presence of allophane was recently recorded by Ola (1980) on a Kaduna laterite. The SiO_2/R_2O_3 ratio is always less than 2 while the SiO_2/l_2O_3 is near 2, signifying that aluminuis minerals are less abundant.

They are inactive as a result of the nature of clay minerals present and are less prone to thermal instability but are likely to be mechanically unstable (Malomo 1983). Areola (1982) is, however, of the opinion that the high kaolinite content and the presence of free oxides which aid particle aggregation give these soils a strong stable structure. The contrast here may be that of instability due to remoulding as against stability of an undisturbed soil from the state of occurrence of the constitutents in the profile.

High CBR values, low Atterberg limits and high densities are characteristics of the ferruginous soil (Lyons Associates (1 971). These are explained on dehydration and aggregation of the particles (Malomo 1983).

2.1.3.2 Ferrallitic soils

Ferrallitic soils, which are non-concretionary, are formed under conditions of greater humidity and heavy leaching in areas of good drainage (Ackroyd 1967). They are also

called aluminous or bauxitic laterite because of the high content of alumina (Maignien 1966).

They possess an oxic B horizon (a mixture of hydrated oxides of aluminium) and are highly defficient in weatherable minerals because of the intense leaching to which they have been subjected (D'Hoore, 1964).

The soils are deep profiled and have only slightly differentiated horizons. The silt/clay ratio is generally less than 0.25 in horizons B and C while the SiO_2/Al_2O_3 ratio is sometimes near 2 but usually below. The clay minerals which are the 1:1 lattice type (Kaolinite) are mostly associated with large quantities of iron minerals (D'Hoore 1964). Halloysite and allophane (to a lesser extent) are fairly common with large quantities of hydrated oxides of iron and/or aluminium and sometimes silica are always present. This renders the soil thermally unstable (Malomo 1983).

2.1.b Classification of Laterite

Marked differences in the behaviour of laterites from temperate zone soils have been reported by many investigators. Reasons adduced for this have been those of the index tests for classification not being readily reproducible for laterites as well as their differing and unique mode of formation and the environment under which they exist (Little, 1969; De Graft-Johnson and Bhatia 1969; Lohnes and Demirel 1973; De Graft-Johnson, 1975; Ola 1978b, Ola 1978a). Accordingly, suggestions have been

made to base classification of laterites on systems other than those used for temperate zone soils. Prominent among these proposed systems are those based on mode of formation, parent material and degree of weathering (De Graft-Johnson et al 1969 ; Little 1969 ; Gidigasu 1971 ; Lohnes and Demirel 1973); those based on void ratio and specific gravity as a reflection of the iron content (Lohnes and Demirel 1973, Tuner and Lohnes 1977; Madu, 1977); and those based on the much talked about silica-sesquioxide ratio due to Joachim and Kondich (1941) which has been used for the classification of Eastern Nigeria laterites (Madu 1977). Ackroyd (1963) had much earlier proposed a method based on mechanical strength of the gravel fraction of the soil, while Correia (1967) proposed a classification based on determining the chemical composition of the gravel and clay fraction of laterite. Lohnes and Demirel also suggested a system based on pedology but quickly dismissed it on the grounds that laterite soils represent a narrow portion of the continuum of fesidual soils. Their plan for engineering classification of laterites and lateritic soils is as shown in Fig 2.1. In the plan from the left, the parent rock in the form of solid is weathered to form a clayey soil with increasing amounts of kaolinite and sesquioxides accompanied by increasing void ratio and decreasing cohesion reflected as strength. As clays become more abundant the cohesion gradually begins to increase. This increase in cohesion is further enhanced by cementation from precipitation of

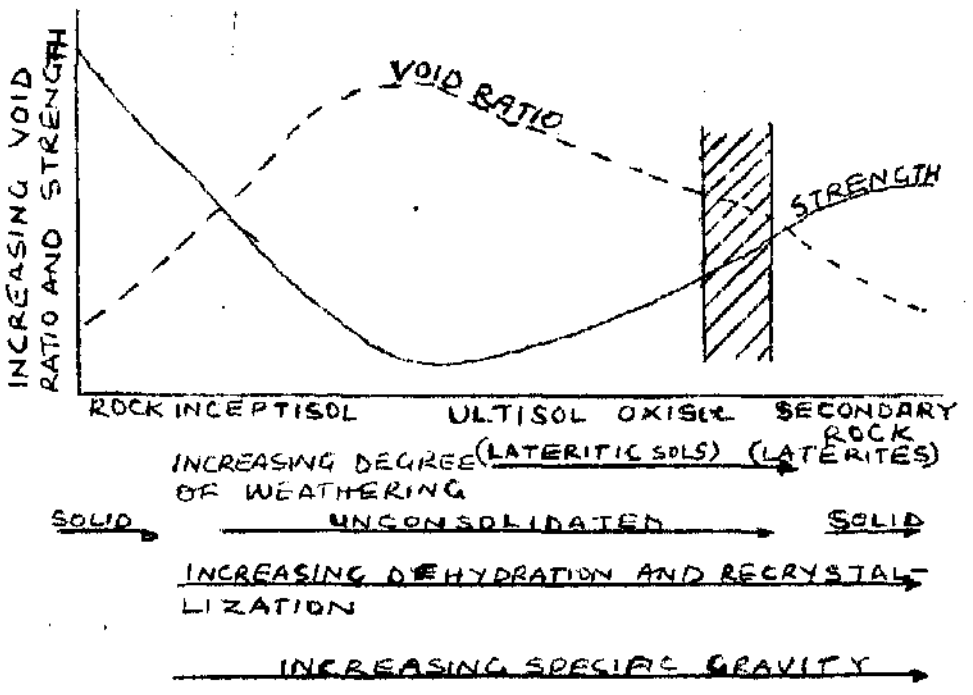


Fig. 2.1: Schematic plan of engineering classification of Laterites and lateritic soils (After Lohnes and Demirel 1973).

sesquioxides. This process tends to cause a reversal in the void ratio curve as the voids become filled. Finally, after considerable cohesion is regained, the voids are nearly filled, a laterite is formed.

There is, as yet, no consensus on the classification system to adopt for laterites because of the fact that some of the properties proposed for classification, e.g. silica-sesquioxide ratio and degree of weathering, do not lend themselves to easy determination on construction sites. Recourse is, therefore, still being made to the use of the

existing classification systems such as the HRB and the Unified Soil Classification Systems.

Under the HRB classification system, Nigerian laterites have usually been falling into all the subgroups excepting the A-3 subgroup (Etteh-Aro and Partners 1970-82, Ola 1978b) to which belongs pure sand of quartz. In the Unified Classification System, these laterites will range from organic clays of high plasticity, OH, to well-graded gravel and gravel-sand mixture, GW, which makes an excellent pavement material.

2.4.5 Problem laterite (as distinct from normal laterite)

Problem laterites are laterite soils which have the reputation of being problematic in road construction. In highway and airfield flexible pavements with these soils as subbases and bases, they are easily noticeable by presence and magnitude of pavement swelling, plastic flows and weakness in the presence of water and heavy wheel loads. These plastic flows usually occur at the edges of the pavement where the soil bulges out due to ingress of water. Weakness which usually forms on one lane (the lane used by heavily and laden commercial vehicles) appear as high-amplitude and broad-crested corrugations parallel to the longitudinal profile of the road, the crests and troughs appearing across the road width. The valleys are usually on the paths of the wheels. More are formed when vehicles try to avoid existing ones.

The problem of non-reproducibility of standard classification tests and the difficulty in evolving satisfactory identification and classification procedures for laterite soils are attributable mainly to the existence of "problem laterite" (Gidigasu 1976). This has been explained on the effect pretest treatments and time have on the properties so identified (Mohr 1977). These pretest preparations and handling disrupt the natural structure of the soils (Little 1969), Holmes et al. 1971; Moh and Mazhar 1969).

Problem laterites have been generally identified as residually weathered soils formed in regions of recent volcanic activity and/or continuous wet climate with an average rainfall generally above 1500mm (60ins) and are characterized by high natural water contents and liquid limits, low natural densities, and friable and/or crumble structures (Millard 1962; Terzaghi 1958). The friable nature of the soils is probably due to the presence of sesquoxides, which as free iron oxides adsorbed on the clay mineral surface or coating the pore wall and/or filling the voids, bind the soil particles to form tiny spherical or spongy aggregations (Alexander and Cady, 1962).

2.1.5.2 Geotechnical Identification of Problem Laterite

Many attempts have been made to classify or identify all laterite soils on the same basis as used for temperate

zone soils. While some successes have been recorded with some laterites, misleading and erroneous results have been obtained for some (Terzaghi 1958; Little 1969). These misleading results have probably been from problem laterites.

Failures on some Nigerian roads are no doubt connected with such misleading results which have led to misjudgement of the soil behaviour and properties. In such cases, there may have been pavement failures where soils met classification requirement for road construction specifications. There could also have been the reverse case in which materials, which may have been sensitive to remoulding, not meeting specification requirements are rejected whereas such materials could have performed well in a pavement. There is therefore the need to base classification and identification criteria on results of tests performed on undisturbed samples as implied in the work of Lohnes and Demirel (1973) on Puerto Rican Laterites. The need for the development of testing procedures related to behaviour in the field and improvement in the execution and control of construction work has been also expressed by De Graft-Johnson (1975).

Gidigasu (1973) proposed a criterion for separating problem laterite soils from the non-problem ones on the basis of a review of previous studies (Table 2.1). Novais-Ferreira (1965) had earlier proposed a criterion for differentiating what he called soft laterites from the hard one more suitable for road construction on the basis of a granulometric modulus

TABLE 2.1 A form for geotechnical identification of problem laterite soils*

Geotechnical Properties			Probable engineering behaviour and field performance				
Laboratory tests	Average range of value		Sensitivity to drying	Structure sensitivity to remoulding	Potential self-stabilization	Field compaction problems	Degradation under traffic and during compaction
	Unfavourable	Favourable					
AASHO. Impact value(%)	> 50	< 30					✓
Water absorption (%)	> 6	< 4					✓
Natural moisture content (%)	LL	below PL	✓	✓		✓	
Linear shrinkage (%)	> 6	< 4				✓	
Potential swell (%)	> 11	< 10			✓		
Predominant clay mineral	Montmorillonite hydrate halloysite	Kaolinite chlorite	✓	✓		✓	
Mica content (%)	>10	10				✓	
Aggregation Index	> 2	≈ 1	✓	✓			

*After Gidigasu, 1973.

(M_c) and a hardness index of the concretions (I_{HC}). He defined M_C as the sum of the percentage of material passing the 1in, ¾in, ½in, 3/8in, No. 4, No. 10, No. 40, and No. 200 ASTM sieves. I_{LC} was defined by the expression

$$I_{LC} = \frac{M_c(\text{natural State})}{M_c(\text{after standardized treatment}} \quad 1$$

The M_C depends on fineness, the smaller the particle, size the larger the M_C.

De Graft-Johnson (1975) attributed the inapplicability of standard tests procedure to the identification of some lateritic soils as below;

1. Different degrees of weathering;
2. The coating of the fine particles by sesquioxides;
3. Production of pseudo coarser fractions from the cementation of the silt and clay particles;
4. Iron oxide induration of the materials so cemented;
5. Various degrees of the cemented materials by the iron oxides;
6. The unique clay mineralogy of the soils especially the content of the "unstable" hydrated halloysite.

The contributions of these factors to difficulties of evaluating some laterites for engineering purpose are very well reviewed by Gidigasu (1974).

2.2 Cement Stabilization

According to Odier et al (1960) the roles of stabilization in the economic realization of the process of road construction are threefold:-

(1) the realization of short term economics by way of reduced construction costs resulting from the use of local materials of mediocre quality instead of importing better and suitable materials;

(2) stabilization makes it possible, by permitting the use of borrow pits of a heterogeneous nature, in certain cases to adopt a single method on long road sections, involving the setting up of important plant, expediting the works and ultimately resulting in savings in construction costs;

(3) stabilization is unavoidable where there is scarcity of materials usable by traditional methods or where prohibitive maintenance and repair costs will be incurred from the use of materials of mediocre quality.

Having decided to stabilize the soil, the task of choosing a method then sets in. In the same report, Odier et al classified soil stabilization into three catergories - mechanical, chemical and physico-chemical.

More recent literature have tended to completely separate chemical from physical stabilization. One of such is the treatise on stabilized earth roads due to Kedzi (1979). In this text,

the three categories of stabilisation, mechanical, chemical and physical were defined as follows:

1. Mechanical stabilization is the process in which the soil stability is ensured without the addition of any foreign material. This is done by compaction only (it is to be noted that compaction is an integral part of every form of stabilization) of the insitu soil or imported soil, drainage and maintenance of the water content at a constant level, and admixture of different soil types;

2. Physical stabilization is the process in which soil stability is made possible by temperature change (freezing and heating), hydration, evaporation and adsorption;

3. Chemical stabilization is the process which uses the chemical reactions of ion-exchange, precipitation polymerization and oxidation. Cement stabilization which is the main focus of the present work obviously belongs to this class of stabilization.

The factors which affect the choice of any of these methods are:

1. type of material;
2. climatic condition and;
3. the availability of different stabilizers corresponding to each type of stabilization (Odier et al, 1960).

Reddi (1979) further states that the material (soil) type factor includes properties of the soil in natural condition, properties expected from the stabilized soil and effects on the stabilized soil after stabilization.

2.2.1 Properties of cement-stabilized soils

A well prepared and cured soil-cement specimen or pavement has the good qualities of strength and durability.

The strength is normally measured by way of the unconfined compressive strength test and the california bearing ratio test. These increase lincarly with cement content as is the case with portland cement concrete. The strength build-up is due to interparticle bonding as well as density or close packing of the compacted soil-cement.

Like in concrete, cement has the effect of high initial strength gain for soil-cement for the first seven days followed by a reduced rate of strength build-up with time till around the twenty-eighth day when strength starts to level up. This is due to the formation of the cementing compound, tobermiorite gel, at two different stages in the cement hydration process initially from tricalcium silicate (for seventh day strength) and dicalcuim silicate (for twenth-eight day strength).

The effect of cement content on the dry density of gravelly clays is to decrease it initially with increasing cement content followed by a rise after attaining a minimum density with further increase in cement content.

According to Maclean (1956), a successfully stabilized pavement will have a closely-spaced overall crack pattern enabling it to act as flexible pavement with high mechanical stability as a result of particle interlock at the fracture

surfaces. Pavements that do not behave like this are those that usually have too high an amount of cement and consequently high 7-day UCS values of the order of 5.52 N/mm^2(800 lB/in^2) and above. The net results are shrinkage stresses giving rise to less regularly spaced cracks across the pavement and are reflected on the bituminous surfacing. This is due to the pavement acting as a rigid pavement or a concrete slab because of its high strength (Metcalf, 1966). This is to be avoided in design considerations. Metcalf (1966), in supporting Maclean's idea of an adequately stabilized pavement, therefore suggested designing and considering a stabilized pavement to be completely cracked and the strength limited with this in mind. If this happens, the cracks will be due to load rather than resistance of the pavement to cracking. These cracks (usually characterized by close pattern) together with the shrinkage stresses will be distributed over a large number so that the movement at any point is rarely sufficient to overstrain a bituminous surfacing and the seal or wearing course remain intact.

2.2.2 Mechanism of reaction

Of the few proposed mechanism of reaction of soil-cement, that due to Kedzi (1979) seems to be the more complete and convicing. Ingles and Metcalf (1972) had earlier proposed a reaction mechanism in which the hydration of cement causes water to react with tricalcium silicate and dicalcium silicate components of the cement "in the presence of the soil" to form gels of mono- and di-calcium silicate hydrate as well as free lime as summed up in the equations below:

$$C_3S + 2H = C_2SH + CH$$

$$C_2S + 2H = CSH + CH$$

where $C = CaO$, $S = SiO_2$ and $H = H_2O$. The insoluble calcium silicate gels crystallize very slowly into an interlocking matrix. A close look at these equations reveals that the reactions only take place in the "presence of the soil" and does not involve the soil. This is only true for lean-mix concrete which is borderline between concrete and soil-cement.

The more recent Kedzi's explanation of this soil-cement reaction mechanism identifies two stages; the primary stage (Ingles and Metcalf's reactions form only a part of this) in which the primary cementing substances are formed; and the secondary stage in which secondary cementing substances are formed.

The primary cementing products are responsible for the high initial strength gains (7 and 28 days) while the secondary cementing products, which are basically products of the reactions of soil and cement components with a product of the primary reaction, are responsible for the continued gain in strength with time. This secondary reaction is of a pozzolanic nature between the free lime product of the primary reactions and the soil silica and alumina of the clay fraction and the hitherto unused components of cements.

Since the components of portland cement are C_3S (tricalcium silicate), C_2S (dicalcium silicate), C_3A (tricalcium

alumuinate), C_4AF (tetracalcium alumino ferrite), the reactions are as below

$$2\ C_3S + 6H = C_3S_2H_3 \text{ (tobermorite gel)} + 3CH(\text{lime})$$

$$2C_2S + 4H = C_3S_2H_3 \text{ (tobermoritegel)} + CH \text{ (lime)}$$

$$C_4AF + 10H + 2CH = C_6AFH_{12} \text{ (Calcium alumin ferrite hydrate)}$$

$$C_3A + 12H + CH = C_4AH_{13} \text{ (tetracaclium aluminate hydrate)}$$

where the additional F and A mean FeO_2 and Al_2O_3. The gypsum of the cement ($C_aSO_4 \cdot 2\ H_2O$) reacts with water and tricalcium aluminate to form caclium monosulfo-aluminate.

The tobermorite from the first reaction forms and crystallizes at an early stage and is responsible for the 7th-day strength. That from the second reaction forms and crystallizes at much later date and is responsible for the 28th-day strength. Both form the primary reaction stage. The products of the next two reactions are weakly cementing compounds. In soils with clay, the pozzolanic reaction reinforces these latter reactions to form the secondary stage which is responsible for the continued gain in strength after 28 days.

2.2.3 Evaluation Criteria

A stabilized material is accepted usually on the basis of either UCS or CBR tests. Some specifications specify the use of triaxial compression tests. The anomaly in the use of the UCS and CBR as a basis for acceptability of

soil-cement mixture and other stabilization methods has been pointed out by Metcalf (1966). The anomaly is that sands having an acceptable CBR value would have an unacceptable UCS value and vice versa for a clay soil. This is confirmed also by the conservative estimates of cement content that have been recorded for clayey laterites by some investigators (Ola 1977b, 1978b) and black cotton clay (Ola, 1978a). The Nigerian General Specification for Roads and Bridges uses the CBR criterion for determining cement content while UCS is used for field control with the apparent belief that CBR values conservatively estimates strength.

The durability of soil-cement is assessed using the PCA wetting and drying tests, the swell (CBR) tests the weight loss test and the compressive strength of oven dried samples after 12 cycles. The British standard considers the PCA durability tests too harsh and specified only immersion in water for the UCS specimens. It, infact, considers durability requirements satisfied if the specified UCS values are attained after the required curing period which is usually 7 days. No value of resistance to loss in strength measured in the immersion test is specified. The Portland cement Association (PCA) specifies a value of 2% as the maximum allowable swell, 7% maximum allowable weight loss, and 2% maximum allowable volume change (wet and dry test). Ola (1978) is rightly of the opinion that the PCA freeze-thaw test is not applicable to tropical conditions.

For the test for loss in strength of UCS specimens soaked for four days after curing for seven days, a value of 20% is the maximum allowable loss in strength.

The usual value of 7-day UCS is 1.73N/mm^2 (250Ib/in^2). The British (Department of Transport) Specification for Road and Bridge works (1970), however, specifies a value of 2.8N/mm^2 (400Ib/mm^2) and 3.5N/mm^2 (500Ib/in^2) for cylindrical and cubical specimens respectively. These values are very high, hence a material that satisfies this UCS criterion will obviously satisfy the durability criterion.

The CBR value specified by the Nigerian General Specification is 180% to be attained in the laboratory for materials designed to be built by the mix-in-place method or 160% for a material to be built by the stationary-plant-mix and travel-mix methods.

2.2.4: Factors affecting the strength and durability of Cement-stabilized soils

A number of factors affect the strength and durability of soil-cement. As enumerated by Road Research Laboratory (1952) and Lambe (1962), these are soil type, amount and type of cement, method and extent of mixing, amount of compaction, moisture, age, curing, presence of admixtures and temperature.

Soil Type:- Of all the factors enumerated above, soil type is the most important. According to the PCA, almost all soils that can be pulverized can be cement-stabilized. However,

because of the difficulty in pulverizing some heavy clays, the HRB set limits for soils that can be economically stabilized with cement as below:

Particle size distribution Limits		B.S. Plasticity tests limits	
Maximum size	3in.	Maximum Liquid Limit	40%
Passing 3/16 sieve	50%	Maximum Plasticity Index	18%
Passing No. 36 B.S. sieve	15%		
Passing No 200 B.S. sieve	50%		

Many soils exist that do not meet these limits. Notable among them in Nigeria are black cotton clays and problem laterites. Most of these soils contain plenty of clay materials and because of the high specific surface associated with the clay content they require excessive amounts of cement for stabilization. Clays like montmorillonite to which the black cotton clay belongs are definitely unsuitable for cement stabilization. Experments on the black cotton clays of Norther Eastern Nigeria showed that these soils cannot be effectively stabilized even with 8% cement for which a value of $1.0N/mm^2$ was recorded for the UCS at 7 day (Ola, 1978a). An A-7-6 laterite was however effectively stabilized with 5% cement (Ola, 1978b). For this same soil, 7% cement gave a UCS value of $2.21N/mm^2$. According to the Nigerian General Specification for Roads and Bridges, a cement content greater than 7% is uneconomical.

Since cation exchange reaction is not unlikely for soil-cement, the exchangable ions that the soil carries will

also influence the response of the soil to cement treatment. The manner in which the other listed factors affect the strength and durability of soil cement are presented below, based mainly on the information published by Road Research Laboratory (1952) and Lambe (1962) in "Soil Mechanics for Road Engineers" and "Soil Stabilization" in Foundation Engineering (ed. Leonards G.A.) respectively.

Amount and type of cement

Although anytype of cement can be used for cement stabilization, Portland cement is most commonly used. High-early strength cement is usually more effective than ordinary portland cement. In general, the higher the cement content, the higher the strength and the more durable the soil-cement. Any soil that does not exhibit such a trend is not suitable for cement stabilization.

Moisture content

Moisture affects the soil-cement mixture in two ways. It provides a medium for cement hydration and it facilitates compaction. There is no information about the best moisture content for compaction but it is generally agreed that the amount of water determined by moisture-density relationship is enough for hydration as well as for compaction. Experiments have however shown that there is a tendency for strengths to increase with moisture content for a given dry density. The best moisture content should depend on the soil type and method of compaction.

Compaction

As with all forms of stabilization, the properties of soil-cement depend largely on the compacted density as strength is due partly to density and partly to interparticle bonding.

Depending on the type of soil, UCS may increase with dry density at the same cement content.

Mixing

A thoroughly carried out mixing operation will ensure high strength and durability because of the even dispersion of cement within the soil mass. The intimacy of mixing is not necessarily a linear function of the mixing energy. However, prolonged mixing is detrimental to strength development as mixing might be carried past optimum resulting in the segregation of already bound components. Mixing beyond cement hydration also reduces strength. This is why specifications set limits for mixing time. The Nigerian General Specification specifies that mixing should not exceed two hours or that compaction should be carried out within two hours of mixing. The B.S. 1924 specifies that mixing in the laboratory should not exceed 20 minutes.

In the field, strength depends on the type of plant used for mixing. Mix-in-place plants generally give lower strengths than efficient rotary tillers. This is why the Nigerian General Specification specifies a higher value of CBR (180%) to be met for mixtures to be processed in the field by the mix-in-place

method than the 160% required for the stationary-plant mix.

Age and Curing

The manner in which curing period influences the resulting product is same as with concrete; strength increases with age. For curing to be effective, it must be carried out in a moist environment or a situation created where water will not evaporate from the compacted specimen. A common way of doing this in the laboratory is by coating the specimen with wax. In the field, curing is effected by spraying a priming coat onto the surface of the compacted soil-cement base or subbase.

Admixtures

Many works have been done on the effect of admixtures on strength improvement for soil-cement. One of the pioneering works in this field is that due to Lambe et al (1960). They found that the efficacy of admixtures depend on the amount of reactive silicate in the soil. From their various works dramatic improvements in strength were recorded especially with sodium Chloride (NaCL) and Sodium Sulphate ($N_a SO_4$). Ola (1978a) recorded improvement in the strengths of cement-stabilized black-cotton clays with lime as the admixture. Though improvements were high in terms of percentage, strengths were still generally low. A mixture with 6% lime and 8% cement gave 1.2MN/mm^2 (180 lb/in^2). The durability of the specimens was however found to be adequate.

Lime in its raw form is abundant in Nigeria as is evidenced by the siting of numerous cement production factories in several locations in the country. Ola (1977a) has expressed the need for the production of building lime in the country. Using laboratory prepared building lime from local lime for soil stabilization and soil property improvements, Ola recorded comparable results with those obtained using imported building lime. It is with the anticipation of this production of building lime that lime has been chosen as an admixture in this present work.

2.2.5 The Effect of Lime as an admixture

The effect of lime as an admixture on the final product of stabilization is to be seen or better appreciated in the light of its improvement on soil properties as well as strength development of the stabilized soil.

2.2.5.1 Effect of lime on soil properties

The earliest reports about attempts to study the properties of lime-treated soils could be those of Aaron (1948) and Maclean (1953). Further works however followed by such investigators as Lund and Ramsey (1959), Hilt and Davidson (1960), Mateos (1964), Wang et al (1963), Fosseberg (1965), Thompson (1965, 1966), De Wet and Skinner (1967), and very lately by Sabry and Parcher (1979).

Generally as presented by Lambe (1962), Igles and Metcalf (1972), Yoder and Witzack (1975) and Kedzi (1979), the

addition of lime to a soil has the following effects:-

1. reduction in plasticity of highly plastic soils and increase in plasticity of low-plasticity soils;
2. reduction in maximum dry density and corresponding increase in optimum moisture content;
3. increase in grain size and reduction in clay fraction;
4. reduction in swell potential; and
5. increase in permeability and hence drainability.

The extent to which these improvements occur will depend on soil type, lime content and type, water content, temperature and time. Better results are to be obtained for soils with appreciable amount of clay as well as for clays containing high activity clay minerals (Ingles and Metcalf, 1972).

Of the two types of lime, quicklime and hydrated lime, quicklime gives better results. But is less preferred for use because of its hazardous effect on the skin.

Water, according to De Wet and Skinner (1967), has the role of a transporting agent for the dissolved Ca^{++} and OH^{-} ions. Sabry and Barber (1979) put this more clearly that high water content fosters two helpful phenomena of efficient calcium diffusion and production of a high pH environment around most of the clay particles. This diffusion reduces the possibility of local lime concentration as well as provides most of the sample with uniform amounts of Ca^{++} and OH^{-} ions. Potable water, and not acid or mineral water, is all that is required for good property improvements.

The benefits derivable from the improved soil properties, according to Lambe (1957), are reduction in the amount of agent required for subsequent stabilization of the improved soil and the rendering usable of soils which could not otherwise be stabilized with such agents as cement and bitumen. Another benefit is the reduction of crack formation the soil clay content having been reduced by the addition of lime (Ingles and Metcalf, 1972). On a more practical note, there will also be a reduction in the tendency of picking-up under rollers during compaction (Maclean, 1953), probably due to the more friable nature of the soil after modification. The direct implication of the second benefit is the increase in the range of usable soils for road construction.

2.2.5.2 Mechanism of reaction in lime-soil modification

Some of the few proposals that have been made on the reaction mechanism of lime-treated soils are those of Lambe (1962), Diamond and Kinter (1965), Thompson (1966) and De Vet and Skinner (1967).

There seems to be a general consensus that when lime is added to a soil, it dissociates in the presence of water into Ca^{++} and OH^{-} ions thereby increasing the soil pH. The excess Ca^{++} ions supplied in this process replaces dissimilar cations from the exchange complex of the soil or is adsorbed by the soil in order to satisfy the unbalanced negative charges on the particle surface. This is cation exchange reaction.

According to Gillict (1968) the extent and rate at which it happens depends on

1. the clay particle size;
2. the adsorbed atom in the double layer;
3. ionic diffusion velocity;
4. temperature;
5. concentration and permeability of natural clays.

Following cation exchange are flocculation and agglomeration. Flocculation may be as a result of cation exchange or may more likely be due to the overcrowding of the excess calcium ions on the adsorbed surface of the clay particle thereby causing the lowering of the zeta potential or the repelling power of like ions in the medium (O'Flaherty, 1974). Agglomeration is caused by the attraction of the remaining soil particles still carrying negative charges to the positively charged surface of the cation exchange affected particles or the flocculated particles.

The above explanation which is entirely due to cation exchange, does not include the action of the cementing gel material product of pozzolanic reaction as claimed by some authors. Pozzolanic reaction which is responsible for strength gain in lime stabiliztion may only be of any benefit after compaction.

2.2.5.3 Effect of Lime on strength development

Various other admixtures have been used in cement stabilization. Promising results have been reported especially

for sodium chloride and sodium sulphate which recorded very high strength gains (Lambe 1957, Lambe et al 1960). Admixtures have been used in the main to make possible the effective stabilization of otherwise unstabilizable soils. They have also been used to aid pulverization and hence mixing with cement for very clayey soils. For stabilizable soils, they have served to produce higher strengths at reduced cement contents.

Very limited information is to be found in literature on soil-lime-cement mixtures. While only mere mention was made about the premixing of lime to highly plastic soils to facilitate pulverization of the soil before admixing with cement as a common practice in Britain (O'Flaherty, 1974), marked increases in strengths were obtained from laboratory investigations of soil-lime-cement mixtures using black-cotton clay (Ola, 1979b).

Lambe (1957) showed that very marginal increase in strength was recorded with the use of hydrated lime for the soil tested. However, much higher strength gains may be possible by using laterite because of its existence in a high temperature region and hence high strength level (Lohnes and Demiriel, 1973), since the rate of strength gain of a stabilized soil, particularly that stabilized with lime, rises with increasing temperature (Kedzi 1979). An increase in temperature is known to affect stabilization, especially with lime, by increasing the velocity of the reactions

(De Wet and Skinner 1967). Vallerga et al (1969) have in fact shown that for some laterites, mixtures of cement and lime in predetermined proportions can give a substantially better gain in strength than the use of single stabilizers.

In cement and lime stabilizations, the main contributions of the stabilizing agents are rapid initial strength gain for cement, and better continued strength gain with time for lime (Clare, 1960). It is, therefore, envisaged that, apart from the modifications potentials of lime on otherwise unsuitable materials, the addition of more lime to the free lime product of cement hydration will further enhance the continued and increased strength gain of the soil-lime-cement mixture with time.

The temperature differential between tropical and temperate zone soils is a very significant feature which has been proved to be a contributory factor to the much reduced cement proportions needed for the stabilization of laterite as against a typical temperate zone soil (Ola, 1974) for which the Highway Research Board limits were set. It is therefore envisaged that the utilisation of the combined property improvement of lime and cement on soils and existence of laterite at a high temperature will yield combined lime and cement proportions to produce an inexpensive pavement by way of considerably reduced cement proportions. There will also be the added advantage of speedier construction operation with the improved soil. A compromise will therefore be struck

between providing a pavement of adequate strength and a road structure of minimum initial construction costs and reduced maintenance costs. The initial construction cost factor however can only be gainfully tapped if production of lime is started in Nigeria since the cost of the available imported lime is at present more than three times its Portland cement equivalent by weight. The need for and potentials of lime production in Nigeria has been expressed by Ola (1977a).

The properties of a soil-lime-cement mixture should be similar to those of soil-cement. The difference may be the effect of lime addition. Two things are likely to happen with lime addition and these are:

1. Strength may be higher than in ordinary soil-cement but the relationship between cement content and UCS or CBR will remain linear. This has actually been confirmed on black cotton clay (Ola, 1978a);
2. Strength may initially be higher than in ordinary soil cement, but may reduce after reaching a peak with further addition of cement i.e. situation is now similar to lime stabilization.

The extent to which either of these happens will depend on the type and amount of clay present in the soil. The former is expected for montmonillonite soil because of their high activity (high cation exchange capacity). The latter event may happen for kaolinite soils because of their low activity and reactivity with lime. Works by Ola (1978b) on lime stabilization of laterite

and by Lees et al (1982) on laboratory synthesized kaolinite soil confirmed this trend of initial increase in strength followed by a fall after which strength levels up. No attempt was made by any of the investigators to study the effect of lime on strength development of the kaolinite soils stabilized with cement. All things being equal, initial higher strengths are expected for the reaons given above.

2.2.5.4 Mechanism of reaction in cement stabilization using lime as an admixture.

The addition of cement to soil-lime mixture may result in chemical as well as physico-chemical reactions between the reaction products of soil-lime and cement, and between the soil and cement. Better absolute gain in strength is therefore expected with soil-lime-cement than with soil-cement.

This gain in strength may be due to the continuation of the secondary reaction of cement stabilization (see section 2.3.2) which is of a pozzolanic nature now accentuated by the admixed lime or a tertiary reaction of a completely different nature.

A reaction of the latter type is yet unknown but if it exists, the new cementing compounds must be strong and of slowly or rapidly developing nature. According to Gilliot and Sereda (1966) the structure of the newly formed hydrated lime from cement hydration is in a state of strain which might render it unusually reactive. The admixed lime

together with this reactive lime from cement hydration will increase the reaction with the silica and alumina of the soil as well as with those from cement hydration to form more of the cementitious compounds. There is therefore the likelihood of epitaxy as suggested by Gilliot (1968), which is the extension of an already exisiting crystal structure by orientied overgrowth. In this case the crystal structure may be the cementitious component formed from the reaction of lime with the silica and alumina of the cement and the overgrowth being due to the additional lime causing more of the reactions. According to Gilliot (1968) this is expected to favour strength development.

Another form of reaction that can be expected could be that of the combined action of lime flocculation of clay particle from the ion exchange reaction and the strong and rapid action of Portland cement through the tobermorite gel and other products of cement hydration, acting simultaneously but the lime's modification action causing flocculation starting first because of the rapidity of its action. In this case the cementation of the larger flocculated and aggregated particles form a stronger matrix from the cementing action and the expected increased density due to the addition of cement of heavier weight than lime particles. This is a clear case of interparticle bonding and density acting together to build up strength.

CHAPTER THREE

3. LABORATORY INVESTIGATIONS AND TESTS

3.1. Tests Performed

Experiments were carried out in three stages as follows:

(a) Preliminary Tests:- These tests were performed for the purpose of identifying and classifying the soil. These tests were performed on air-dried samples. They included grain size distribution (mechanical and sedimentation sedimentation analyses), atterberg limits, X-ray diffraction analysis and differential thermal analysis (DTA). Tests were performed in accordance with B.S 1377: 1967 and ASTM-STP 479:1970.

(b) Modification Tests:- These were carried out in order to determine lime modification optimum and the moulding water requirements for the soil-lime mixtures. Tests performed were Atterberg limits, grain size distribution (mechanical and sedimentation analyses) and moisture-density relationship for varying proportions of hydrated lime starting with 1% by weight of the dry soil till the lime content X% (X = lime modification optimum) that gave the properties meeting the requirements of the Highway Research Board (HRB) was obtained. The tests were performed in accordance with ASTM-STP 479. Further tests were performed to investigate the stress-strain, and cohesive and frictional characteristics of the soil (natural and modified) by way of direct shear tests.

Lohnes et al (1971) have found that the direct shear test is very useful for the evaluation of the shear strength characteristics of laterite clay soils. These tests were performed on air-dried samples for both natural and modified soils.

(c) Stabilization Tests:- The purpose of these tests was to investigate the strength performance of the soil-lime-cement mixtures for 0%, (X-1)% and X% lime and varying proportions of Portland cement. The (X-1)% lime content was included in order to have a somewhat continuous basis of comparison of results between 0% and X% lime contents. The CBR and UCS were determined for samples compacted at optimum moisture contents for soil-lime-cement mixtures. Soaked and unsoaked tests were performed for the UCS specimens while only soaked tests (as required by the Nigerian General Specification for Roads and Bridges) were performed for the CBR specimens. The tests were performed in accordance with B.S. 1924; 1975. Modifications of these tests were made along the guidelines given in the Nigerian General Specification.

3.2 Preparation of Samples and Specimens

The Agbede laterite soil samples were collected in sacs at about 100m. from the Benin-Auchi-Okene Road. Samples were collected at a depth of between 1.4m. and 1.8m corresponding to the B-horizon usually characterized by accummulation of materials leached from the overlying A-horizon.

The natural soil was air-dried for a period of five days in an average room temperature of 32°C. Oven drying was not possible because of the amount of soil involved. However, air-drying in the tropics is a reasonable alternative to oven drying (oven drying is normally carried out at an oven temperature of not more than 50°C for a relatively short period) as long as the soil is properly spread to a thin, layer on a dry platform and the drying carried out over a long period. Clods were crushed with rubbercovered pestle and the soil sieved through a 2.36mm B.S. sieve.

Preliminary classification tests as well as tests to determine the C.C and MDD were performed on the soil in accordance with B.S. 1377:1975. Sodium Hexametaphosphate was used as the dispersant in the sedimentation (pipette) analysis. Sodium hexametaphosphate has been proved to be a better dispersant than sodium oxalate for soils suspected to have appreciable clay content since the method of dispersion used in sedimentation analysis affects the silt/clay ratio of laterites (Madu, 1977). The clay mineralogy of the soil was determined for material passing the B.S No. 200 sieve by way of X-ray diffraction analysis supplemented by differential thermal analysis (DTA).

Hydrated lime was prepared by the slaking of quicklime because of the unavailability of hydrated lime in the market. The quicklime used was the imported Buxton brand which is

readily available in the market but rather expensive. The slaking was carried out by adding 50% water by dry weight of the dry quicklime which came in klinkers. Mixing was thoroughly carried out until all the klinkers turned into powder. A lot of heat was evolved during the mixing. The resulting hydrated lime powder was stored in an airtight container and left to cool for 24 hours before use. Each quantity of hydrated lime so prepared remained in the airtight container till exhausted for the experiments in order to avoid carbonation which may result from exposure to the air. The quantity of hydrated lime just sufficient for one day's operation was usually prepared.

For the modification experiments, precalculated amounts of lime were mixed dry with corresponding amounts of the soil. Water was later added and the mixing continued till no dry patch was noticed. The mixing was done manually in a sample tray with a bricklayer's trowel. The resulting soil-lime mixtures were covered and left to mellow for the required periods in hours' from 1 hour to 7 hours at intervals of 2 hours) before the tests were performed on them. Tests to determine the grain size distribution and the cohesion and angle of internal friction were performed after a time of 7 hours because of the need to reduce time that would have been required for the magnitude and number of tests especially for the sedimentation analysis which usually requires several hours to complete.

For the stabilization experiments on soil-cement samples, the measured soil sample was mixed dry with the precalculated

amount of portland cement to a uniform colour and the required amount of water (determined from moisture-density relationship for the soil-cement mixtures) added and mixing continued manually, but quickly, till no dry patch was noticeable. The specimens were compacted within 20 minutes of mixing as specified in B.S. 1925:1975.

For the stabilization experiments using soil-lime-cement, the precalculated amount of cement was added to the dry mixture of soil-lime and mixed thoroughly to achieve a uniform colour. The required amount of water (determined from moisture-density relationship for soil-lime-cement mixtures) was later added and the mixing continued. The various specimens used for UCS and CBR tests were compacted within 20 minutes of mixing. Mixing in all cases was assumed to start at time water was added.

While the effect of time on the modification was measured through the determination of Atterberg limits at different time intervals after mixing, it was not considered necessary to do the same for the stabilization experiments as it will be impracticable on site operations (will take too much of contractor's time to construct or rather, site production will be too slow leading to unnecessary delays and may leave the contractor at the mercy of unpredictable inclement weather) to wait for long hours for modification reactions to reach desired level. Lumbe et al (1960) have infact proved

that there may be no need to have a reaction time if stabilization is the ultimate goal. In their experiments on the effect of alkali metals compounds as admixtures on soil-cement, it was discovered that best results could be achieved by the simultaneous addition of the compound and the cement to the soil. This simultaneous addition of lime and cement to the soil was given credence to by the observed immediate increase in grain size of the soil after mixing with lime and water (in the modification experiments which preceded the stabilization experiments) and the much better workability achieved with soil-lime-cement compared with soil-cement in the trials.

3.3 Compaction and Curing Procedure

The standard proctor compactive energy was used for the UCS and CBR specimens in accordance with the Nigerian General Specification which specifies this for stabilized soils.

The UCS specimens were compacted in three layers in a 101.6 x 101.6 x 101.6mm (4 x 4 x 4 ins) mould, each layer receiving 28 blows while the CBR specimens were compacted in three layers in a CBR mould each layer receiving 62 blows. The UCS specimens after demoulding, were coated with wax while the CBR specimens, still in the moulds, had their tops covered with wax. Specimens, after preparation in the above manner, were left to harden for the required curing periods.

For each cement content, 8 UCS specimens and 2 CBR specimens were prepared. The 2 CBR specimens were cured for six days, the wax removed after the sixth day, and soaked, in water for 24 hours before testing. This is as specified in the Nigerian General Specification. 2 each of the UCS specimens were tested unsoaked after seven, fourteen and twenty-eight days curing in wax coating - a total of 6 cubes. The remaining 2 specimens were stripped of the wax coating after seven days and soaked in water for a further seven days before testing. The latter soaking procedure for the UCS specimens (in accordance with B.S. 1924; 1975) was adopted in preference to the six-day curing and one-day soaking recommended by the Nigerian General Specifications because the seven-day soaking period is considered a more realistic period over which loss in strength or resistance to loss in strength can be measured as well as being a suitable alternative (British Practice) to the American wetting and drying durability test. According to Ingles and Metcalf (1972) durability tests are only appropriate where moisture penetration is likely to occur. The water table in the project area is not known to be close to the ground surface. Moreover, the one-day soaking procedure serves only to give a basis for field control as specified in the General specification.

3.4 Testing Equipments

The testing of the CBR specimens was done on a Wykeham Farrance CBR machine. The tests were performed at a

standard rate of penetration of 1.27mm (0.050 in) per minute with a 5.0kg surcharge weight simulating a 75mm. asphalt surfacing on top of the stabilized base.

The UCS specimens were tested on an ELE (Engineering Laboratory Equipment Ltd) Mark XII 2000 KN Digital Compression Machine Model EE 32-350. The cubes were crushed at a loading rate of 1.7KN/mm^2 ($250lb/in^2$).

The DTA for the soil mineral identification was performed on an Eberbach DTA Machine of Weston Model 301-57. The material was heated to a max temperature of 840°C.

The X-ray diffraction analysis was performed on the Phillips Automatic X-ray Powder Diffractometer, Model PW 1390 using the Cu K (α) radiation at a gonimmeter speed of 1.0°/min.

The shear tests were performed on an ELE Shear Box Machine, model 1090-01-0187 using a 60mm square box at a constant rate of strain of 1.06mm/min.

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1 PRELIMINARY TESTS

Identification of soils: The results for the identification of soils are presented in Table 4.1. The grain size distribution of the original soil is as shown in Figure 4.1. It is seen that the soil is classified under the A-7-6 subgroup (AASHTO Classification system). It is a reddish-brown well-graded fine-grained soil with inorganic clay of high plasticity. Its value as a road foundation material is poor to very poor (CP. 2003: 1959, Earthworks). The clay content is about 44% (See Fig 4.1).

Clay Mineralogy:- The results of the X-ray diffraction and differential thermal analyses on the fraction of the soil passing the B.S. No. 200 sieve are shown in Figures 4.2 and 4.3 respectively. Halloysite is confirmed by the 7.25Å reflections being weaker than the 3.39 Å reflections and the banded appearance of the peaks which lack the detail of the Kaolinite pattern at these peaks. Quartz was also confirmed by the 4.27Å and 3.36Å peaks. The relatively smaller peaks at 4.44Å and 2.38Å showed the presence of illite and Kaolinite. Goethite and hematite were also confirmed by the peaks 3.71Å and 2.51 Å respectively.

A comparison of the differential thermogram for the soil with standard curves due to Mackenzie (1957) and Mitchell (1976) suggests the predominance of hydrated halloysite from

TABLE 4.1: Properties of soil before modification

AASHTO Classification	A-7-6
Liquid Limit (%)	50
Plastic Limit (%)	20
Plasticity Index (%)	30
Linear Shrinkage (%)	10
Percentage Passing No. 200 B.S. Sieve	55
Group Index	13
C. KN/m^2	55
ϕ degrees	$31^{o}0'$
E KN/m^2	1785
MDD, Mg/m^3	1805
Specific Gravity	2.68
Colour	Reddish brown

TABLE 4.2: Properties of soil after modification

Properties / Lime Content	Passing No 200 B.S. sieve (%)	Liquid Limit LL (%)	Plastic Limit PL (%)	Plasticity Index PI (%)	Linear Shrinkage LS (%)	OMC (%)	MDD Kg/m^2	Cohesion C KN/m^2	Angle of Internal friction ϕ (°)	AASHTO Class'n	Elastic Modulus E KN/m^2
1%	52	38.9	22.1	16.8	7.4	16.50	1768	37.50	33° 40'	A-6 (6)	1315.0
2%	51	35.6	23.3	12.3	6.3	17.50	1750	22.5	35° 0'	A-4 (8)	1390.0
3%	50	31.4	24.3	7.1	5.1	18.40	1710	7.50	36° 52'	SA-4 (1)	1515.0
0% Before Modification)	55	50.0	20.0	30.0	10.0	16.0	1805	55.0	31° 0'	A-7-6 (12)	1785.0

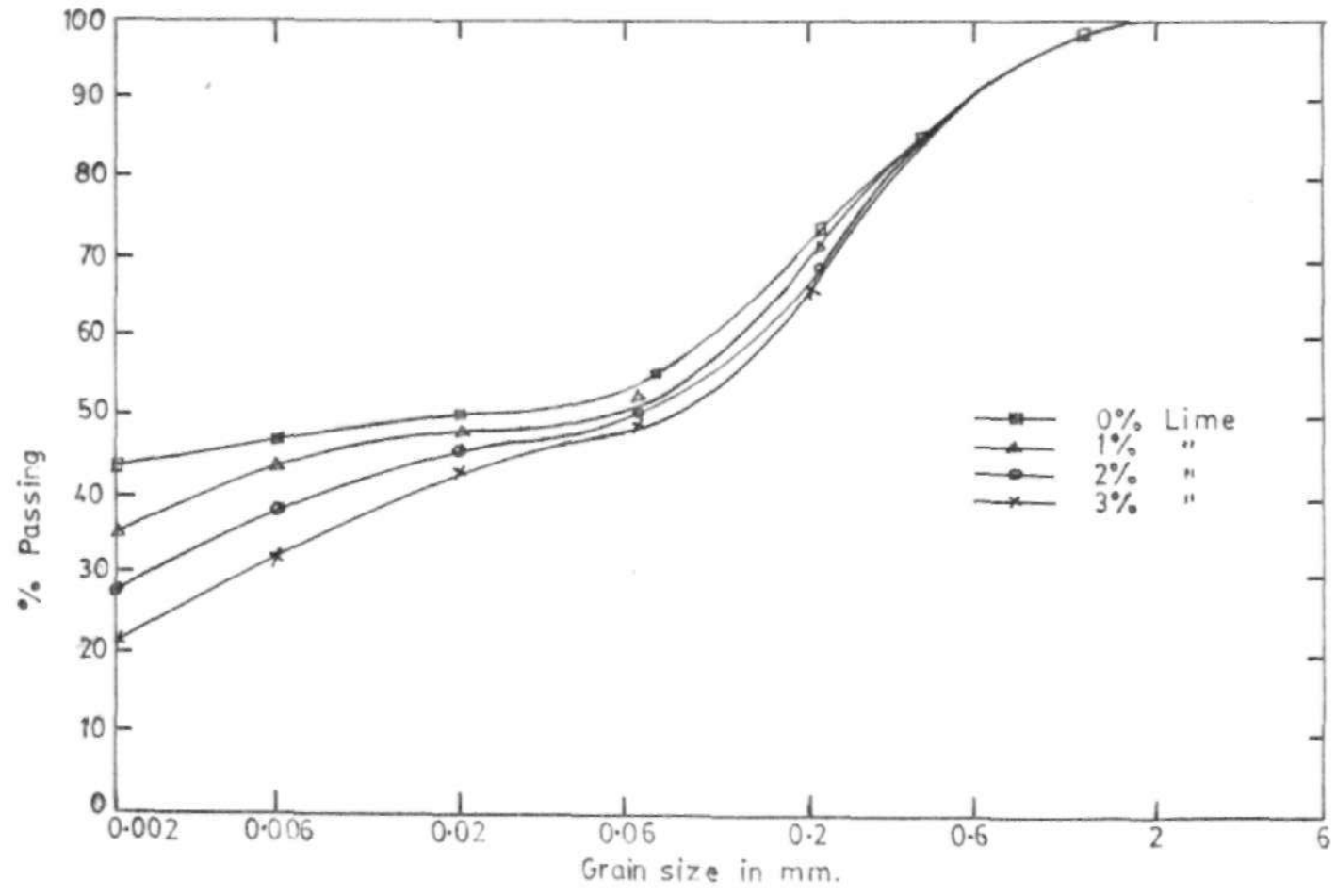


Fig.4.1: Grain size distribution for soil-lime mixtures.

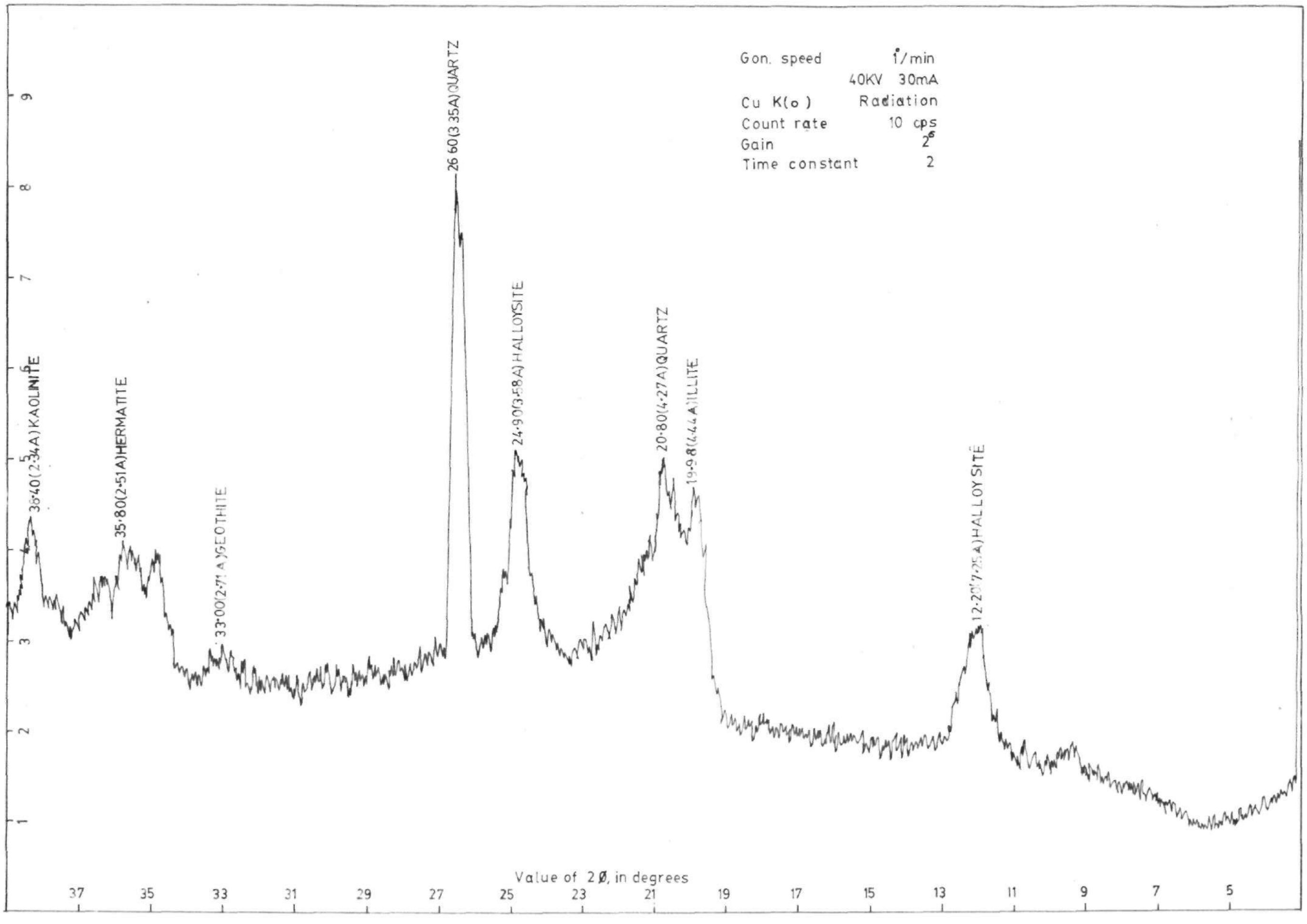


Fig. 4.2: X-ray diffractograph for soil fraction passing B.S. No.200 Sieve

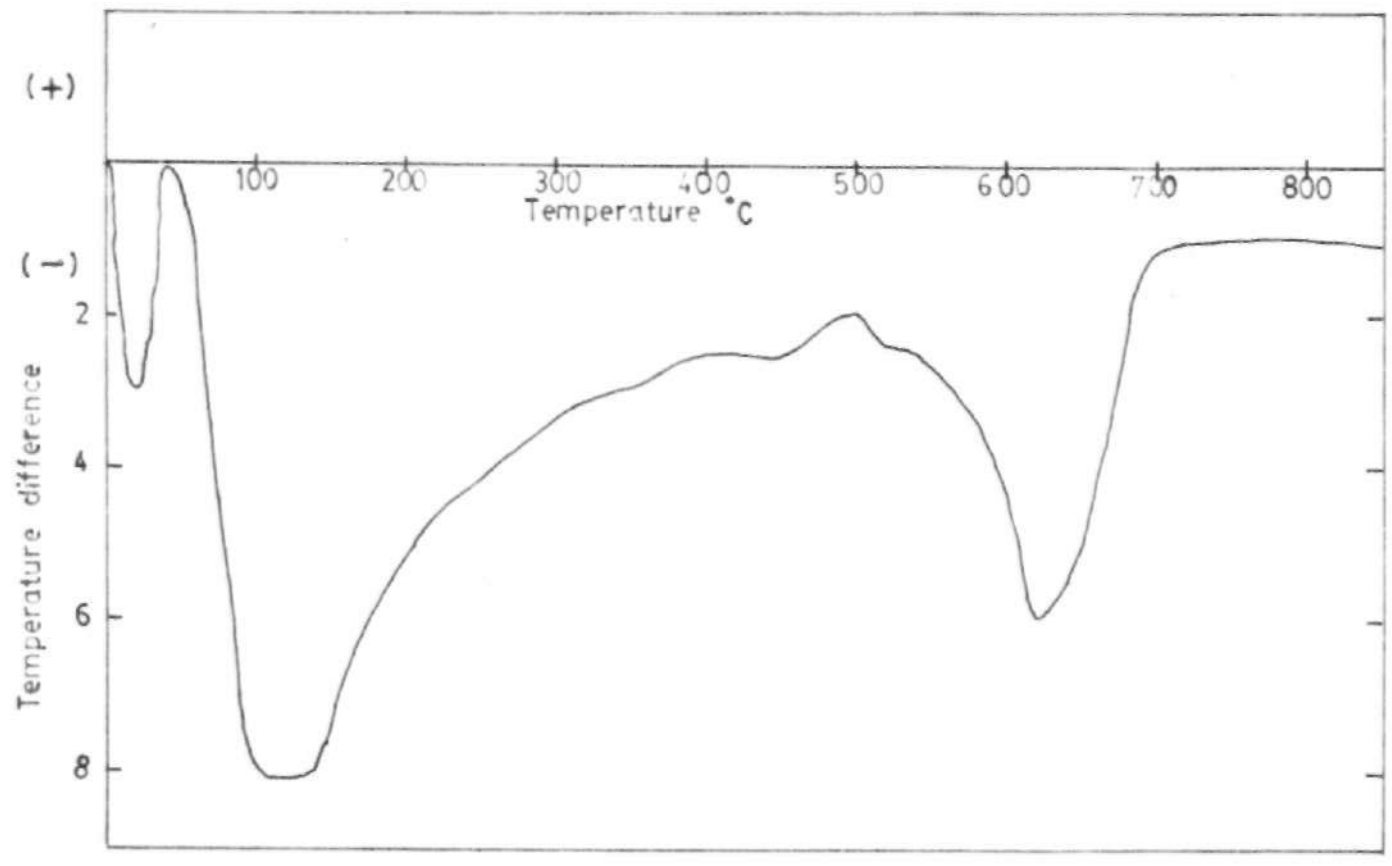


Fig. 4.3: Differential thermogram for soil fraction passing B.S. No. 200 Sieve.

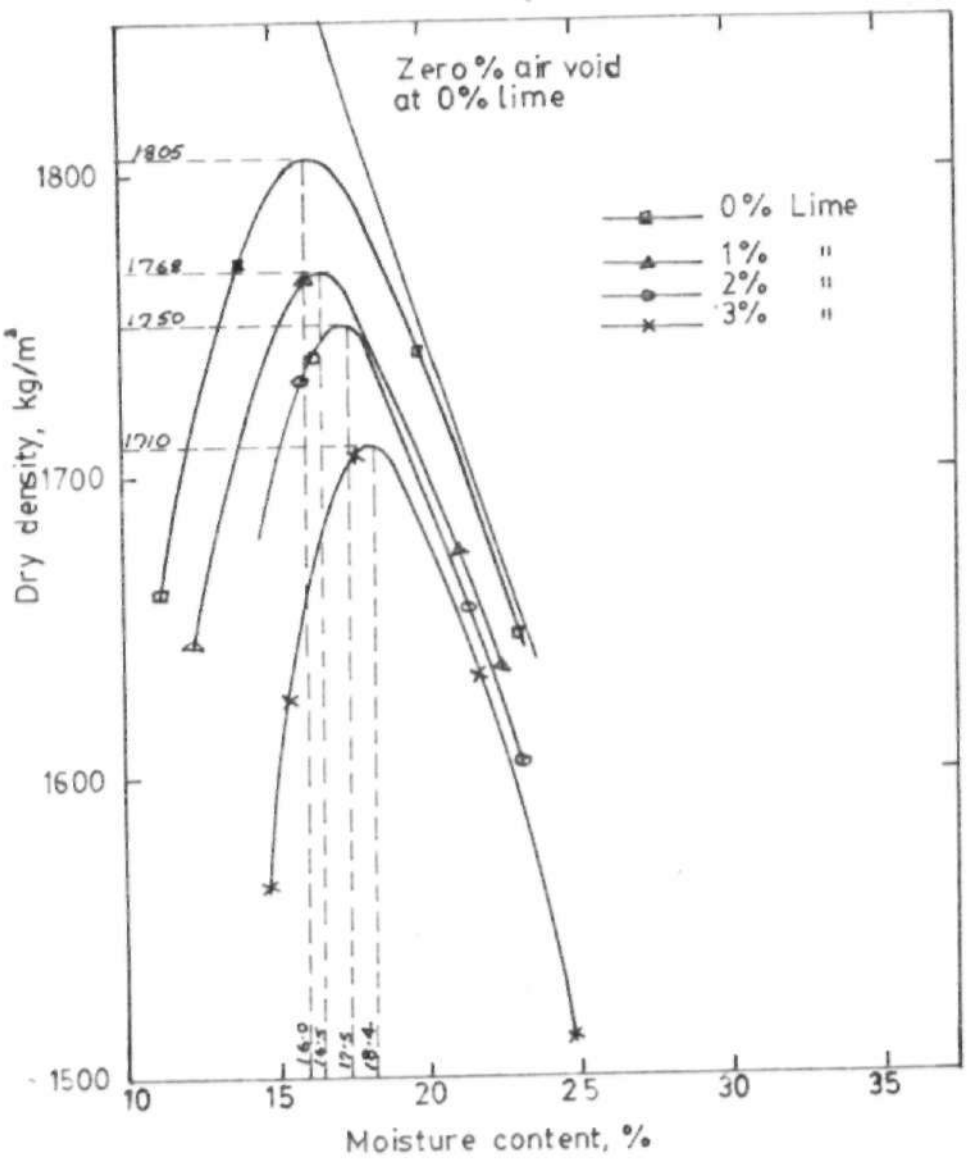


Fig.4.4: Variation of dry density with moisture content for soil-lime mixtures.

the pronounced and broad crested dehydration enotherm at 120°C and the lower peak of dehydroxylation at 620°C.

These are in agreement with the clay mineralogy of ferrallitic soils as identified by D'Hoore (1964) and Ackroyd (1967).

4.2 MODIFICATION TESTS

Compaction Characteristics - The effect of lime content on the maximum dry density (MDD) and Optimum Moisture Content (OMC) on the modified soil is shown in Figures 4.4 and 4.5. Specimens show higher OMC and lower MDD with increasing lime content. There is a sharp initial drop in MDD with lime followed by a reduced rate of drop with increasing lime content (lime contents greater than 3% not shown in the diagram). The initial drop in density is as a result of the flocculated and agglomerated clay particles (caused by cation exchange) occupying larger spaces leading to a corresponding decrease in dry density (Ola, 1977b, Lees et al 1982). The smaller drop in density with further additions of lime may be due to the replacement of soil particles in a given volume by particles of lime of comparatively lower specific gravity (Lees et al, 1982).

The increasing OMC with increasing lime content is not in line with what is expected with increasing grain size and hence decreasing surface area. These unexpected results according to Ola (1977b) are as a result of the pozzolanic reaction of lime with the soil clay constituents tending to increase the OMC. There is definitely a desire for more water in order for the soil to derive full

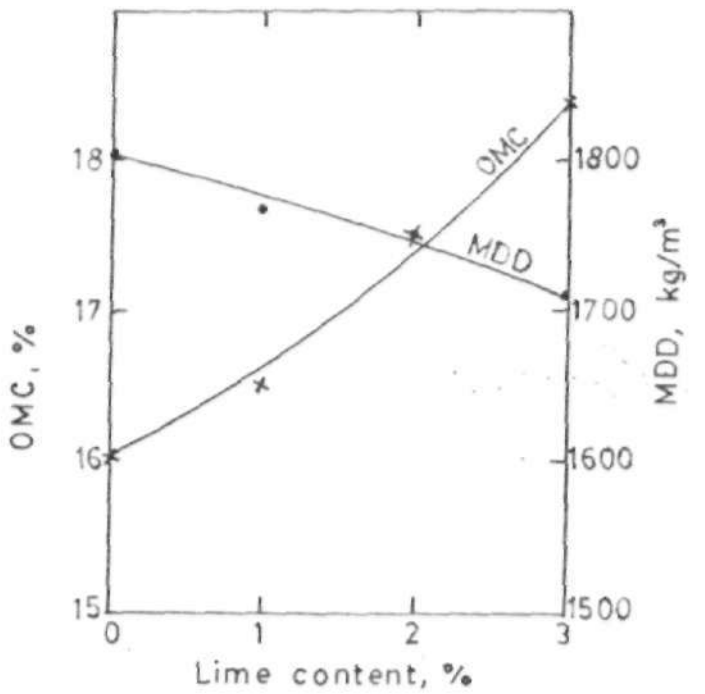


Fig. 4.5: Variation of OMC, MDD with lime content for soil-lime mixtures.

benefit from this pozzolanic reaction as the lime content is increased. Losses of 3% and 5.3% were recorded for the MDD with 2% and 3% lime respectively.

Plasticity characteristics: The effect of lime content on Atterberg limits and the variation of the values with time are shown in Figures 4.8 and 4.9, and Figures 4.6 and 4.7 respectively. The liquid limit (LL) and linear shrinkage (LS) reduce with increasing lime content while the plastic limit (PL) increases. The magnitude of the increase in plastic limit is such that the subsequent decrease in plasticity index (PI) tends to level up at higher lime contents. The reduction in liquid limit with lime content can be as a result of the cation exchange reaction which flocculates the soil particles and reduces the clay size fraction and hence the soil surface area. The reduction in Liquid limit with time lends more credence to the assertion of a reaction of an ion exchange nature. The increase in PL seems to be of immediate effect as not much change takes place with time for any lime content. This may be the contribution of the highly plastic nature of lime, the net result being an immediate increase in the PL that may be unconnected with any lime-soil reactions.

TABLE 4.2: Effect of lime content on Atterbeg limits for soil-lime mixtures

Time and Lime Content / Atterberg Limits		Liquid Limit LL (%)	Plastic Limit PL (%)	Plasticity Index PI (%)	Linear Shrinkage LS (%)
1hr	1%	48.5	20.7	27.8	8.3
	2%	45.9	22.4	23.5	6.9
	3%	42.2	23.2	19.0	5.8
3hr	1%	46.0	21.2	24.8	8.1
	2%	41.8	22.5	18.8	6.7
	3%	37.6	23.7	13.9	5.5
5hr	1%	41.4	22.2	19.2	7.9
	2%	39.3	23.5	15.5	6.5
	3%	34.0	24.7	10.3	5.2
7hr	1%	38.9	22.1	16.8	7.3
	2%	35.2	23.3	11.9	6.3
	3%	31.4	24.3	7.2	5.1

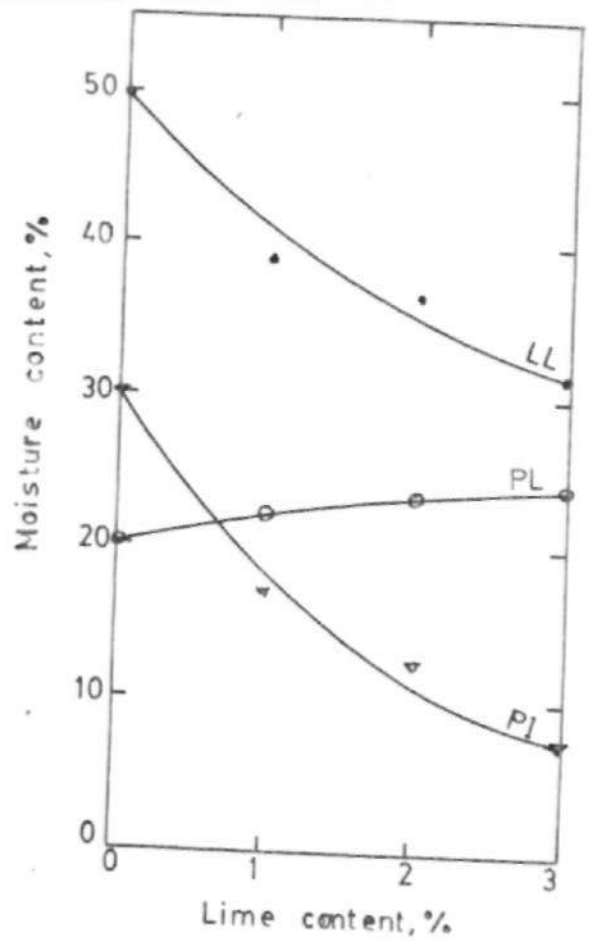


Fig. 4.6: Variation of LL, PL, and PI with lime content.

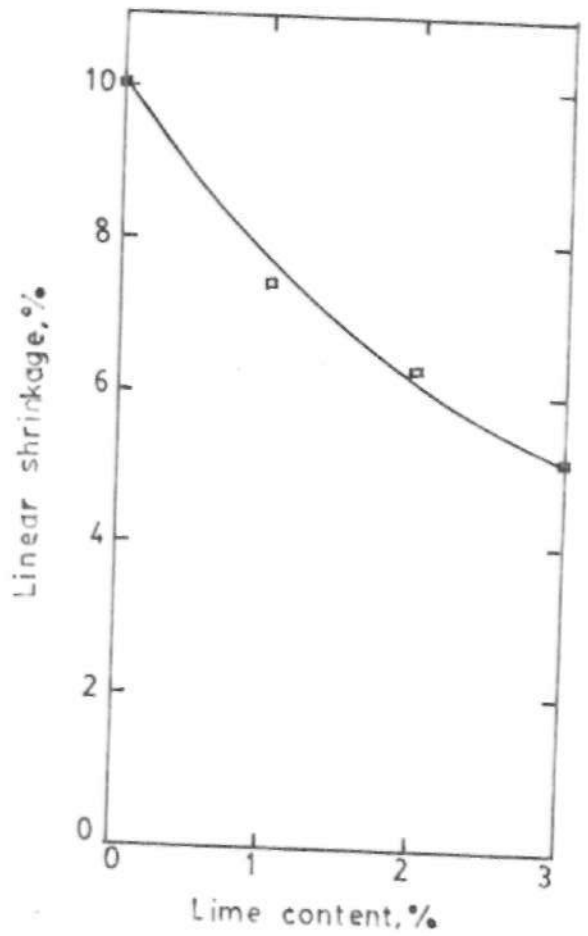


Fig. 4.7: Variation of Linear Shrinkage with lime content.

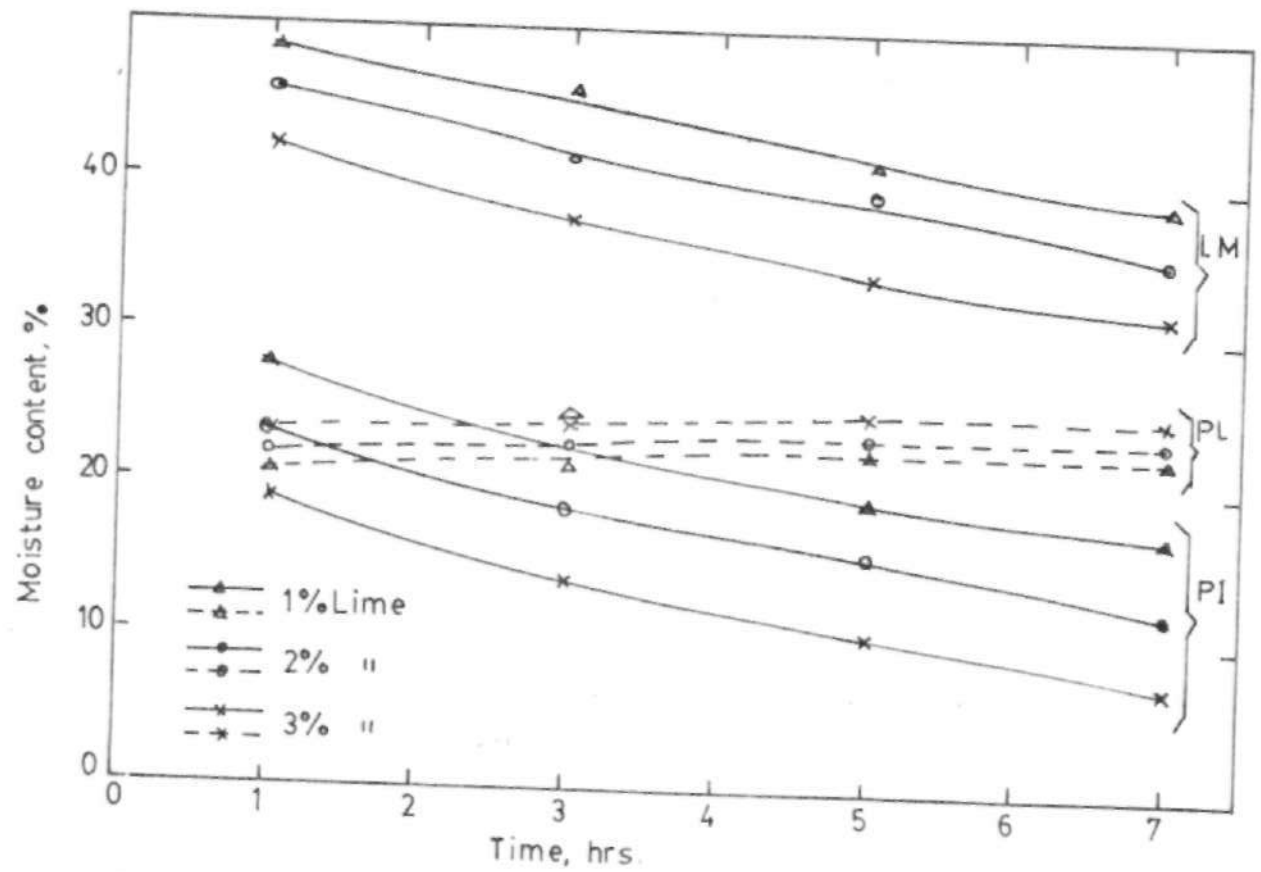


Fig. 4. 8: Variation of Atterberg limits with time for soil-lime mixtures.

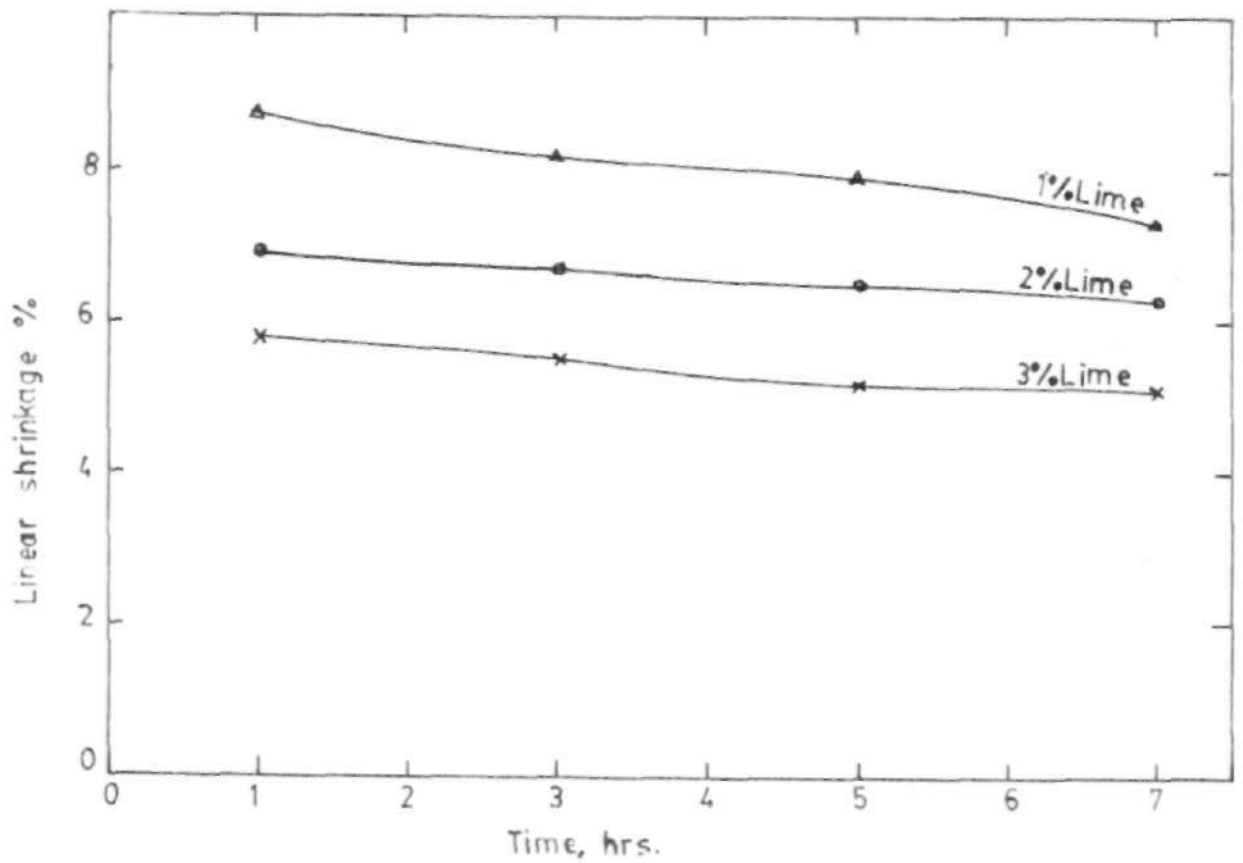


Fig.4.9: Variation of linear shrinkage with time for soil-lime mixtures.

Grain size Distribution: The grain size distribution curves for the modified soil are shown in Figure 4.1. A reduction in the percentage of fines with lime content is noticed. Not much change is noticed in the coarser sizes which predominantly are quartz and iron oxide (hematite). Changes in the shapes of the curves are very apparent from 600 and become more marked at 200 down to the clay size. This is an indication that the modification reaction is between lime and the clay minerals. The reaction is certainly that of cation exchange since little or no benefit can yet be derived at this early stage (a matter of few hours) from pozzolanic reaction.

The original soil contained 44% clay size which decreased to 35% at 1% lime, 27% at 2% lime and 22% at 3% lime. The rate of reduction in clay size is obviously decreasing with increasing lime content.

From a consideration of the grain size and plasticity characteristics of the lime-treated soils, very little benefit (which may not offset increased costs) is gained after 3% lime. The CBR requirements are met with 3% lime after seven hours "modification reaction period". All the lime contents considered seem to pass the plasticity requirements but the grain size requirement, of not more than 50% passing the U.S. No. 200 sieve, is marginally met at 3% lime. If a reaction time of 7 hours is considered enough in order to avoid delays

to farmers site works, the LL and PI limits of 40% and 16% respectively will also give a "lime modification optimum" of 3%. Since the main purpose of modification is to reduce soil plasticity and hence increase workability, the compromise of 2 hours modification reaction time and the use of plasticity criterion alone is well worth adopting.

C - ϕ parameters: The effects of lime content on the cohesion and angle of internal friction are shown in Figure 4.12. The plots of shearing stress versus normal stress from which the cohesion and angle of internal friction are derived are shown in Figure 4.10. The stress-strain relationship and the variation of modulus of elasticity with lime content are presented in Figures 4.11 and 4.13 respectively.

The cohesion is seen to reduce while the angle of internal friction increases with increasing lime content. These are as a result of the reduction in clay size fraction which resulted from the ion exchange reduction. These are in agreement with Fossberg (1965) and Lees et al (1982).

4.3 STABILIZATION TESTS

Compaction Characteristics: Data for Optimum moisture content (OMC) and Maximum dry density (MDD) for soil-lime-cement mixtures are presented in Table 4.4 and plotted in Figures 4.17 and 4.18 for various cement contents. The base optimum moisture content/maximum dry density relationships for the

TABLE 4.4 : Variation of OMC (%) and MDD (Kg/m^3) with lime and Cement Contents

Cement Content / OMC/MDD / Lime Content	0%		2%		4%		6%		8%	
	OMC	MDD	OMC	MDD	OMC	MDD	OMC	MDD	OMC	MDD
0%	16.0	1805	-	-	20.5	1580	19.7	1599	19.1	1661
2%	17.5	1750	19.7	1735	19.2	1752	18.2	1760	16.7	1801
3%	18.4	1710	19.4	1644	18.7	1667	17.7	1683	15.7	1693

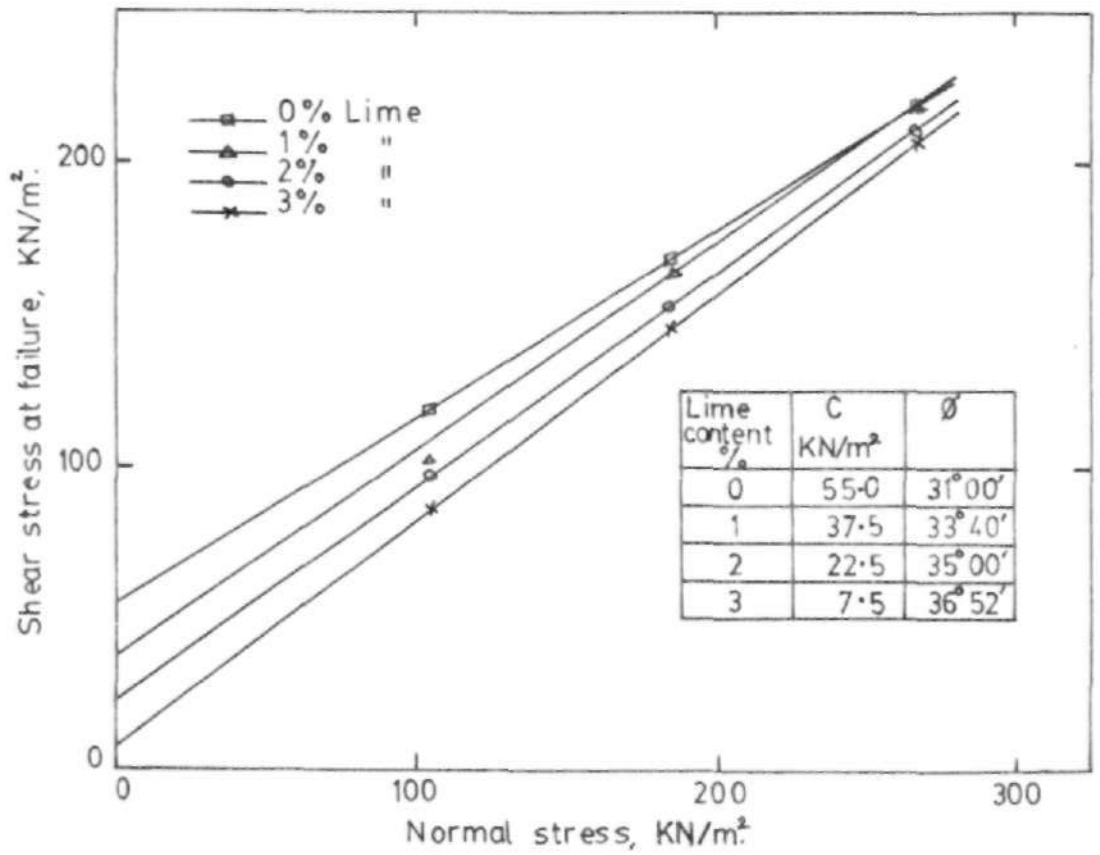


Fig. 4.10: Variation of shear stress with normal stress for soil-lime mixtures.

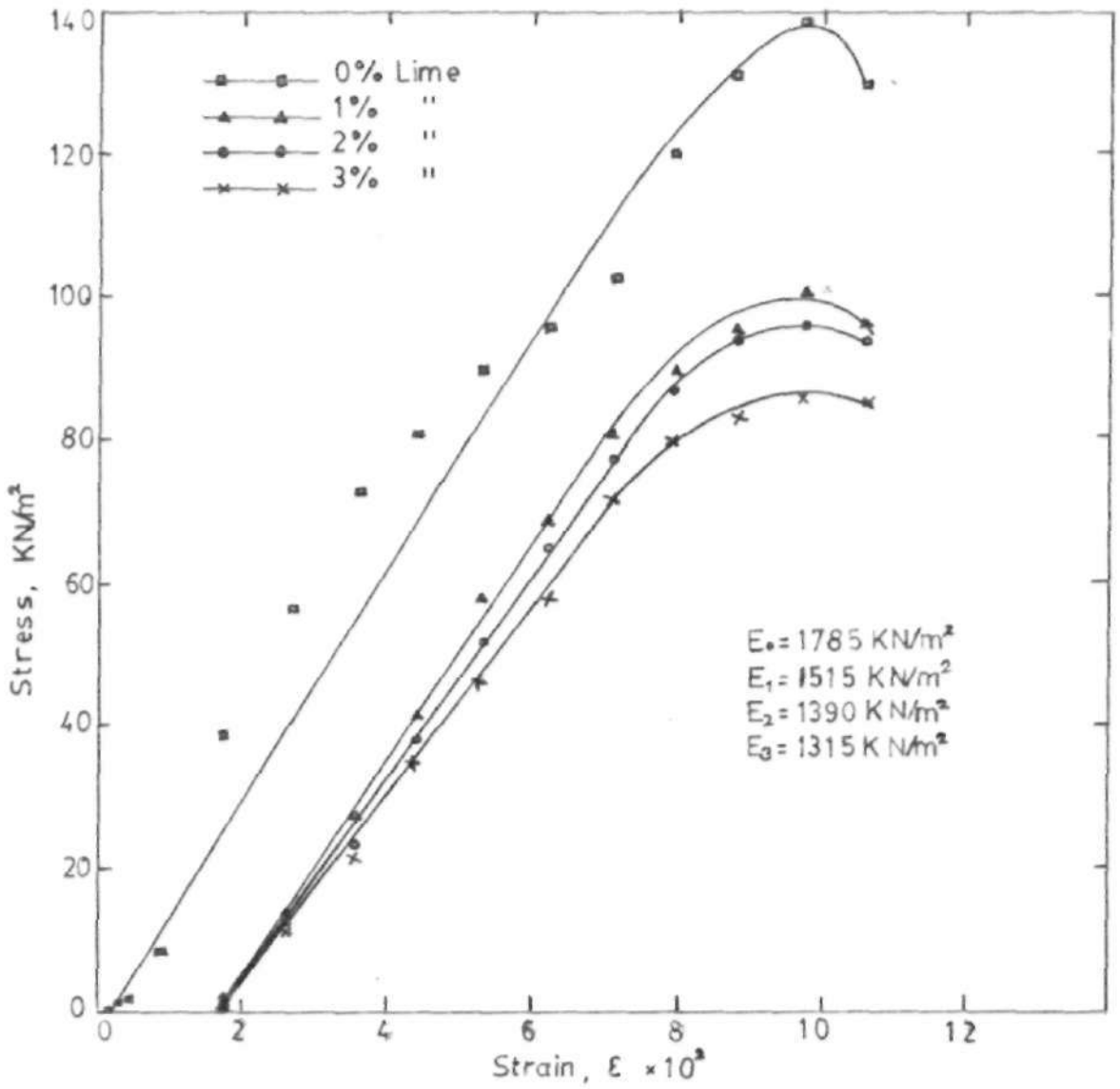


Fig.4.11: Stress-strain curves for soil-lime mixtures.

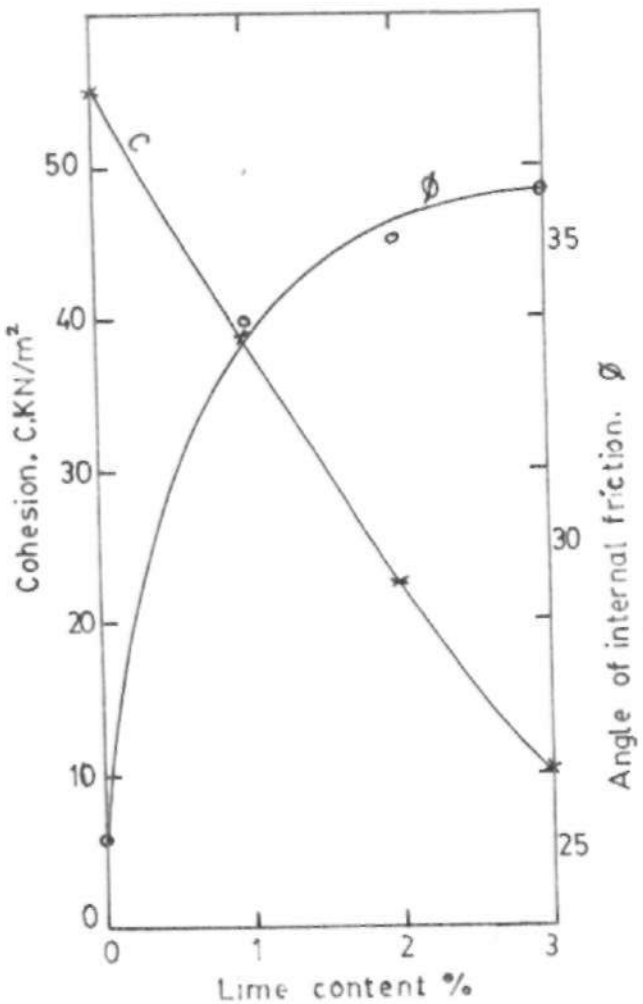


Fig.4.12: Variation of C and Ø with lime for soil-lime mixtures.

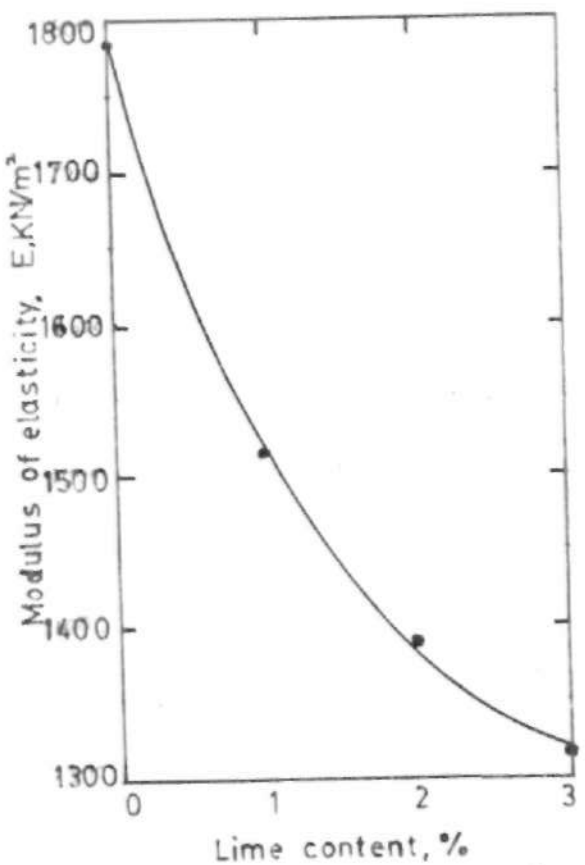


Fig.4.13: Variation of Modulus of Elasticity (E) with lime for soil-lime mixtures.

individual mixtures are plotted in Figures 4.14, 4.15 and 4.16 for 0%, 2%, and 3% lime contents respectively.

The OMC generally increases with cement content for each lime content until a maximum is reached after which there is a fall (see Figure 4.17). The initial increase may be due to the increased surface area of particles caused by cement addition. This now needed more water to lubricate the entire matrix of soil, lime and cement to enhance compaction, in addition to the water taken up by the cement hydration reaction. Further addition of cement meant increased ion exchange reaction resulting in accentuated flocculation and hence increased grain size which now required less water for compaction. This reaction is the lowering of the zeta potential or repelling action of the clay particles due to the oversaturation of calcium ions on the clay particle surfaces from the hydrated lime product of cement hydration.

The MDD - cement content curves are virtually a mirror image (up-side-down) of the OMC-cement content curves. Generally, for each lime content, there is a decrease in density with cement content until a minimum is reached when the density starts rising. With reference to the OMC-cement content curves, this trend is in order and agrees with the increasing OMC-decreasing MDD and decreasing OMC -increasing MDD pattern. The initial decrease in density is due to the excess free lime from admixed lime. As the cement content is increased the heavier cement particles fill the voids of the flocculated

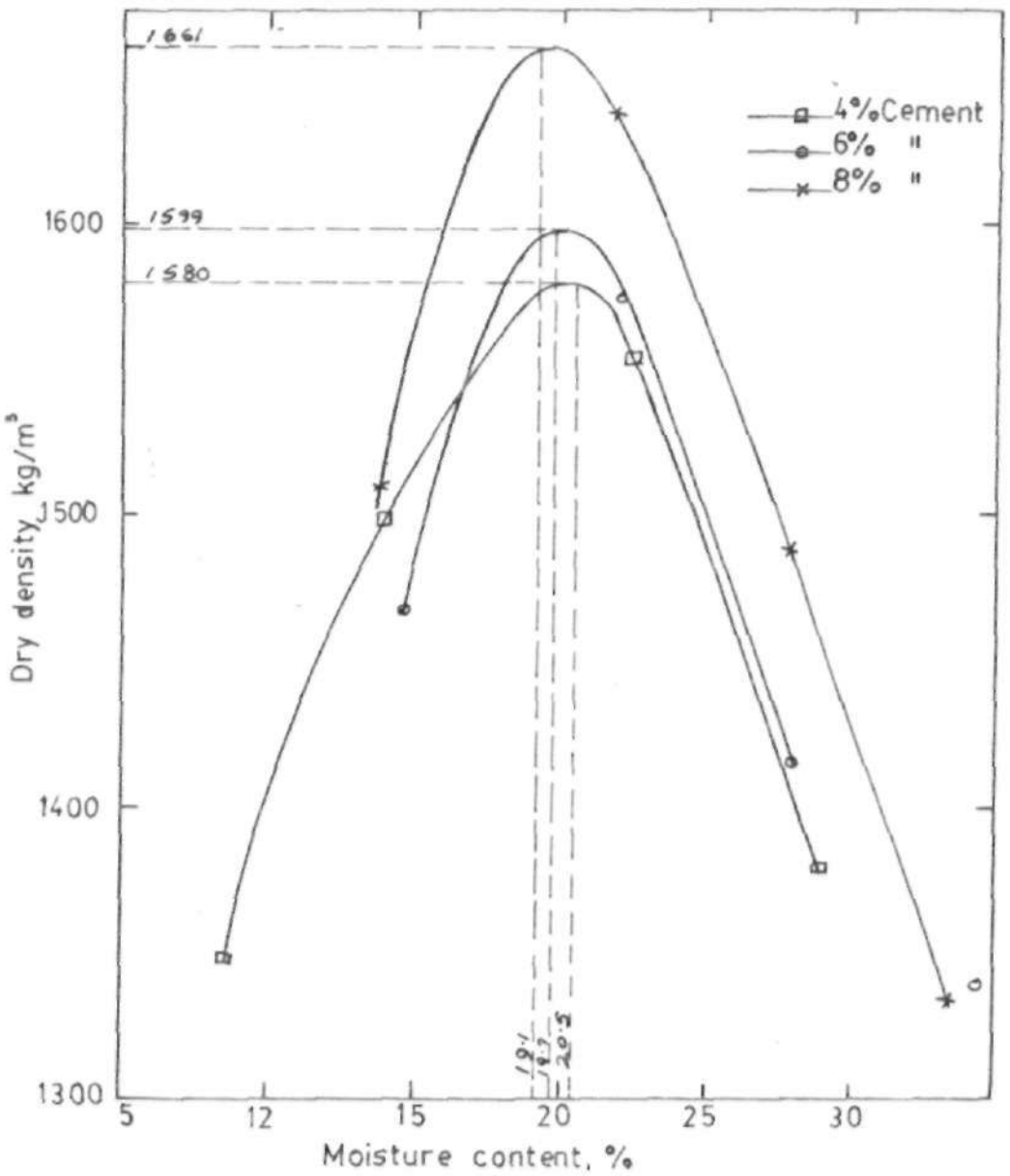


Fig. 4.14: Variation of dry density with moisture content for soil-cement (i.e. 0% lime).

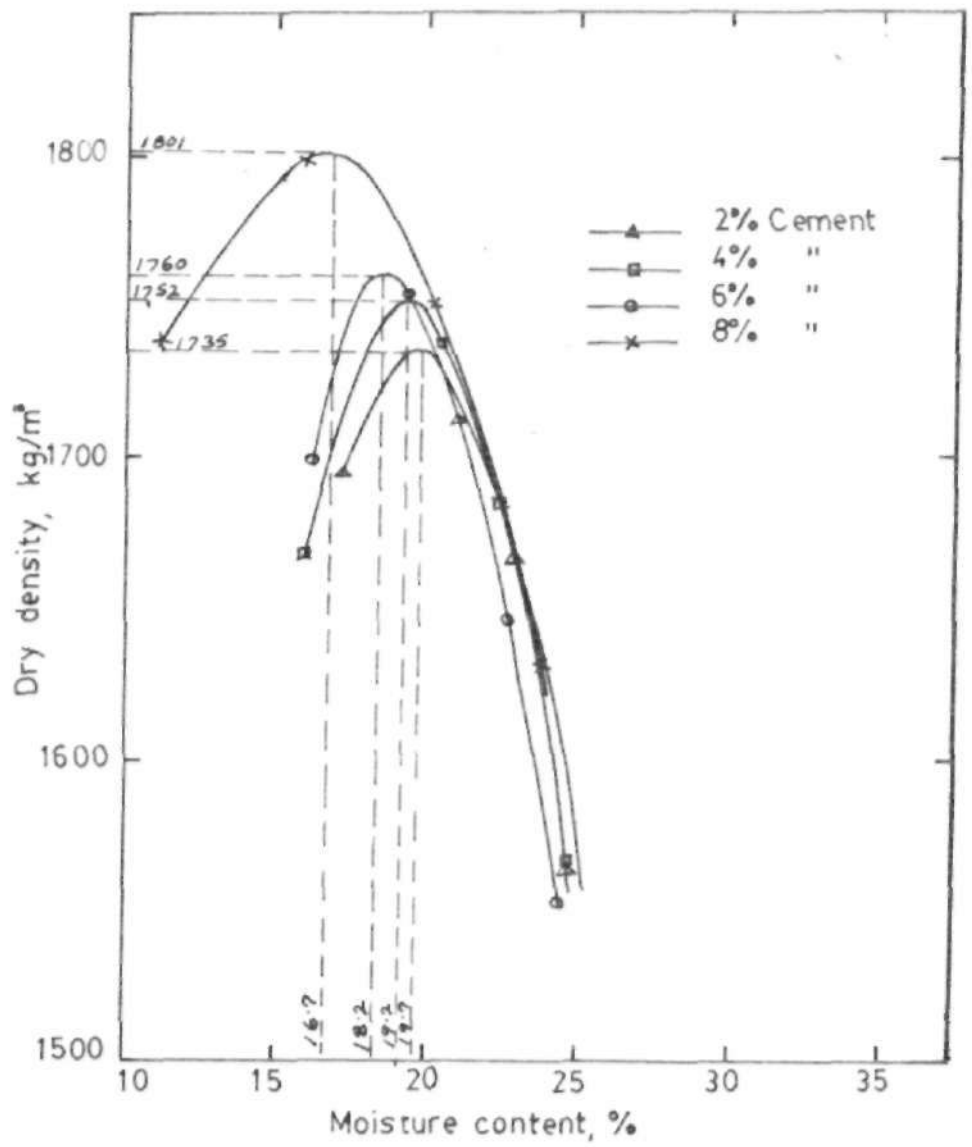


Fig. 4.15: Variation of dry density with mosture content for soil-lime-cement mixtures—2% lime.

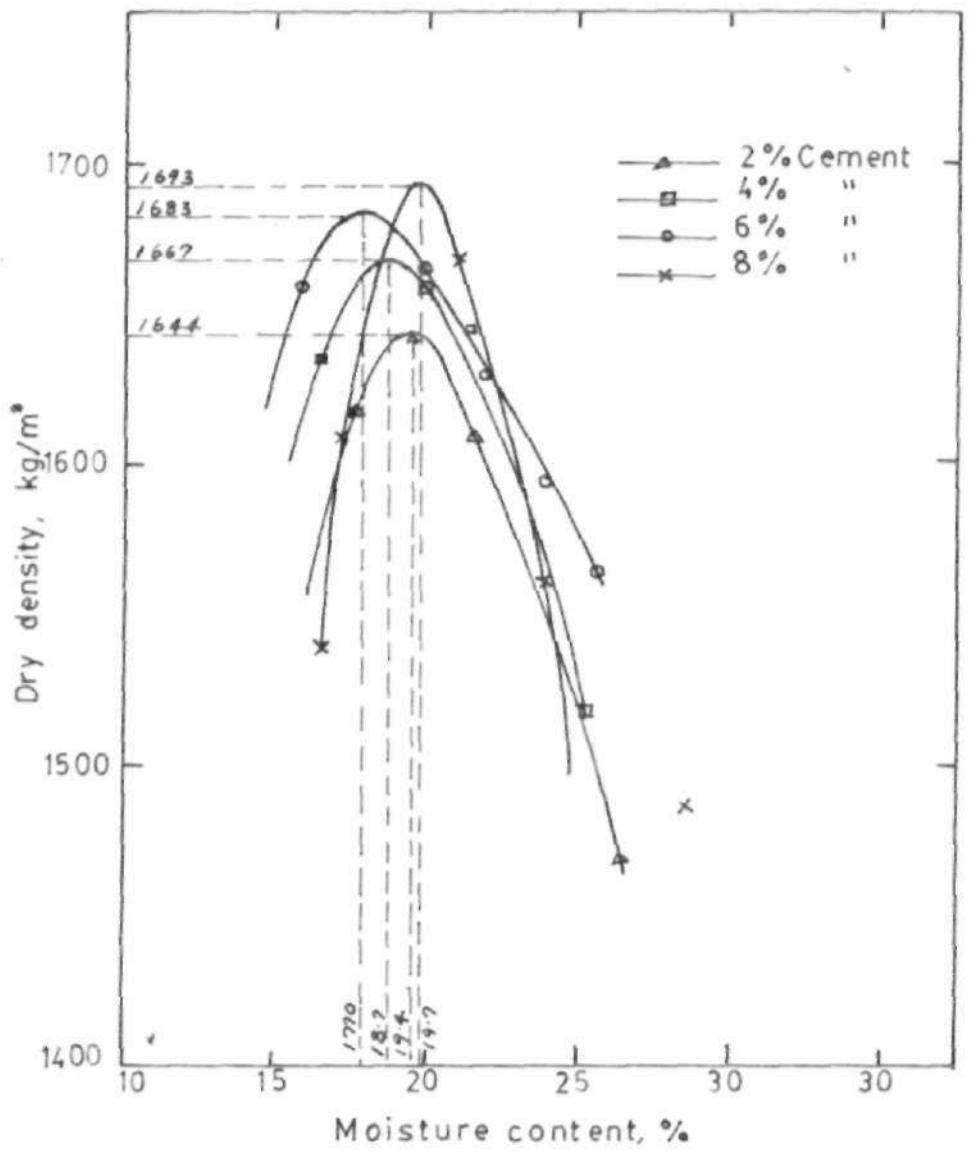


Fig. 4.16: Variation of dry density with moisture content for soil-lime-cement mixtures—3% lime.

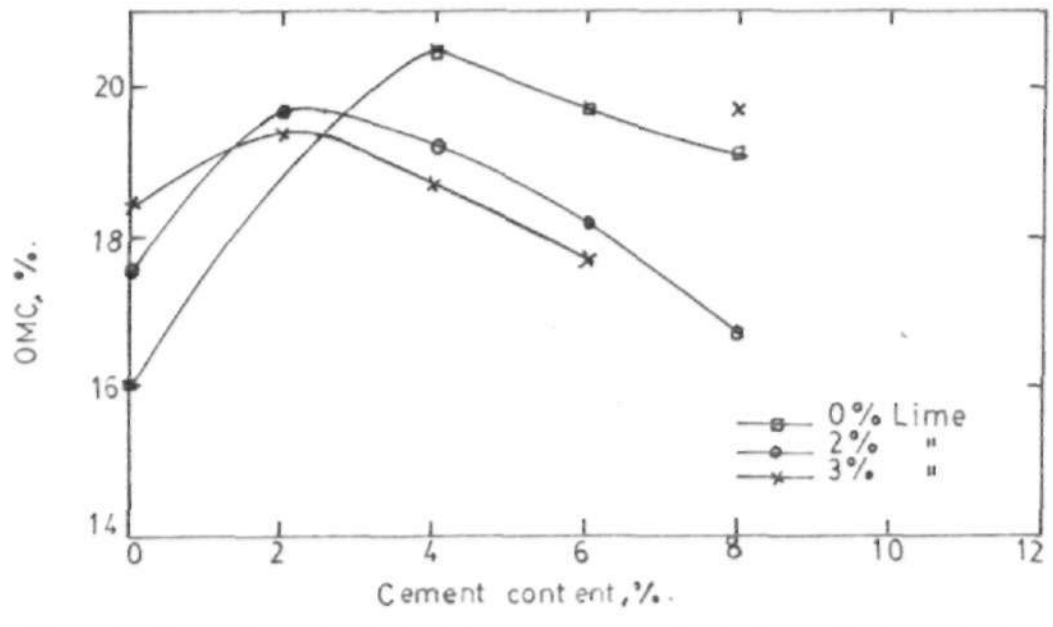


Fig.4.17: Variation of OMC with cement content for soil-lime-cement mixtures.

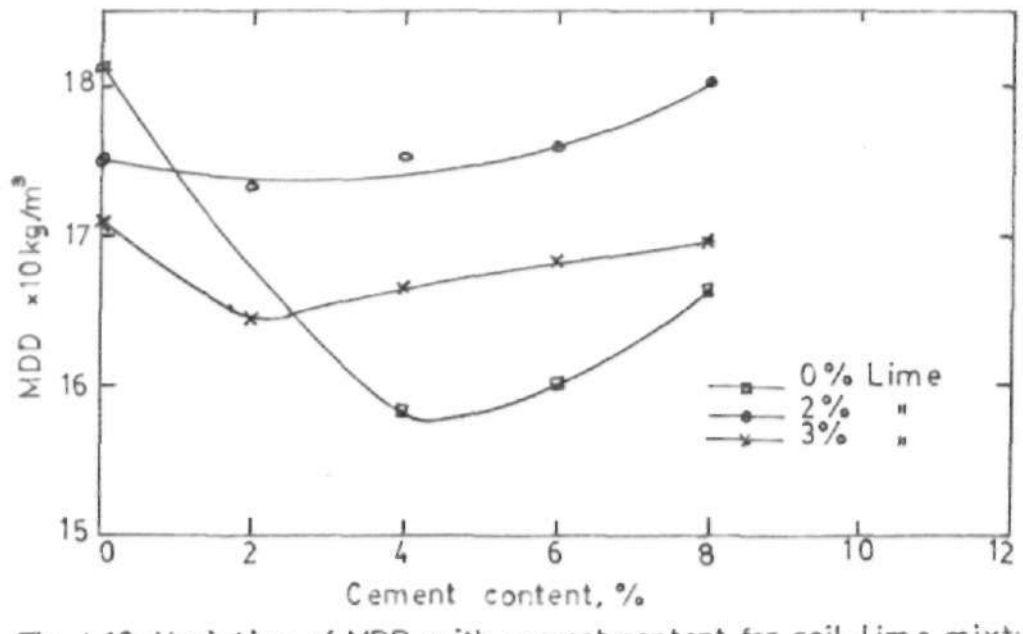


Fig.4.18: Variation of MDD with cement content for soil-lime mixtures.

soil to give a denser matrix.

Substantial part of the MDD lost by modification is restored by the addition of cement for the 2% lime soil-lime-cement mixtures while even at 8% cement for 3% lime soil-lime-cement mixtures no loss is recovered. 93% of the loss due to 2% lime was restored by 8% cement addition while 6% cement addition restored 18% of the loss in MDD.

Strength characteristics:-

The unconfined compressive strength (UCS) tests results are presented in Tables 4.5 and 4.6 and plotted in Figure 4.20. Those of the california bearing ratio (CBR) are presented in Table 4.7 and plotted in Figure 4.19. The graphical relationship between UCS and CBR and between UCS and MDD are as shown in Figures 4.21 and 4.22 respectively.

The UCS-cement relationship seem to be orderly from the performance of the specimens at 7 days and 28 days though strengths are generally lower than the 1.73N/mm^2 criterion especially for the soil-lime-cement mixtures. This poor showing is very much contrary to expectations. Generally, higher strengths were developed for soil-lime-cement mixtures at about 6% cement content and lower, than for soil-cement. The reverse trend is to be noticed after this cement content. Although additional strengths developed when curing was extended to 14 and 28 days (Table 4.5), rate of gain in strength is not as high as with mixtures made with cement alone. More losses in strength and less resistance to loss

TABLE 4.5: Effect of cement content on UCS (N/mm^2) for soil-lime-cement mixtures

Cement content / Curing Period (days) / Lime content	2%			4%			6%			8%			12%
	7	14	28	7	14	28	7	14	28	7	14	28	7
0%	-	-	-	0.48	0.69	1.84	1.16	1.45	3.68	1.65	2.03	[illegible]	2.42
2%	0.48	0.69	1.45	1.26	-	2.03	1.36	1.65	3.78	1.45	2.42	[illegible]	-
3%	0.69	0.97	1.84	1.36	-	2.52	1.45	1.74	3.97	1.65	2.23	[illegible]	-

TABLE 4.6: Effect of immersion in water on UCS for soil-lime-cement mixtures

Cement content / Curing Period (days) / Lime content	2%			4%			6%			8%		
	14	7+7	Ri* %	14	7+7	Ri* %	14	7+7	Ri* %	14	7+7	Ri* %
0%	0.69	0.39	57	1.45	0.87	60	2.03	1.84	91			
2%	0.69	0.48	70	1.65	1.26	76	2.42	1.84	76			
3%	0.97	0.48	50	1.74	1.16	67	2.65	1.84	83			

*Ri = Resistance to loss in strength $= \frac{P_i \times 100\%}{P_c}$, P_i is soaked UCS, P_c is unsoaked UCS

TABLE 4.7 CBR (%) values for soil-lime-cement mixtures.

Cement content / Lime content	2%	4%	6%	8%	12%
0%	-	9	50	95	130
2%	26	-	90	30	-
3%	27	-	70	95	-

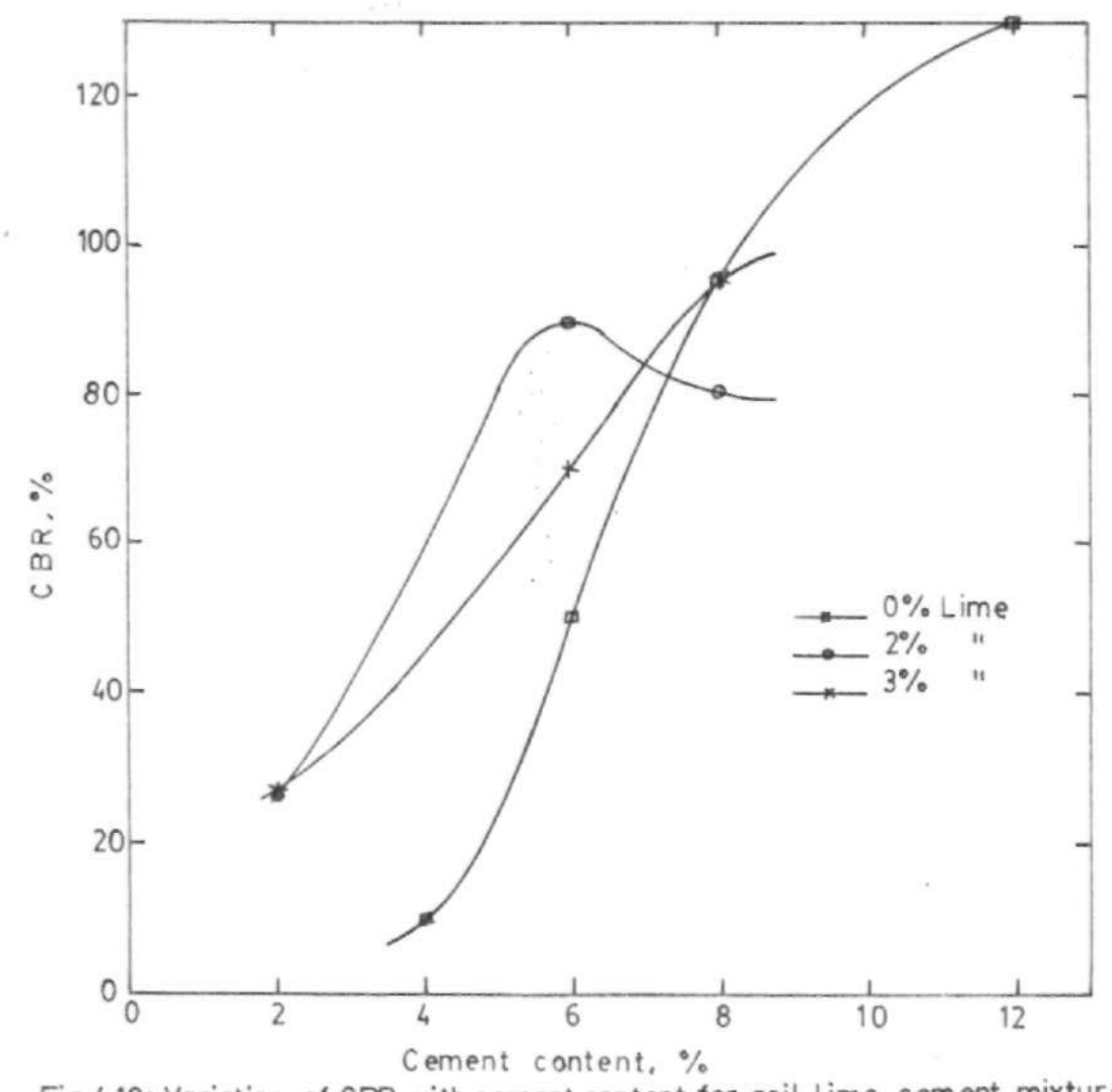


Fig.4.19: Variation of CBR with cement content for soil-lime-cement mixtures.

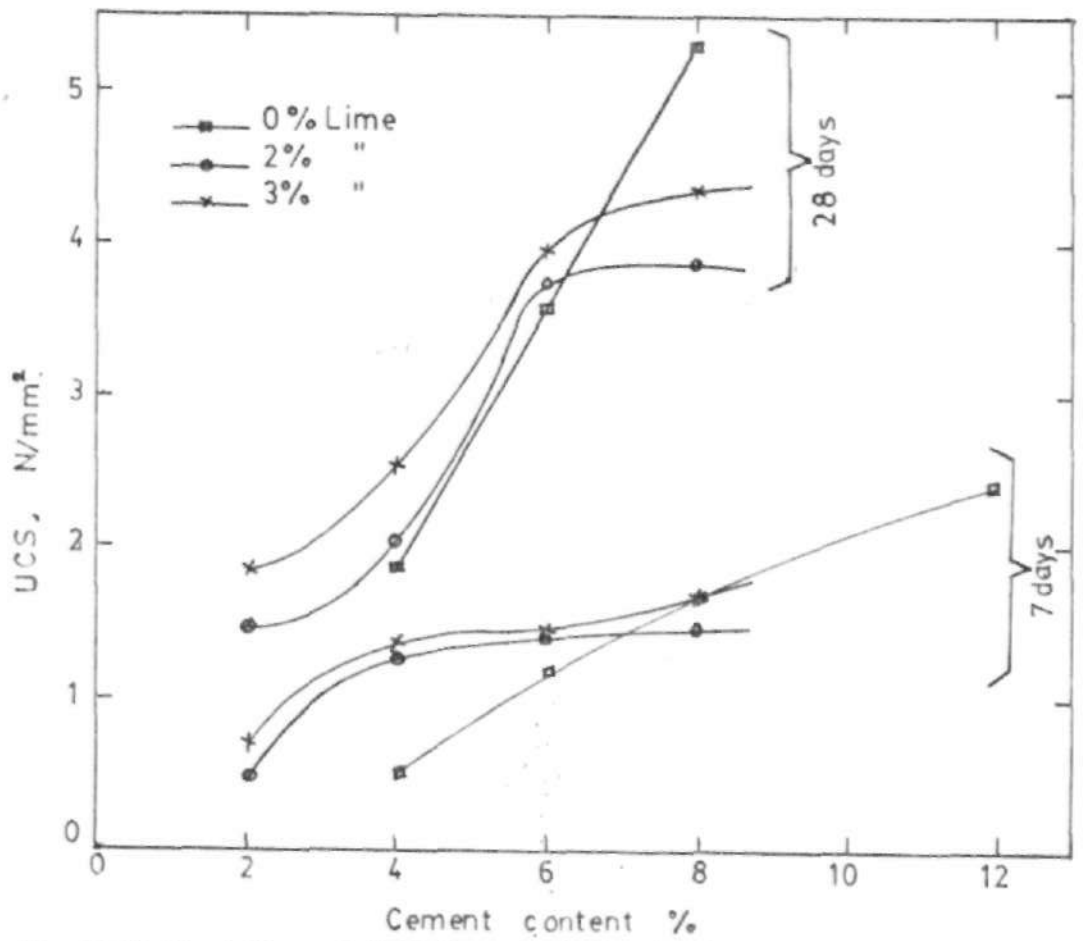


Fig.4.20: Variation of UCS (7 and 28 days) with cement content for soil-lime-cement mixtures.

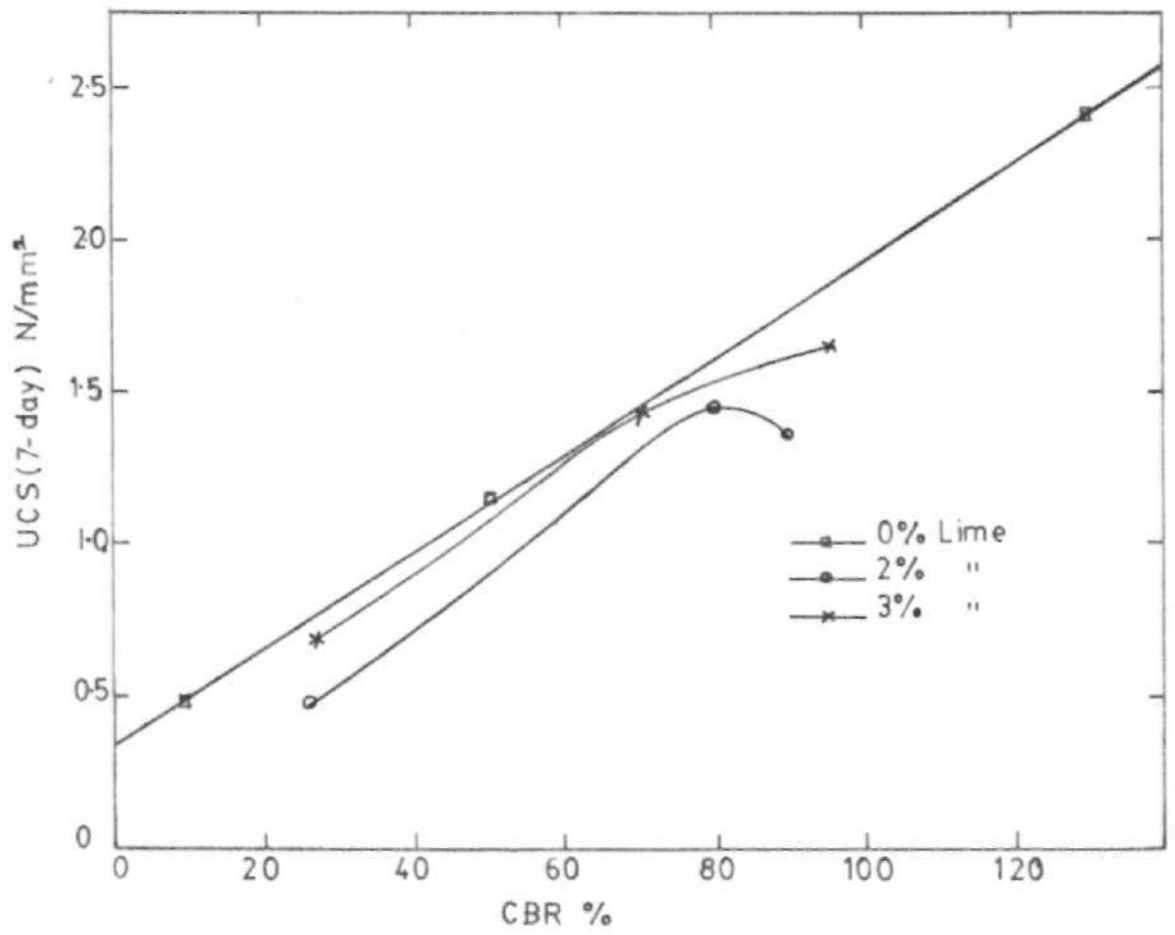


Fig.4.21: Variation of UCS with CBR for soil-lime-cement mixtures.

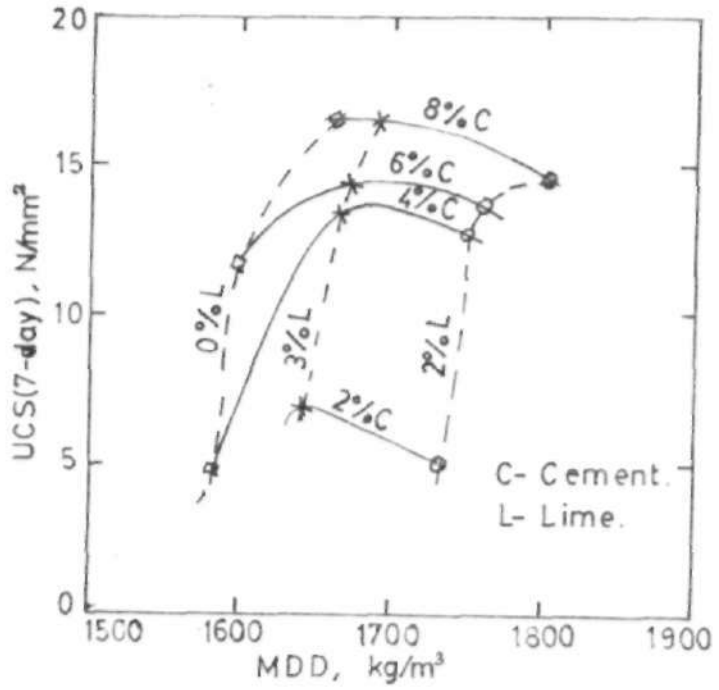


Fig.4.22: Variation of UCS with MDD for soil-lime-cement mixtures.

in strength (Table 4.6) were recorded for soil-lime-cement mixtures. All these go to show that there is little or no benefit in using lime as an admixture in the cement stabilization operation on this Agbede laterite. The reason for this is not far fetched. A comparison of UCS (7day) results for soil-cement with those for a similar soil (Ola 1978b) shows that the clay content has had a major role to play in the strength development. The high amount of clay in the Agbede laterite (44%) compared with 22% in that used by Ola would have needed much higher cement for adequate stabilization.

Even now that much higher strenths have not been achieved for soil-lime-cement mixtures, the liberation of free lime will increase the cation exchange reaction by increasing the exchange sites, and the pozzolanic reaction which will eventually give noticeable difference in strength at longer periods of curing than considered here.

However, the low strengths could have been increased by wetting the mix after compaction or increasing the compaction water. Experiments by Sabry and Parcher (1979) on soil-lime samples compacted at OMC, dryer and wetter, showed higher strength gains with the specimens compacted wetter than OMC, especially for high lime contents. This they explained on the efficient calcium diffusion fostered by high water contents which reduces the possibilities of local lime concentration in order to produce uniform amounts of Ca^{++} and OH^{-} ions.

The explanation for the low UCS values may be that the pozzolanic reaction of a typical lime stabilization now

predominates over the hydration reactions of a typical cement stabilization which would have led to high initial strength gain. The hydration of cement produces tobermonite gel (Calcium silicate hydrate) and hydrated lime from the tricalcium and dicalcium silicate components of cement. The production of the usual amount of tobermonite (this is a high initial strength gaining product of cement hydration (Hedzi, 1975))is hindered by the presence of admixed lime which has started a pozzolanic reaction with the clay component of the soil. The extent to which this hindrance occurs will obviously be proportional to the amount of clay in the soil. At much higher clay contents, initial strengths may be less for soil-lime-cement mixtures than for soil-cement mixtures as has been the case in this work. Part of the highly reactive hydrated lime produced from cement hydration (Gilliot and Berrada, 1966) reinforces and accentuates the pozzolanic reaction started by the lime of modification while the remaining part is left to react with the tricalcium aluminate and tetracalcium aluminoferrite components of the cement to form the low continued strength producing compounds of tetracalcium aluminate hydrate and calcium alumino-ferrite hydrate. The latter reaction either continues in parallel with the pozzolanic reaction till around a few months after which only the pozzolanic reaction continues or reinforces the pozzolanic reaction to give increased continued strength gain even after several months. Full benefit of soil-lime-cement stabilization is, therefore, to be realized

only during and after this latter part of the reactions.

A look at Table 4.6 shows that there is increased resistance to loss of strength with increasing lime and cement contents. The best values obtained are for 0% lime with 8% cement which is 90%, 2% lime with 6% cement which is 76%, 2% lime and 8% cement which is 76%. The mixture with 3% lime and 6% cement recorded very low value while that with 3% lime with 8% cement gave 63%. More efficient mixing could have given better results especially for the soil-lime-cement mixtures. The fact that better general resistance to loss in strength is obtained with 2% lime means that the success of stabilization does not depend on the time allowed for modification and hence the extent of modification. It suggests therefore that strength gain for soil-lime-cement mixtures is partly due to increased density or close packing of the soil particles and partly due to the physico-chemical reactions between the lime and cement and the soil particles resulting in interparticle bonding. This is clearly shown in Figure 4.22.

Recorded California bearing ratio (CBR) values are generally low. The Nigerian General specification's 180% criterion was not met for the range of mixtures tested. The highest recorded CBR is 130% for soil-cement with 12% cement content. The highest for the soil-lime-cement mixtures is 95% for 3% lime with 9% cement. Generally, the trend is increasing CBR with increasing cement content for mixtures with 0% lime

(soil-cement) and 3% lime up to a point where rate of increase in CBR reduces. The CBR-cement content relationship seem to follow the same pattern as the UCS-cement content relationship. The same argument will thus apply. The low values show that either this form of stabilization cannot be economically applied to this soil using the 160% CBR value, or a low value should be used if the lime stabilization reaction effect is to be accepted for this form of stabilization. From the curves, a value of 90% would seem appropriate in which case a mixture with 2% lime and 6% cement will be adequate. With 3% lime, a mixture with 7% cement will also be adequate but obviously more expensive than 2% lime with 6% cement.

CHAPTER FIVE

5. CONCLUSIONS

From the results of the investigations carried out, the following conclusions are made:-

1. The agbede laterite, identified to be an A-7-6 soil in the AASHTO classification system, contains hydrated halloysite as the predominant clay mineral with traces of kaolinite and illite. Hematite, geothite and quartz were also identified. These are in general agreement with the description of ferrallitic laterites by D'Hoore (1964) and Ackroyd (1967).

2. As expected and in agreement with Kedzi (1975), lime modification significantly increased the soil grain size especially in the silt-clay fraction rendering the soil more friable and granular. The clay size was reduced from 44% to 22% (50% reduction) by the addition of 3% lime.

3. Lime modification produced significant increases in the soil's angle of internal friction and reductions in the elastic modulus. The frictional angle increased by 8.6%, 12.9% and 18.9% with the addition of 1%, 2% and 3% lime respectively. Cohesion was reduced by as much as 81.9% with 3% lime, while the elastic modulus was reduced by 26.3% from 1785KN/m^2 to 1315KN/m^2 with 3% lime addition. All these are after 7 hours "modification reaction time".

4. With the addition of lime, liquid limit (LL) reduced while plastic limit (PL) increased and consequently plasticity index (PI) reduced. The improvement was likely to have continued for days but at very reduced rate as reported by Kedzi (1975). The reduction in PI was due to a marked reduction in LL. Very marginal increase was recorded for PL with time suggesting that this reaction is of an immediate nature resulting from the addition of the more plastic lime material to the soil material.

5. The Agbede laterite can be modified with 3% hydrated lime by weight of dry soil to meet the HRB limits for materials that can be stabilized with cement. If, however, the passing No 200 B.S. sieve criterion is disregarded for the plasticity criterion (very marginal reduction was recorded for the former), 2% lime by weight of dry soil can be given a trial in the field especially for A-7-6 laterite containing lesser amount of clay. If also the passing No. 200 B.S. sieve is disregarded for plasticity with respect to time acceptable modification could be said to have been achieved after 2 hours for 3% lime when the LL = 39% and PI = 18%, and 4 hours for 2% lime when the LL = 39.5% and PI = 16.5%.

6. Contrary to expectations, using the Nigerian General Specification standards, strength gain is inadequate for soil-cement as well as for soil-lime-cement using the Agbede laterite. It is therefore uneconomical to stabilize the Agbede laterite using cement as a single stabilizer or with lime as an admixture. If however, the UCS criterion is used (as against the CBR criterion

of the Nigerian General Specification) and specifying a lower value of say 1.38N/mm^2(200 lb/in^2) a soil-lime-cement mixture with 2% lime and 6% cement will be adequate judging from the reasonable R_i (resistance to loss of strength) value of 76% i.e. 24% loss in strength, recorded after soaking for 7 days (Table 6) taking into account the fact that the usual 20% max allowable loss in strength is for a 4-day soaking period.

7. The type of clay mineral and amount of clay in a soil affect the performance of a stabilized soil. The low strength values recorded may have been due to the high clay content of the natural soils as well as the presence of hydrated halloysite and illite in the soil. Better results for soil-cement were obtained for A-7-6 laterite with lower clay content and a predominance of the Kaolinite clay mineral (Ola, 1978b).

8. The cementitious reaction in soil-lime-cement mixtures is similar to that of lime stabilization i.e. reaction is more of a pozzolanic nature because of the dominant action of the increased lime content from admixed lime.

9. The usual linear relationship between UCS and CBR for soil-cement is confirmed. This relationship is however not linear but somewhat parabolic for soil-lime-cement mixtures for the range of lime and cement contents considered.

10. The very friable nature of the modified-soil even with 1% lime coupled with the generally good workability of the soil-lime-cement mixtures as well as the general increase in the

UCS suggests that a modification optimum does not have to be obtained before determining the lime and cement proportions for making soil-lime--cement mixtures especially from the relatively better performance of the soil-lime--cement mixtures of 2% lime than 3% lime. Considering, therefore, the pozzolanic nature of this soil-cement mixture (see 7 above), and using only the plasticity criterion if need be (see 2 above) for modification, there is very little or no benefit in using more than 2% lime if the ultimate purpose is that of stabilization with cement.

11. From the point of view of stability (using UCS value of 1.38 N/mm^2 (200 lb/in^2) and durability (using R_i value of 75% which is equivalent to 25% loss in strength after 7 days soaking) soil-lime--cement mixture with 2% lime and 6% cement will be adequate as a pavement material. A CBR of 90% is recommended for this form of stabilization.

12. Modification reaction is essentially due to cation exchange reactions. Modification reaction time may not affect strength gain for soil-lime-cement mixtures.

13. Strength gain for soil-lime-cement mixtures is due partly to interparticle bonding and partly to density.

REFERENCES

Aaron, H. (1948), "Report of Committee on Lime Stabilization". American Road Build. Assoc. Tech. Bull., 147

Ackroyd, L.W. (1963). "The correlation between engineering and pedological soil classification systems in Western Nigeria and its application", Proc. 3rd Reg. Conf. for Africa on Soil Mech. and Foundn. Engng. Salisbury, South Africa.

Ackroyd, L.W. (1967). "Formation and properties of concretionary and non-concretionary soils in Western Nigeria," Proc. 4th Reg. Conf. for Africa on Soil Mech. and Foundn. Engng. Cape Town, South Africa.

Alexander, L.T. and Cady, J.G. (1962). "Genesis and hardening of laterite in soils." U.S. Dept. Agric. Tech. Bull. 1282:90pp.

ASTM, STP 479 (1970). "Special procedures for testing soil and rock for engineering purposes", American Society for Testing and Materials Phil. U.SA.

Andrew, W.P. (1960). "Soil cement roads", Concrete and Cement Association. Contractors Record. London. 20pp.

Areola, O. (1982). "Soils" in Barbour, K.M. (ed) (1982) Nigeria in Maps. Hodder and Stoughton, London.

Arulanandan, K. and Tunbridge, M.D (1969). "The Performance of soil-cement roads in Ghana". Spec. Sess. Lateritic soils. Proc. 7th Int. Conf. on soil mech. and Foundn. Engn. Mexico.

Berman, S. (1963). "Lime tames wet clay for early spring start-Dan Ryan Super highway, Chicago", Roads and Streets. 106: 32-35.

B.S.I : C.P. 2003 (1959). "Earthworks" British Standards Institution, London.

BSI: B.S. 1377. (1975). "Methods of testing soils for Civil Engineering purposes", British Standards Institution, London.

BSI: B.S. 1924. (1975). "Methods of test of stabilized soil", British Standards Institution, London

Clare, K.E. (1960). "Factors affecting the design of stabilised soils" in RRL (1960) Soil Stabilization in the tropics. Road Research Laboratory. Dept. of Scientific and Industrial Research. Symposium on soil stabilization, Nairobi.

Clare, K.E. and Cruchley, A.E. (1957). "Laboratory experiments on stabilization of clays with hydrated lime", Geotechnique. 7:97

Correia, J.A. (1967) "Some results of chemical analysis of laterites", Proc. 4th Reg. Conf. for Africa on Soil Mech. and Fourdn. Engng. Cape Town, South Africa: 57-60.

Croft, J.B. (1967). "The influence of soil mineralogical composition on cement stabilization", Engng. Geol. 17: 119-135.

De Carvalho, G. (1967). "The genesis of laterites from the geological point of view", Proc. 4th Reg. Conf. for Africa on Soil Mech. and Foundn. Engng. Cape Town, South Africa: 61-64

De Graft-Johnson, J.W.S. (1975). "Laterite soils in road construction", Proc. 6th Regional Conf. for Africa on Soil Mech. and Foundn. Engng. Durham South Africa. 1:89-98.

De Graft-Johnson, J.W.S., Bhatia, H.S. and Gidigasu, M.D. (1969). "The engineering characteristics of laterite gravels of Ghana", Proc. Spec. Sess. Engineering Properties of lateritic soils. 7th Int. Conf. on soil mech. and Foundn. Engng. Mexico. 1: 117-128.

D'Hoore J.L. (1964). "Soil Map of Africa. Explanatory Notes", Commission for Technical Cooperation in Africa (CCTA). Lagos.

Dept. of Transport. (1976). "Specification for Road and Bridge Works", H.M.S.O. London.

De Wet, J.J. and Skinner, D.G. (1967). "A fundamental study of some clay lime reactions", Proc. 4th Reg. Conf. for Africa on Soil Mech. and Foundn. Engng. Cape Town, South Africa. : 277-281.

Diamond, S. and Kinter, E.B. (1966). "Mechanisms of Soil-lime stabilization", Public Roads 33:260-265.

Otteh-Iro and Partners, (1970-1982). "Soils and materials Survey Reports for numerous road jobs in Nigeria. (unpublished).

Federal Ministry of Works. (1970). "General Specification - Roads and Bridges", Fed. Govt. of Nigeria. Lagos.

Wossberg, P.E. (1965). "Some fundamental engineering properties of lime stabilized clay", Proc. 6th Int. Conf. on soil mech and foundn. engineering. Canada. 1:221-225.

Gidigasu, M.D. (1971). "The importance of soil genesis in the engineering classification on Ghana soils", Engng. Geol. 5: 17-161.

Gidigasu, M.D. (1973). "Review of identification of problem laterite soils in Highway Engineering", Transp. Res. Board. Rec. 497 : 96 - 111

Gidigasu, M.D. (1974). "The degree of weathering in the evaluation of laterite materials for engineering purposes", Engng. Geol. 8:2 13-216.

Gidigasu, M.D. (1976). "Laterite Soil Engineering", Elserier Scientific Publishing Co. New York. 554pp.

Gilliot, E.E. (1968). "Clay in Engineering Geology" Elsevier Publishing Co. London. 296 pp.

Gilliot, J.E. and Sereda, P.J. (1966). "Strain crystals detected by X-ray". Nature. 209 (5018)

Hilt, C.H. and Davidson, D.T. (1960). "Lime fixation in clayey soils", Highway Res. Board, Bull. 262:20-32

HRB. (1943) "Use of soil-cement mixtures for base courses", Wartime Road Problems No. 7 Highway Res. Board.

Ingles, O.G. and Metcalf, J.B. (1972). "Soil stabilization - Principles and Practice". Butterworths, Sydney. 374pp.

Joachin, A.W.R. and Kandiah, S. (1941). "The composition of some local soil concretions and clays", Trop. Agric. 96: 67-75.

Kedzi, A. (1979) "Stabilized Earth Roads" Elsevier Scientific Publishing Co. Amsterdam. 327pp.

Lambe, T.W. (1957). "Soil stabilization research at the Massachauset Institute of Technology", Proc. 4th. Int. Conf. on Soil Mech. and Foundn. Engng. London: 139 - 144.

Lambe, T.W. (1962). "Soil Stabilization" in Leonards, G.A. (ed) (1962) Foundation Engineering. McGraw Hill, New York. 351 - 437 (1136pp).

Lambe, T.W., Michaels, A.S. and Moh. Z.C. (1960). "Improvement of soil-cement with alkali metal compounds", Highway Res. Board, Bulletin 241

Lees, G., Abdelkader, M.O. and Hamdani, S.K. (1982). "Effect of the clay fraction on some mechanical properties of lime-soil mixtures", J. Inst. of Highway Engrs. 29 (11): 2-9.

Little, A.L. (1969). "General Report: Definition, Formation and Classification". Proc. Spec. Sess. Engineering Properties of lateritic soils. 7th. Int. Conf. on Soil Mech. and Foundn. Engng. Mexico.

Lohnes, R.A. and Demirel, T. (1973). "Strength and Structure of Laterite and lateritic soils", Engng. Geol. 7:13-33.

Lohnes, R.A., Fish, R.L., and Demirel, T. (1971). "Geotechnical properties of selected Pueto Rican Soils in relation to climate and parent rock", Geol. Soc. American. Bull. 82 : 2617-2624.

Lund, O.L. and Ramsey, W.J. (1959). "Experimental lime Stabilization in Nebraska", Highway Res. Board. Bull. 231: 24-59.

Lyons Associates. (1971). "Laterite and lateritic soils and other problem soils in Africa", An Engineering study for Agency for Int. Dev. AID/CSD - 2164, Lyons Associates/ BRRI, Kumasi, Ghana.

Maclean, D.J. (1953). "Investigation of some problems on soil stabilization", Proc. 3rd. Int. Conf. on Soil Mech. and Foundn. Engng. Switzerland. 263-267.

Madu, R.M. (1977). "An investigation into the geotechnical and engineering properties of some laterites in Eastern Nigeria", Engng. Geol. 11: 101-125.

RASHMI

Maignien, R. (1966). "Review of research on laterites", UNESCO Natural Resources Research IV. Paris. 148pp.

Malomo, S. (1983). "Weathering and weathering products of Nigerian rocks, engineering implications" in Ola, S.A. (ed) (1983). Tropical soils of Nigeria in Engineering Practice. Balkema, Rotterdam. 39-60 (326pp).

Mateos, M. (1964). "Soil lime research at Iowa State University", Proc. ASCE. J. Soil Mech. and Foundn Engng. Div. 90(SM2): 127-153.

Matéos, M. (1965). "Research on the stabilization of soils with lime and flyash", Proc. 6th Int. Conf. on Soil Mech. and Foundn. Engng. Canada. 2:132-136.

Metcalf, J.B. (1966). "Some comments on the design, testing and specification of stabilized soils for highway construction", Commonwealth Scientific and Industrial Research Organisation, Commwealth of Australia. 7pp.

Millard, M.S. (1960). "The need for soil stabilization" in RRL (1960) Soil stabilization in the tropics Road Research Laboratory Dept. of Scientific and Industrila Research. Symposium on soil Stabilization, Nairobi.

Millard, M.S. (1962). "Road building in the tropics", J. Appl. Chem. London. 12: 342-357.

Moh, Z.C. (1962). "Soil stabilization with cement and sodium additives", Proc. ASCE. J. Soil Mech. and Foundn. Engng. Div. 88 (SM6): 81-105

Moh, Z.C. and Mazhar, F.M. (1969). "Effect of method of preparation on index properties of lateritic soils". Proc. Spec. Sess. on Engineering Properties of lateritic soils. 7th Int. Conf. on Soil Mech. and Foundn. Engng. Mexico. 1: 23-25.

Novais-Ferreira, H. (1965). "The hardness of lateritic concretions and its influence in the performance of soil mechanics tests", 6th Int. Conf. on Soil Mech. and Foundn. Engng. Canada. 1: 82-86.

Obi, B.C.A. (1974). "Cement stabilized bases save money", Highway Design and Construction: 18-21

Odier, M.L., dos Santos, M.P., Lewis, P.C. and Millard, R.S. (1960). "Report on Role of Stabilization", Symposium on soil stabilization, Commission for Technical Cooperation in Africa South of Sahara (CCTA). Nairobi, Publication No. 65: 15-16.

O'Flaherty, C.A. (1974). "Highway Engineering Vol. 2", Edward Arnold London.

Ola, S.A. (1974). "Need for estimated cement requirements for stabilizating laterite soils", Proc. ASCE. J. Transp. Engng. Div. 100(TE2). 1053. 379-388.

Ola, S.A. (1977a). "Lime deposits and small-scale production of lime in Nigeria", Engng. Geol. 11: 127-137.

Ola, S.A. (1977b). "The potentials of lime stabilization of lateritic soils", Engng. Geol. 11.305-317.

Ola, S.A. (1978a). "The geology and geotechnical properties of Black Cotton soils of Northeastern Nigeria", Engng. Geol. 12: 375-391.

Ola, S.A. (1978b). "Geotechnical properties and behaviour of some stabilized Nigerian lateritic soils", Engng. Geol. 12: 145-160.

Ola S.A. (1980). "Mineralogical properties of some Nigerian residual soils in relation with building problems", Engng. Geol. 115 : 1-13.

Persons, B.S. (1970). "Laterite, Genesis, location and use". Plenum Press. New York. 103pp.

Portland cement Association. (1959). "Soil Cement Laboratory Handbook". Chicago.

Rahaman, M.A. and Malomo, S. (1983). "Sedimentary and crystalline rocks of Nigeria" in Ola, S.A. (ed) (1983) Tropical soils of Nigeria in Engineering Practice. Balkema. Rotterdam: 17-38 (326pp).

Rastall, R.H. (1968). "Textbook of Geology", Edward Arnold, London.

Road Research Laboratory. Road Note No. 29. (1970). "A guide to the Structural Design of Pavements for New Roads". Research Laboratory. Dept of the Environment. H.M.S.O. Bot. London.

Road Research Laboratory. (1952). "Soil Mechanics for Road Engineers", H.M.S.O. London

Sabry, M.A.M. and Parcher, J.V. (1979). "Engineering properties of soil-lime mixes", Proc. ASCE J. Transp. Engrg. Div. 105 (TE1). 14276 : 59-70.

Sherwood, J.N. and Sienkiela, E.J. (1972). "Lime reclaims unstable interior subgrade during construction", Civil Eng. 42: 74-75.

Sherwood, P.T. and Corell, C.S. (1959). "The effect of hydrated lime and Calquim Chloride on the Strength/age relation for cement - stabilized organic soils", RN/3436/PTS, GSC. Dept. of Scientific and Industrial Research, Road Research Laboratory.

Smith, G.N. (1978). "Elements of Soil Mechanics for Civil and Mining Engineers. Granada Publishers, London.

Terzaghi, K. (1958). "Design and performance of Sasumua Dam", Proc. Inst. Civil Eng. London. 9:369-394.

Thompson, M.R. (1965). "Influence of soil properties on lime-soil reactions", Pub. Works. 96:120-123

Thompson, M.R. (1966). "Lime reactivity of Illinois soils", Proc. ASCE. J Div. of Soil Mech.96 : 120 -123

Thompson, M.R. (1968). "Lime-treated soils for pavement construction". Proc. ASCE. J. of Hwy. Div. 94(HW2) 6249: 191-217.

Tomlinson, M.J. (1975). "Foundation Design and Construction", Pitman Publishing Ltd., 2ed. London. 768pp.

Tuncer, E.R. and Lohnes, R.A. (1977). "An engineering classification for certain Basalt derived lateritic soils", Engng. Geol. 11: 319-339

Turnbull, W.J. and Foster, C.R. (1956). "Stabilization of materials by compaction", Proc. ASCE. 82

Vallerga, B.A., Schuster, J.A., Love, A.L. and VanTil, L.J. (1969). "Engineering Study of laterite and lateritic soils in connection with construction of roads, highways and airfields", U.S. Agency Int. Dev. Report AID/CSD 1810: 165pp.

Wang, J.W.H, Mateos, M. and Davidson, D.T. (1963). "Comparative effects of hydraulic calcitic and dolomitic limes and cement in soil stabilization", Highway Res Rec. 29 42-54.

Wooltorton, F.L.D. (1975). "Foreword" of : Laterite Soil Engineering by Gidigasu, M.D. (1976). Elservier Scientific Publishing Co. New York. VII-XVII.

Yoder, E.J. and Witzack, M.W. (1975). "Principles of Pavement Design", John Wiley. New York. 2ed. 711pp.