PROPERTIES AND CHARACTERISTICS OF LIME TREATED LATERITE SOILS

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CERTIFICATION

This is to certify that this research work was carried out and written by **Faluyi, Sunday Olubunmi**.

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21-08-2008
Date

DEDICATION

This project is dedicated to the only one who lay down His life for the sins of the whole world - **OUR LORD JESUS CHRIST.**

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ABSTRACT

The construction of rural and low traffic roads to meet the present day and future requirements is the greatest challenge for transportation engineers in the developing countries. The greatest handicap for the construction of good road network and the improvement of village roads is the scarcity of suitable local construction materials coupled with inadequate funds.

Construction cost can be reduced through stabilization. One of the methods is by lime treatment. In this study, the influence of the physical characteristics of five selected laterite samples on their engineering behaviour in the natural untreated state and on their responsiveness to treatment with lime as chemical stabilizing agent was determined in the laboratory. The percentages of lime used on the laterite samples varied from 2% to 10%.

Tests on both the cured and uncured mixture produced increase in CBR value, unconfined compressive strength, compressive strength, shear strength and failure stress with increasing lime content. The rate of increase depends on the soil type. The optimum moisture content, the plastic limit and the liquid limit of the mixture increase with increasing lime content while the maximum dry density, plasticity index, ultimate strain, linear shrinkage and swell decrease with increasing lime content.

Beneficial results were obtained using lime additive for treating all the samples especially the highly plastic and organic soil types such as sample type A, which in its untreated condition can be often detrimental for use as subbase material for road construction as a result of differential expansion and shrinkage when it undergoes changes in moisture

content. Lime treatment of the soil sample reduces the change in volume when it undergoes changes in moisture content and improves strength characteristics which make the treated soil suitable for use as subbase material.

The increase in strength of treated soil samples C and D make them suitable for use as base material. The results also show that the minimum lime content for treatment of the tested lateritic soils for use as either subbase or base material in pavement construction is 6%.

TABLE OF CONTENTS

CONTENT **PAGE**

CONTENT PAGE

CONTENT **PAGE**

LIST OF TABLES

TABLE **PAGE**

LIST OF FIGURES

LIST OF SYMBOLS AND ABBREVIATIONS

CBR	-	California Bearing Ratio
SiO_2	-	Silica
Al_2O_3	-	Aluminium Sesquioxide
Fe_2O_3	-	Iron Sesquioxide
$CaCO_3$	-	Calcium Carbonate
$MgCO_3$	-	Magnesium Carbonate
CaO	-	Calcium Oxide
$Ca(OH)_2$	-	Calcium Hydroxide
MgO	-	MagnesiumOxide
H_2O	-	Water
UCS	-	Unconfined Compressive Strength
G.I	-	Group Index
LL	-	Liquid Limit
PL	-	Plastic Limit
PI	-	Plasticity Index
AASHTO	-	American Association of State Highway and Transportation Officials
Cu	-	Coefficient of Unformity
LOI	-	Loss On Ignition

U	-	Undetermined
MDD	-	Maximum Dry Density
OMC	-	Optinum Moisture Content
c	-	Cohesion
Ø	-	Angle of Friction
SL	-	Shrinkage Limit

CHAPTER ONE

INTRODUCTION

1.1 PREAMBLE

Good supplies of suitable inexpensive construction materials are a scarce and fast diminishing resource for most countries. The construction of village roads and roads carrying daily traffic less than 200 vehicles to meet the present day and future requirements is the greatest challenging task for transportation engineers in developing countries.

A large percentage of the roads in developing countries are low volume traffic roads. There should be the feasibility of providing low cost construction for low volume roads. These low cost roads when constructed should be able to satisfy the present traffic needs with suitable geometric standards at the minimum cost and at the same time provide a basis on which roads of higher specifications can be built later when traffic on these roads increase without changing their alignments.

In order to meet most of these requirements, soil stabilized roads are constructed. The aim of stabilising soil is to alter its properties to improve its engineering performance. The effective utilization of locally available soils and other construction materials and stabilizers would ensure cost minimization of construction to the maximum extent.

There are various ways of stabilizing soils; the techniques can be broadly classified as follows:

i. Mechanical methods of stabilization
ii. Electrical methods of stabilization
iii. Thermal methods of stabilization
iv. Drainage methods of stabilization

v. Pre-compression methods of stabilization

vi. Chemical methods of stabilization.

The degree of stabilization attained may differ within a given method and between the different methods. Since soils exist in a broad range of types differing markedly in the values of their various properties and since different soils react differently to the methods, it is evident that there exists not only several types but also a wide range in the degree of stabilization.

The common chemical methods of stabilization are lime, cement and bitumen. The stabilizer used in this project is lime. Lime stabilization of soils is among the oldest techniques for road construction, dating back to the Roman highways several hundreds years before Christ. For the Roman roads, lack of compaction equipment necessitated the placement of soil lime as mortar.

In modern practice, the lime and water contents are greatly reduced, and the moist mix is spread and compacted with rollers. It is then covered with a base course or a bituminous wearing surface.

Initial work by earlier investigators **Maignien (1966). Little (1969)** and **Gidigasu (1976)** showed that laterite soils which are tropical soils responded more favourably to lime stabilization than temperate zone soils. The reasons deduced by these investigators for the differences in the tropical soils and temperate soils are mainly the chemical composition of the soils and the prevailing climatic conditions.

Laterite soil is a residual soil formed from the weathering of igneous rock under conditions of high temperature and high rainfall of the tropical region where the

decomposition process results in a soil leached of silica (SiO_2) and calcium carbonate ($CaCO_3$) but retaining high concentration of iron and aluminium sesquioxides (Fe_2O_3, Al_2O_3). It is red, reddish brown or dark brown in colour.

Laterite soils are the local types of soil in Nigeria. Since lime stabilization has been most widely and successfully used in United States for road construction, greater interest should be developed towards the use of lime stabilized laterite soils for road construction in Nigeria.

Various forms of lime have been successfully utilized as a soil stabilizer for many years. However the most commonly used lime are hydrated high calcium lime ($Ca(OH)_2$), monohydrated dolomitic quicklime (CaO. MgO), calcitic quicklime (CaO), and dolomitic quicklime (CaO.MgO).

Many significant engineering properties of soils are beneficially modified by lime treatment. Although lime is primarily utilized to treat clayey soils, it can also be used to modify the characteristics of the fine fraction of more granular soils.

The position of lime stabilized soil layer in the pavement thickness is controlled by the quality of the lime stabilized soil and other pavement design considerations. Railway subgrades have also been successfully stabilised with lime.

Lime stabilized soil may be used as subbase or base course for roads, airfields, parking lots, highway shoulders, village roads and similar applications. Lime stabilized lateritic soils are used in embarkment construction for railways, earth dams and levees to enhance the shear strength of the soils. They are also used to support foundations.

When lime stabilized laterite soils are protected from percolating and migrating water

and the effects of heavy repetitive loading, they can be used successfully as an economic alternative to the more expensive and normally scarce construction materials.

1.2 BACKGROUND INFORMATION TO PROPOSED STUDY

Soil treatments with lime is among the oldest technique for road construction, dating back to the Roman highways several Hundred years B.C. In the United State, lime stabilization has been most widely used in Texas, but is gaining in use throughout the United State and in various parts of the world (**Woods, 1960**). In Zambia, 3% of hydrated lime has been used extensively to stabilize the clayey gravels which are widely available. (**Ashworth, 1979**). In Nigeria, the use of stabilizing agent in road construction has been in practice for some thirty years (**Anifowose, 1989**).

The use of lime treated laterite soils has been investigated before but their effectiveness when in use has not been investigated to rather conclusive ends. Also there is limited information on the performance of lime treated laterite soils in Nigeria because of limited literature on the usage and its characteristics. This present research is an attempt to fill the gaps pointed out above.

1.3 OBJECTIVES

The objectives of this study are:-

i. To determine the general engineering properties of the laterite soils taken from a number of road construction borrow pits in Ado-Ekiti area.

ii. To determine the mineralogical composition of these soils for establishing a relationship between the composition and performance of lime stabilization

iii. To determine the effects of lime stabilization on the engineering properties of the

soils.

iv. To establish the minimum lime content required, in achieving maximum utilization of these local material which are available for road construction in Ado-Ekiti environs.

1.4 JUSTIFICATION FOR THE STUDY

Laterite soil is a variable and complex material which, because of its availability and low cost, it is frequently used for construction purpose. It possesses a range of mineralogical composition and properties which makes its performance unpredictable. At a particular location, however, a laterite soil may not be wholly suitable for the desired purpose. In such a case, its engineering properties may be altered by the addition of a small amount, by weight of lime. The amenability of the soil to such treatment depends not only on the type and amount of cementatious material added but also on the chemical and mineralogical composition of the soil.

In order to provide a reliable technical information on stabilization of soils treated with lime in Ado-Ekiti environs and similar locations, it is necessary to investigate the treatment of laterite soils with lime and determine their engineering properties. The technical information will allow for comparison with soils treated with cement which is hitherto a popular method of stabilization. Results of this study will provide useful guidelines for highway engineers in material selection for road construction especially in our own tropical environment.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Considerable work has been carried out on lateritic soils in many different countries over the years. The attention given to lateritic soils by soil engineers is as a result of their wide-spread distribution and engineering application. The word laterite was originally suggested by **Buchanan (1807)** as a name for a highly ferruginous deposit first observed in Malabar during his journey through the countries of Mysore, Canava and Malabar in 1800-1801. Laterites have been described as soils which were developed together with red and yellow soils under optimal moisture contents. They result from changes which occur in zones of permanent and intermittent saturation in the vicinity of a water table. **Martin and Doyne (1930)** works on the chemical composition of lateritic soils defined laterite soils in terms of the silica alumina ratio (SiO_2:Al_2O_3) as

i. True laterites (those with silica - alumina ratio less than 1.33)

ii. Lateritic soils (those with ratio between 1.33 and 2.00)

iii. Non-lateritic with a ratio greater than 2.00

Overseas Unit of the Transport and Road Research Laboratory (1952) defined laterite as a tropical soil in which the weathering processes have resulted in an accumulation of sesquioxides particularly iron. **Little (1969)** defined laterites as in-situ products of tropically weathered igneous rock which has decomposed partially or totally with a concentration of iron or aluminium sesquioxides at the expense of silica. **Vallerge, *et al*, (1969)** gave what seems the most generally accepted definition of laterite as a hardened material formed by the primary weathering of residual soil in which sesquioxides are present in parent rock or by

the secondary leaching cementation of transported or residual soil in which the sesquioxides are supplied by lateral ground water movement. A good number of studies of several investigators revealed that laterite is a weathered product of various materials including crystalline igneous rock sediments and volcanic ash.

From the various definitions above, it can be concluded that lateritic soils contain aluminium, iron and silica in varied proportions and based on the constituent components of laterite.

2.2 LATERITES

2.2.1 Origin And Formation of Laterites

Laterite is a residual soil formed from the weathering of igneous rock under conditions of high temperature and high rainfall such as those typically occurring in tropical regions, where the decomposition process results in a soil leached of silica (SiO_2) and calcium carbonate but retaining high concentration of iron and aluminium sesquioxides (Fe_2O_3, Al_2O_3). The resulting concentration of iron and aluminium oxides sharply differentiate laterization from temperate climate weathering in which the end product is largely clay minerals. As a result of heavy rainfall during the wet season, followed by the dry seasonal high temperature and rapid evaporation in the tropics, plants draw water far below the ground to replenish the ones lost to the atmosphere, as long as the supply holds. The weak solution produced by leaching of the rocks during the wet season thus become concentrated by evaporation and dissolved minerals are deposited. This leads to the accumulation of iron and aluminium hydroxides which are insoluble and are not re-dissolved during the raining season and are therefore left at or near the surface where they form a thick

red to blackish soil called laterite.

Maignien (1966) and **Gidigasu (1976)**, identified three major stages in the process of laterization. The first stage, (decomposition) is characterized by physico-chemical breakdown of primary minerals and the release of consistituent elements. The second stage involves the leaching under appropriate drainage conditions of combined silica and bases and the relative accumulation or enrichment from outside sources. Absolute accumulation of oxides and hydroxides of sesquioxides (mainly Al_2O_3 and Fe_2O_3, the most resistant components to leaching). The third stage (dehydration or desiccation) involves partial or complete dehydration sometimes involving hardening of the sesquioxide rich materials and secondary minerals.

Under tropical conditions of high temperature and rainfall, the clay minerals tend to decompose into various forms of aluminious and iron oxides in relation to the nature of weathering conditions. The free iron oxide content and the degree of dehydration and/or hardening as well as the unique granular structure of some lateritic soils are believed to underlie the difference in engineering behaviour of some lateritic soils from the temperate soils (**Gidigasu, 1976**).

Maignien (1966), suggested that contemporary lateritic soils have developed at mean annual temperatures of around 25°C. Laterite soils are believed to always correspond to tropical climate.

Little (1969) suggested that laterization effects are produced due to bacterial metabolism which are sufficiently stimulated in the high temperatures of the tropics.

The most significant differences in the temperate soils and lateritic soils are:

i. The significant proportions of iron and aluminium oxides found in lateritic soils which tend to cement the soil particles to form a coarse grained weakly bonded particulate material and

ii. The relatively high temperature of laterite formation which is a feature of tropical climate.

2.2.2 Distribution and Morphology of Laterites

Laterites are found all over the world especially in inter-tropical regions of Africa, Australia, South-East Asia, India and South America. Their extension show that their formation is favoured by some conditions at one time or the other in the history of the world but not necessarily simultaneous at all points.

McCarthy (1993), said that deposits of laterites are found in a hard or cemented state particularly in areas where vegetation is sparse or has been removed. **Prescot and Pendleton (1952)** concluded that laterites are widely distributed in the semi-humid and humid intertropical regions of the world, and laterites, in fossil states are found in dried climates and sometimes in temperate climates. Also as hardened occurrences, laterite extends beyond sub-humid tropical climate to desert regions where there are indications of humidity in the past.

Abundance of laterite gravels and soils are located in the Precambian basement complex area in the northern part of Nigeria. They are found in large quantities in the Southern part of Nigeria and constitute about 50-70% of their total soil. Laterites can be associated with a characteristic profile forming the following level in descending order as

shown in fig 2.1

i. A surface zone with ferruginous incrustation and concreting

ii. A lateritic bed

iii. A zone of primary alteration to kaolinite

iv. A fresh zone of unaltered rock.

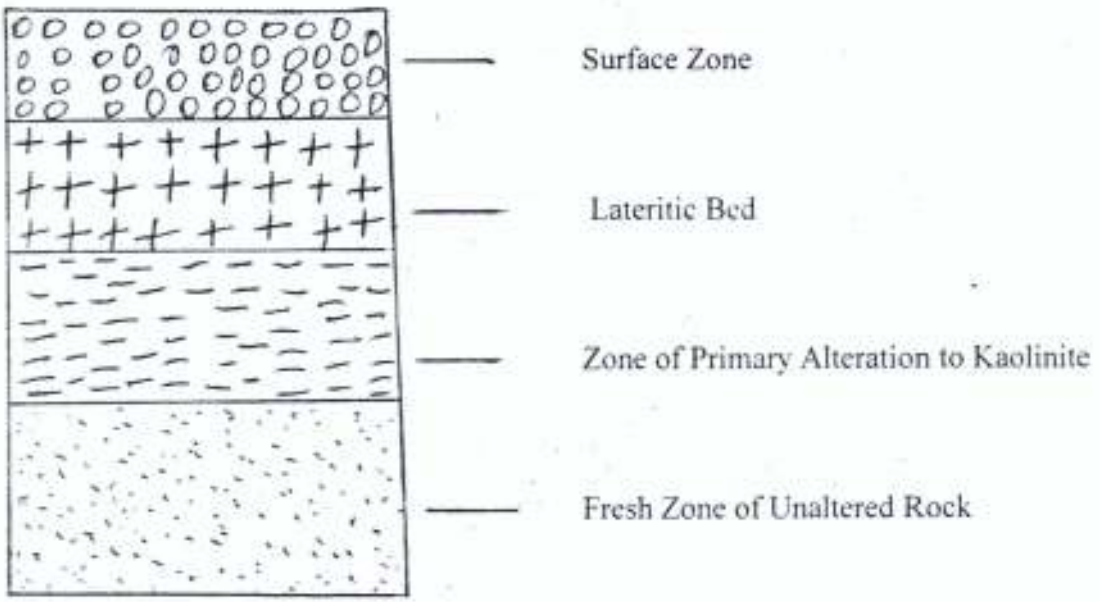


Fig. 2.1: Typical Lateritic Soil Profile

2.2.3 **Properties And Composition of laterite soil.**

The totality of the properties of laterite depends on the properties of the microstructure that makes up the soil. The physical properties of laterite depend on the properties inherited by the soil from the parent rock material, environmental factors and characteristics of the locality in which the soil has been in the course of its geologic history **(Gidigasu, 1976).**

De.Graft - Johnson (1969), suggested that the specific gravity of laterite falls within

a wide range of 2.2 to 2.8. He also observed that the higher the degree of laterization, the greater the specific gravity. Specific gravity of laterites vary in relation to their chemical composition, increasing with iron content and decreasing with alumina; the coarser the soil fraction, the higher the specific gravity. The looser the soil structure, the lower the specific gravity. The specific gravity of oxidized forms is higher than that of hydrated forms.

Lateritic soils seem to be typified by a gap grading existing with a prevalence of gravel sized and clay materials but having limited sand and silt-sized particles.

The pH value of laterite is dependent on the exchangeable cation of the soil. The cation type that can exist in a particular soil depends on.

i. Mineralogy of the parent material

ii. Organic proportion present in the soil.

Kaolinite, the predominate mineral in Nigeria lateritic soils has low cation exchangeable cation and the pH value does not deviate much from 7. Soils that have high cation exchangeable cation have their pH value in the alkaline range. Most laterite soils have sodium as exchangeable cation and the pH value is the slightly acidic to neutral (5-7).

The mechanical grain sizes of the lateritic soil will affect its reaction under load. If the laterite is more of sandy soil, its properties will exhibit low compressibility, high permeability high friction and low cohesion but if it is more of clay soils, its properties will exhibit high compressibility, low permeability, high cohesion and low friction.

Laterites generally have granular structure: therefore they have fairly large internal angle of friction. The higher the angle of friction, the higher the strength of the soil.

Another property of laterites is the tendency of expansiveness or compressiveness of

the soils depending on the specific surface area of the consistuent particles. Kaolinites that have low specific surface area have low expansiveness or compressiveness but montmorillonites that have high specific surface area have high expansiveness or compressiveness.

The mineralogical composition of most laterites are kaolinite, geothite, gibbsite, heamatite, microcline, illite, halloysite, attapulgite and quartz.

2.2.4 Economic Importance of Laterite

Laterites in their native regions, are of economic necessity. They are used as construction material for road bases and embarkment structures and to support foundations. Indiscriminate use of laterites without fully appreciating their limitations, however, has resulted in the failure of a large number of road pavements in this country. Based on the engineering properties of the laterite, they are also used for roofing local buildings, brick making, making local ovens for baking, and construction of local silos for storage purposes. In their natural state, they are good as subgrade material for airfield and highway pavements. When protected from percolating or migrating water and the effects of heavy repetitive loading, they can be used successfully.

2.3 TYPES OF LIME

Lime is produced when limestone is calcined. There are two types of limestone the pure limestone and the impure limestone. The calcitic limestone occurs throughout the geologic column but dolomite limestone is common only in older geological deposits, particularly those of the early Paleozic. Dolomite is a result of mineral replacement, and not uncommonly both limestone and dolomite occur within the same bed or in the same quarry.

Watson, (1991), describes lime as a binder which forms a hydrated matrix in which aggregate may be set. **O'flaherty, (1975),** defines lime as calcium oxide which occurs in practice as the oxides and hydroxides of calcium and calcium magnesium.

Merritt and Kurtz, (1983) and **Hausmann, (1990)**, in their own contribution to the availability of lime and its commercial production stated that lime is available in practice as calcium oxide (CaO) known as quicklime which is produced commercially by burning limestone in kilns, as hydrated lime ($Ca(OH)_2$) when calcium oxide is mixed with water which reacts violently with the evolution of much heat, and as quick lime which is produced commercially as less aggressive lime.

Pure lime ($CaCO_3$) will produce a high calcium or calcitic lime or fat lime. If the stone being calcined is a dolomitic limestone containing a high proportion of magnesium carbonate ($MgCO_3$), the lime produced is dolomitic lime or high magnesium carbonate lime. High - calcium lime contains less than about 5% magnesium oxide while most dolomitic quicklimes contain between 25% and 45% magnesium oxide (MgO).

Impure limestone which contains less than 15% clay content, when calcined, the lime produced is called a semi-hydraulic lime while the one that has clay content greater than 15% and up to 35% is termed hydraulic lime. They may either be calcitic or dolomitic in nature and may or may not be hydrated.

During the normal calcining of argillaceous and siliceous limestones (impure limestones) the calcium carbonate is caused to dissociate by the heat following which solid reactions take place between the calcium oxide which is formed and the silica, alumina and ferric oxide components of the clay. This leads primarily to the formation of the dicalcium

silicate and tricalcium aluminate compounds which lead to the formation of the hydraulic and semi hydraulic limes. They have cement like properties like setting under water while retaining the plastic properties of lime. Quick lime is normally produced by direct calcination of crushed limestones with the following reactions taking place:

$CaCO_3$ + heat → $CaO + CO_2$

At atmospheric pressure and at a heat of 900°C, the lime product of the above reaction is termed calcitic or high calcium quicklime. For dolomitic quicklime at atmospheric pressure and at a heat of 730°C, the dolomitic limestone decomposes to form magnesium oxide, carbon dioxide and calcium carbonate. But at 900°C the calcium carbonate ($CaCo_3$) decomposes to produce calcium oxide and the end result, MgO + CaO is called dolomitic quicklime.

Hydrated limes are produced by slaking quick lime with an excess amount of water to lime slurry of varying degrees of consistency; when quicklime is hydrated, the product is a dry powdered lime known as calcium hydroxide $Ca(OH)_2$.

Cao + H_2O → Ca $(OH)_2$ + 65.3KJ/mol.

The transition from quicklime to hydrated lime is characterized by the following index properties.

Table 2.1 Index properties of transition from quicklime to hydrated lime

	Quicklime	Water	Hydrated lime
Molecular weight	56	18	78
Specific gravity	3.3	1	2.2
Relative weight	1	0.32	1.32
Relative Volume	1	0.99	1.99

Ease of slaking of quicklime is greatly affected by both composition and the temperature of burning and quicklimes may be classified as rapid slaking, medium slaking and slow slaking. In general, higher calcination temperatures results in slow slaking. Temperature becomes relatively more important for dolomitic quicklime than for the high calcium variety since $MgCO_3$ has a lower dissociation temperature.

A significant feature of quicklime which retards its usage in soil stabilization is its high exothermic hydration reaction and causticity which can cause destruction to living tissues. Hence particularly in windy weather, construction workers without adequate protective clothing may severely damage their eyes and skin when working with quicklime.

In construction sites, the slaking process typically creates clouds of steam. Slaked lime is used in the form of a fine powder within bulk density of 0.45 to 0.6 t/m^3 or as a slurry with a water content of 80 to 100%. Quicklime is more cost effective than slaked lime in terms of handling and transport.

Rotary kilns produce quicklime in pebble form (6 to 60mm). Whereas vertical kilns produce quicklime in lumps (150 to 200mm). Both types are then ground and sieved prior to slaking or use as a powder.

Table 2.2 Types and grades of lime

Types	Formular
Calcia (high calcium quicklime)	CaO
Hydrated high calcium lime	$Ca(OH)_2$
Dolomitic lime	CaO + MgO
Normal hydrated or Monohydrated dolomitic lime	$Ca(OH)_2$ + MgO
Pressure hydrated or dihydrated dolomitic lime	$Ca(OH)_2 + Mg(OH)_2$

The higher the magnesium content of the quicklime or hydrated lime, the less is the water affinity and the heat developed in mixing with water.

2.4 SOIL STABILIZATION

Winterkorn and Fang (1975) define soil stabilization as the collective term used for any physical, chemical or biological method or any combination of such methods, employed to improve certain properties of a natural soil to make it serve adequately an intended engineering purpose.

Baker, *et al*, (1975) define soil stabilization as applied to pavement construction as a means of treating soils and/or base materials by markedly increasing their strength and bearing capacity and decreasing their water sensitivity, volume change during wet-dry cycles. **Wright and Paquette, (1987)** define soil stabilization as the combination and manipulation of soils with or without admixtures to produce a firm mass that is capable of supporting traffic in all weather conditions restricted to between 60 and 200mm in thickness. The type and degree of stabilization required in any given instance are largely a function of the availability and cost of the required materials as well as the use that is to be made of the

stabilized soil mixture.

The simplest and most cost-effective forms include improvement in soil by compaction and drainage. Another method known as mechanical stabilization involves the blending of different natural materials to obtain a mixture with the desired engineering properties in terms of grading and plasticity. More commonly, soil stabilization is the improvement of natural materials by the addition of a small proportion, usually less than 10% of a stabilizing agent. This may involve the addition of additives such as cement, lime, bitumen, sodium silicate, calcium chloride, industrial waste, fly ash and resins depending on the nature of soil to be stabilized.

The stabilization requirements of a soil thus increases with increasing proximity of the soil structure to the pavement surface, where the normal temperature and moisture variation exhibit greatest amplitude. Also a highway base course of the same soil type and built for the same traffic loads and frequencies has considerably higher stabilization requirement in the more severe climate than in the mild climate.

Cost of construction can also be brought down considerably by selecting locally available soils which, if very poor and cannot withstand the expected traffic wheel loads, can be improved by adopting soil stabilization technique which is a method of processing available material for the development of stable strips of low cost roads. The prime requirements of any stabilizer used in roadway construction are that it should retain or increase the shear resistance of the soil even in the presence of saturating amounts of water and it should increase the volumetric stability of the soil.

The primary aim of treating soils with chemicals is to make soil a suitable alternative to crushed rock or aggregate in roadbases and subbases. Chemical stabilizers act by cementation and/or water proofing action. The cementing type stabilizers actually bond the soil grains together and improve the strength of the soil mass under certain conditions.

2.4.1. **Lime Stabilization**

Treatment of soil with quicklime or hydrated lime is especially effective in improving the engineering properties of heavy clay soils or of granular soils which because of high water affinity in their silt-clay fraction, fall short of having a dependable granular skeleton. Lime stabilized bases and subbases are effective moisture barrier to downward percolation or capillary action. Lime stabilization of clay soils, therefore, is used to provide base courses beneath all types of pavements and to provide surfaces for secondary and tertiary roads **(Hunt, 1986)**.

According to ARBA (1959) construction manual on lime stabilization, the physical changes effected by lime treatment of clay soils can be summarized as follows:

i. The plasticity index drops sharply by a factor of 3 or more in instances

ii. The plastic limit generally increases and the liquid limit decreases.

iii. The soil binder content decreases substantially

iv. The linear shrinkage and swell decrease markedly

v. Lime and water accelerate disintegration of clay clods during pulverization. Soils become friable and can be worked easily.

vi. Unconfined compressive strength increases considerably.

vii. Load bearing values increase substantially

viii. In swampy areas, or soils with over optimum moisture content, the application of lime facilitates drying of the soil.

ix. Lime stabilized bases or subbases form a water-resistant barrier by stopping penetration of gravity water and by rapid evaporation of existing moisture. The stabilized clay sheds water more readily during rain, thereby minimizing construction delays.

Al-Rawi and Awad (1981) indicated that the permeability of a clayey soil is increased when treated with lime as the latter causes flocculation of the soil. The higher the clay fraction, the more the permeability the soil-lime mixture increases. Soils compacted on the dry side of optimum moisture content developed a higher permeability than those compacted wet of optimum. However with increasing age, the permeability declines this being explained by **Brandl (1981)** as due to the increased production of gelatinous reaction products which occupy an increasing amount of void space and eventually harden.

Lime stabilization is limited to plastic soils only as the presence of clay minerals is necessary for the stabilizing process.

Lime is frequently used as an initial pretreatment of soil in preparation for further additions of lime or cement and as a convenient short term means of producing sufficient improvement in a soil to allow support for construction traffic. In most cases, the quality of lime used for stabilization will typically be within the range 2-8% of the dry weight of soil **(Ingles and Metcalf, 1972)**.

4.1.1 Soil and Lime Reaction

The natural clay consists of laminar particles, which slide freely on one another and

whose ability to slide is aided by the lubricating action of free water present in the system and of water molecules which become attached to the surface of the clay particles which gives it its plasticity property. When lime is added to soil, a number of reactions takes place, the short term reactions are hydration (for quicklime) and flocculation (ion exchange) and the long term reactions are cementation and carbonation. **(Hansmann, 1990)**.

Hydration is the first reaction to take place between the free water in clay and the quick lime which slakes the lime and reacts immediately with the clay particles causing drying action which is beneficial in the treatment of moist clays. In the placement of lime columns and layers, the heat generation and expansion of the lime further enhance the consolidation effect.

The next reaction takes place within few hours causing the clay particles to flocculate or aggregate and change the nature of the soil to a relatively coarse-grained, free draining medium of much lower plasticity. A flocculation effect is observed which is reflected in the lowering of liquid limit of the soil, the raising of the plastic limit and shrinkage limit and a lowering of maximum dry unit weight of the soil. The alteration of soil character is due to:

i. A cation exchange reaction which occurs with the bivalent calcium ions supplied by the lime replacing less firmly attached monovalent ions in the double layer surrounding the clay particles. This tends to decrease the thickness of the double layer and to depress the zeta potential (a measure of the effectiveness of the particle negative charges in repelling a second particle), thereby causing flocculation.

ii. The crowding of additional cations on to the clay particles, thereby again depressing the zeta potential.

iii. Pozzolanic reaction is initiated between lime and the clay which contributes to soil aggregation.

These reactions suggest that the modification of a clay soil by the addition of lime is clearly dependent upon the nature of the dominant cation originally adsorbed in the double layer and also upon the type of clay. For example, a sodium dominated montmorrillonite (which has a high cation exchange capacity) will require a relatively high addition of lime to achieve calcium saturation and full potential flocculation effect. In contrast, a kaolinitic clay will require a considerably lesser amount of lime to achieve the full potential effects. Aggregation also reduces the optimum moisture content for compaction. The reduction in the dry density does not initially affect the strength of the soil.

The third reaction is the cementation of the mixture as a result of pozzolanic reaction which takes several years to complete and reach equilibrium state. This reaction involves the lime and the alumina and silica present in the soil in the formation of hydrous calcium aluminates and silicates which are similar to the reaction products of hydrated cement. The cementation process depends on the amount of available silica. Increasing the quantities of lime added will increase strength only up to the point where all silica of the clay is used up. The structure of the stabilized clay was described by **Herzog (1967)**, as an assembly of hard-skinned, lime poor lumps of clay embedded in a lime rich, fine grained soil matrix.

The strength gain due to cementation is generally very much dependent on the amount and type of clay mineral present in the soil. According to **Dunlop (1977)**, a reactive soil will give CBRs in excess of 100 or unconfined compressive strengths higher than 0.86Mpa after lime treatment. The most reactive clays are those containing minerals

belonging to the montmorillonite group, distinguished by a three-layer primary element in their crystalline structure and a high base exchange capacity. The illites, kaolinites and chlorites clays are less reactive.

The fourth reaction is that of carbonation where carbon dioxide (CO_2) from the air and rain water converts the calcium and magnesium oxides and hydroxides back to their respective carbonates ($CaCO_3$ and $MgCO_3$) which are weak cements. The detrimental effects resulting from this out-weighs the minimal strength gain from it. The detrimental effects are more pronounced particularly in industrial areas where the carbon dioxide content of the air is considerably higher than in rural areas.

Carbonation must be prevented for best strengths to develop in a road. Prevention involves care in storage of lime prior to its use, avoidance of undue delays during construction to prevent prolonged exposure to the atmosphere and so compaction should take place as soon as possible and protection of the road base or subbase against infiltrating carbon dioxide **(Woods, 1960 and O' flaherty, 1975).**

The microscopic nature of the reaction was studied by **Kezdi (1979).** In his own contribution he stated that lime addition will increase the pH value of water content in the soil and give rise to increased solubility. The initial stage is started by the dissociation of the admixed $Ca(OH)_2$ into Ca^{2+} and OH^- ions which modifies the electrical surface forces of the clay minerals. The lime assumes an amorphous form and creates a transition to the gel phase. The gel phase is the most important stage from the aspect of strength as the gel substance will connect and cement the mineral grains thus changing the entire pore structure. This gel phase could be clearly discerned through the microscope. The finest soil particles

coagulate and thereby the entire structure and consistency of the soil undergoes a significant change. From the gel the neoliths will then slowly develop some crystals being clearly seen through the microscope creating the neolithic phase which is the most important part of the consolidation process. In practice, development of the carbonate phase could only be observed for samples stored in the air. On the atmospheric effects, spherulitic calcium carbonate will develop over the surface of these mineral particles creating a compact crystalline mortar.

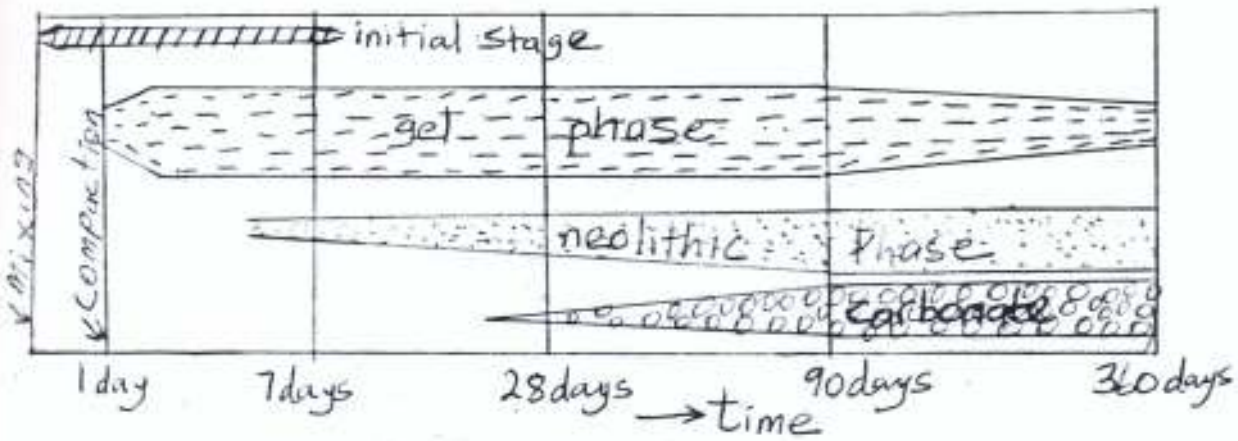


Fig. 2.2 **Development of the individual reaction stages and their participation in changing the soil characteristics**

4.1.2. **Strength Parameters**

The strength parametric effects of stabilization are

i. **Age**

The strength of a lime-soil mixture normally increases with age until all the free lime is used. **(Watson, 1991)**. Over a longer period of time there is strength gain through a pozzolanic reaction with available silica and alumina **(Oglesby, 1975)**. Curing time is another factor that influences the strength of lime-stabilized

soil **(Bell and Conthard, 1990)**, a steady gain in strength over months is characteristic.

ii. **Temperature**

The overall rate of reaction increases with temperature. The rate of increase of strength is more rapid in initial period of curing and higher temperature. Higher temperatures accelerate curing and this gives rise to higher strengths. **(Winterkorn and Fang, 1975)**. Temperature also affects the pozzolanic reaction. Temperatures less than 13 to 16°C retard the reaction and higher temperatures accelerate the reaction **(Anday, 1963)**.

iii. **Curing**

Water is an important reaction agent in soil-lime mixture and the reactions are relatively slow, therefore loss of water must be prevented during the reaction period.

The moisture content at which the soil and lime should be mixed can be very critical, with clayey soils, the optimum moisture content for strength tends to be on the high side of the value for dry unit weight. Optimum strength also varies for different curing periods. The strength of lime stabilized soils is influenced by their natural moisture content, the strength decreases with increasing moisture content, **(O'flaherty, 1975)**.

iv. **Unconfined Compressive Strength and Shear Strength**

The unconfined compressive strength has been extensively used to determine the response of materials to lime stabilization, but it has little direct application to

pavement design.

Dolomitic lime can be more effective than calcitic ones in producing high strengths with average soils. This is due to the ability of unhydrated (MgO) magnesium oxide to act as an accelerator of the pozzolanic reaction. Quicklimes are more effective than hydrated limes in producing strength gain while hydraulic limes generally tend to give higher strengths than non-hydraulic limes. Tests produced increase in compressive strength with lime content until it reached the maximum and then subsequently decreased with lime content **(Ola, 1983)**. Hydrated lime has been used successfully in the past decade to increase the shearing strength of plastic soils at or near the ground surface **(Lindy Jr. and Greenfield, 1968).**

v. **CBR**

The soil lime tests produced increases in CBR with lime content for both soaked and unsoaked samples until it reaches the maximum and then subsequently decreased with lime content. The more coarse the lateritic soil the greater is the CBR value of the mixture **(Ola, 1983).**

vi. **Tensile Strength and Strain**

These properties are of particular significance in the design of lime bound layers which are relatively brittle, with a high modulus and very low tensile strain at fracture. Tensile strength is influenced by particle size distribution, moisture content, lime content and density. Lime stabilized materials fail in tension under low strain. The critical strain usually decreases with increasing modulus.

vii. **Elastic Modulus**

When a lime stabilized material is loaded beyond a certain limit, micro cracking first develops at the inter-face between the coarse particles and the matrix and increases upon subsequent loading, and eventually the microcracks join up, the matrix disintegrates, and the treated material no longer behaves like a lime stabilized material.

2.4.1.3 **Conditions Appropriate for Lime Stabilization**

i. **Particle size and surface to volume Ratio**

Solid particles react only on their surfaces and their respective reaction rates are proportional to their surface/volume ratio. This ratio is inversely proportional to the linear particle size. The amount of $CaSiCo_3$ formed per unit of lime and volume will be proportional to the percentage of the smallest size siliceous components of the soil. Also the higher the proportion of silica and the smaller that of aluminium and iron sesquioxides in the smallest size fractions, the greater is their reactivity with $Ca(OH)_2$

ii. **Solubility of Calcium Hydroxide**

The solubility of $Ca(OH)_2$ in water is relatively small about 165 grams per litre at normal temperature. To assure the greatest possible rate of solution of the solid $Ca(OH)_2$, it should be in a colloidally dispersed state. With regard to the amount of hydrated lime required for the stabilization of clay soils, there seems to exist specific threshold or minimum amount below which there is little if any real stabilization, and also high amounts above which increase in amount of stabilizer

does not produce a significant increase in the quality of the system. The threshold values usually lie between 1% and 2% of $Ca(OH)_2$ based on the weight of the dry clay soil.

iii. **Reactions for Proper Balance**

If the soil does not possess the required highly siliceous minerals that are capable of reacting with the lime, then such material can be added in the form of volcanic ash (puzzolan santorin), defatted diatomaceous earth, highly siliceous fly ashes, etc.

In practice ratios of fly ashes to lime run from 3:1 to 5:1 indicating that the major part of the fly ash acts as an heat filler.

iv. **Effectiveness of Lime Treatment**

Treatment with quicklime or hydrated lime is especially effective in improving the engineering properties of heavy clay soils or of granular soils which, because of high water affinity in their silt-clay fraction, fall short of having a dependable granular skeleton.

In accordance with United States experience, treatment with lime is most effective for:

a. Stabilization of clay-gravel materials to serve as bases for pavement: 2-4% of $Ca(OH)_2$ by weight of soil is used for this purpose.

b. Stabilization of heavy clay soils to serve as bases: 5-10% of lime treatment or as subbase 1-3% for pavement.

Lime treatment has been found less effective for silt-loam soils and is not recommended for sandy soils except in combination with added clay, fly ash or other pozzo lanic constituents which serve as both hydraulically reactive and filler to improve the gradation and reactivity of the soil.

v. **Laboratory Testing of Lime Stabilized Soil Specimens**

Recommended percentages of lime for laboratory testing and for construction vary from 2-10%, they are 2, 3 and 5% for coarse soils (Clay-gravels, caliche, sandy soils) having less than 50% silt clay fraction, and for intermediate soils 3, 5 and 7% of lime is indicated **(Winterkorn and Fang, 1975)**.

The common methods of laboratory testing of lime-stabilized soil specimens that have come into practical use are:

a. The triaxial compression method

b. The CBR method

c. The Hveen Stabilometer method.

Other conditions appropriate for lime stabilization like curing, temperature, types and grade of lime have been discussed earlier **(WinterKorn and Fang 1975)**.

2.4.2 **Lime-Soil Stabilization Mix Design**

The choice of lime stabilization depends on economic factors, kind of soil, climate and use. The mix proportions are seldom critical for a given soil, and many mixes could be used. **Woods (1960)** stated that the problem is then to select proportions which give the needed amount of cementation for the lowest cost. Trial batches are mixed in the laboratory, compacted, cured and tested, so that the best mix may be chosen. Several methods are

available for selecting trial batch mix proportions. One method is to prepare mixes to represent all feasible combinations of soil and lime. Both the evaluation and mixture design procedures should be developed in terms of intended use, objective and test conditions. In addition, the evaluation should be based on meaningful tests which provide fundamental engineering properties rather than empirical test results. An attempt should be made to recognize and consider the inherent variation in soil-lime mixtures High strengths can be achieved with lime but care must be taken to avoid potential shrinkage cracks which are associated with the stronger mixes. Under traffic loadings, these cracks can reflect through the surfacing and permit water to enter the pavement and thereby lead to failures. The material must also be checked for sensitivity to water as this can cause a severe loss in strength. The proportion of stabilizer required to desensitise the mix may well be the controlling factor in design.

Normally the maximum proportion of lime used in a mix should be limited to 7-8% because stronger mixes are not economical. Often strengths are increased relatively little by additions beyond a certain percentage of lime. Lime stabilized materials permit longer handling times between mixing and formal compaction than cement which may be a useful characteristic when employing inexperienced labour. Under some circumstances, stabilization with lime alone will be most satisfactory and economical. The major objective of the mixture design process is to establish an appropriate lime content for construction. Mixture design criteria are needed to establish the quality of lime required to produce an acceptable quality mixture.

CHAPTER THREE

METHODOLOGY

3.1 MATERIALS

Five laterite soil samples designated as samples A, B, C, D and E were used for this investigation. The samples are borrow pit material used on.

i. The Federal Polytechnic Ado-Ekiti road rehabilitation projects - Types A and B.

ii. Ado-Ekiti - Ilawe road rehabilitation project - Type C

iii. Ado-Ekiti - Ikere road rehabilitation project - Type D

iv. Ado-Ekiti - Iworoko road rehabilitation project - Type E

All the samples were chosen as representative samples after tests had been carried out on various samples around Ado-Ekiti environs.

After collection, samples were stored in polythene bags immediately they were collected and prior to their use to prevent loss of moisture.

A sizeable amount of quicklime was obtained which was slaked and used as stabilizing agent.

3.2 TESTING PROCEDURES

A. Natural Soil Samples

The following tests were carried out on each of the five samples used:

i. Identification and classification tests which include particle size distribution, Atterberg limits and specific gravity.

ii. Mineralogical composition to determine the clay minerals and other chemical constituents of the soils.

iii. pH of the soil samples

iv. Compaction tests to determine moisture density relations of each sample

v. California Bearing Ratio Tests (CBR)

vi. Unconfined compression strength tests

vii. Triaxial compression tests

The procedures used in carrying out these tests were according to BS Standard methods.

B. Treated samples

The chemical additive used for treating the soil is lime and the proportions used are 2, 6 and 10%.

The following tests were carried out on each treated sample

i. pH of the treated sample

ii. Atterberg limit tests

iii. Compaction tests

iv. California Bearing Ratio Tests (CBR)

v. Unconfined compression strength tests

vi. Triaxial compression tests

vii. Compressive strength tests

PRE-TREATMENT OF SAMPLES

When the samples were taken to the laboratory, the deleterious materials such as roots were removed. The samples were air-dried, broken down with mortar and pestle and passed through a No. 10 sieve to remove large particles.

To compare relative effects of the additives on performance of samples, varying proportions of each additive were used to prepare mixtures. The amounts of additives used are expressed as percentages of the dry weight of the sample. The lime contents used were 2%, 6% and 10%.

Mixing of samples with additives was done manually at the optimum moisture content of natural samples as obtained from compaction tests.

The soil-lime mixtures were hand mixed after they were weighed prior to the mixing in of water at required proportions.

Moulding of tests specimens was started as soon as possible after completion of mixing and the tests were conducted according to BS standards. For each percentage of additive. A total of four specimens were used to obtain each point on the moisture density relation. The specimen used for the tests are for both cured and uncured samples for 7, 21 and 28 days. The strength values reported are averages of the results for each mixture since the standard deviations of the values from the average values used are small.

The results are discussed in chapter four with the following relations shown.

i. Moisture - density relations for natural and treated samples with varying percentages

ii. Maximum dry density versus lime content

iii. Optimum moisture content versus lime content

iv. CBR versus lime content

v. Unconfined compressive strength (UCS) versus lime content

vi. Cohesion versus UCS

vii. Stress-strain curves for both natural and lime treated soils

viii. UCS with age of curing

ix. Particle size Distribution curve.

CHAPTER FOUR

LABORATORY RESULTS AND ANALYSIS OF NATURAL AND TREATED SOIL SAMPLES

4.1 CHARACTERISTICS OF NATURAL SAMPLES

After the samples had been collected and treated as described in chapter three, their properties were determined to ensure that when their response to additives was studied. all relevant factors would be available for establishing a correlation between composition and lime stabilization of the soils.

Results of tests performed on the five samples are as recorded below:

A. Identification and Classification Tests

The particle size distributions of the samples are shown in Table 4.1. The average distributions were

Sample A

Gravel	-	0.4%
Sand	-	41.2%
Clay	-	58.4%

Sample B

Gravel	-	4.2%
Sand	-	69.8%
Clay	-	26.0%

Sample C

Gravel - 6.2%

Sand - 46.4%

Clay - 47.4%

Sample D

Gravel - 8.2%

Sand - 58.6%

Clay - 33.2%

Sample E

Gravel - 7.2%

Sand - 50.6%

Clay - 42.2%

The grain size distribution curves are shown in Fig 4.1

The results of the Atterberg limits tests are shown in Table 4.3 and Fig 4.2.

The calculated Group Index values (G.I) are also shown in Table 4.3.

Soil type A comprises 41.6% coarse material and 58.4% fine with the liquid limit, plastic limit, plasticity index and Group Index values of 39%, 22%, 17% and 8 respectively. The soil is classified as inorganic silt of medium compressibility according to Casangrade, CL (fine grained - silty clay soil) according to Unified Soil Classification system and as A-6 (clayey soil) of fair to poor rating in terms of possible use as subgrade material according to AASHTO classification.

Soil type B comprises 74% coarse material and 26% fine with a liquid limit, plastic limit, plasticity index and Group Index values of 28%, 16%, 12% and 0 respectively. The soil is classified as inorganic clay of low compressibility according to Casangrade, SM (Coarse grained-silty sand soil) according to Unified Soil Classification system and as A-2 -6 (granular silty) of excellent - good rating as subgrade material according to AASHTO classification.

Soil type C comprises 52.6% coarse material and 47.4% fine with a liquid limit, plastic limit, plasticity index and Group Index values of 41%, 27%, 13% and 4 respectively. The soil is classified as organic soil of medium compressibility according to casangrade, SC - (coarse grained - sand-clay mixture soil) according to Unified Soil Classification system and as A-6 (clayey soil) of fair to poor rating as subgrade material according to AASHTO classification.

Soil type D comprises 66.8% coarse material and 33.2% fine with a liquid limit, plastic limit, plasticity Index and Group Index values of 39%, 24%, 15% and 1 respectively.

The soil is classified as inorganic silt of medium compressibility according to Casangrade, SC (coarse grained clayey sand soil) according to Unified Soil Classification system and as A-2-6 (clayey gravel) of excellent to good rating as subgrade material according to AASHTO classification.

Table 4.1: Particle Size Distribution of Samples A, B, C, D and E

	Sample Type A			Sample Type B			Sample Type C			Sample Type D			Sample Type E		
Sieve Opening	Weight Retained (g)	% Cumm Retained	% passing	Weight Retained (g)	% Cumm Retained	% passing	Weight Retained (g)	% Cumm Retained	% passing	Weight Retained (g)	% Cumm Retained	% passing	Weight Retained (g)	% Cumm Retained	% passing
12.500	-	-	100	-	-	100	-	-	100	-	-	100	-	-	100
9.510	2	0.4	99.6	21	4.2	95.8	31	6.2	93.8	41	8.2	91.8	36	7.2	92.8
4.760	4	1.2	98.8	77	19.6	80.4	40	14.2	85.8	94	27.0	73	76	22.4	77.6
3.350	7	2.6	97.4	51	29.8	70.2	-	-	-	-	-	-	-	-	-
2.360	6	3.8	96.2	39	37.6	62.4	58	25.8	74.2	101	47.2	52.8	96	41.6	58.4
1.180	28	9.4	90.6	55	48.6	51.4	30	31.8	68.2	31	53.8	46.6	27	47.0	53.0
0.600	40	17.4	82.6	35	55.6	44.4	38	39.4	60.6	24	58.2	41.8	19	50.8	49.2
0.425	23	22.0	78.0	16	58.8	41.2	19	43.2	56.8	12	60.6	39.4	9	52.6	47.4
0.300	27	27.4	72.8	22	63.2	36.8	15	46.2	53.8	8	62.2	37.8	7	54.0	46.0
0.150	42	35.8	64.2	32	69.6	30.14	21	50.4	49.6	14	65.0	35.0	11	56.2	43.8
0.075	29	41.6	58.4	22	74.0	26.0	11	52.6	47.4	9	66.8	33.2	8	57.8	42.2

Weight of Sample Used = 500g

Table 4.2: Coefficient of Uniformity of Samples

Sample Type	D_{60}(mm)	D_{10}(mm)	Cu
A	0.086	0.004	215
B	2.200	0.009	244
C	0.560	0.001	560
D	3.200	0.005	640
E	2.600	0.001	2600

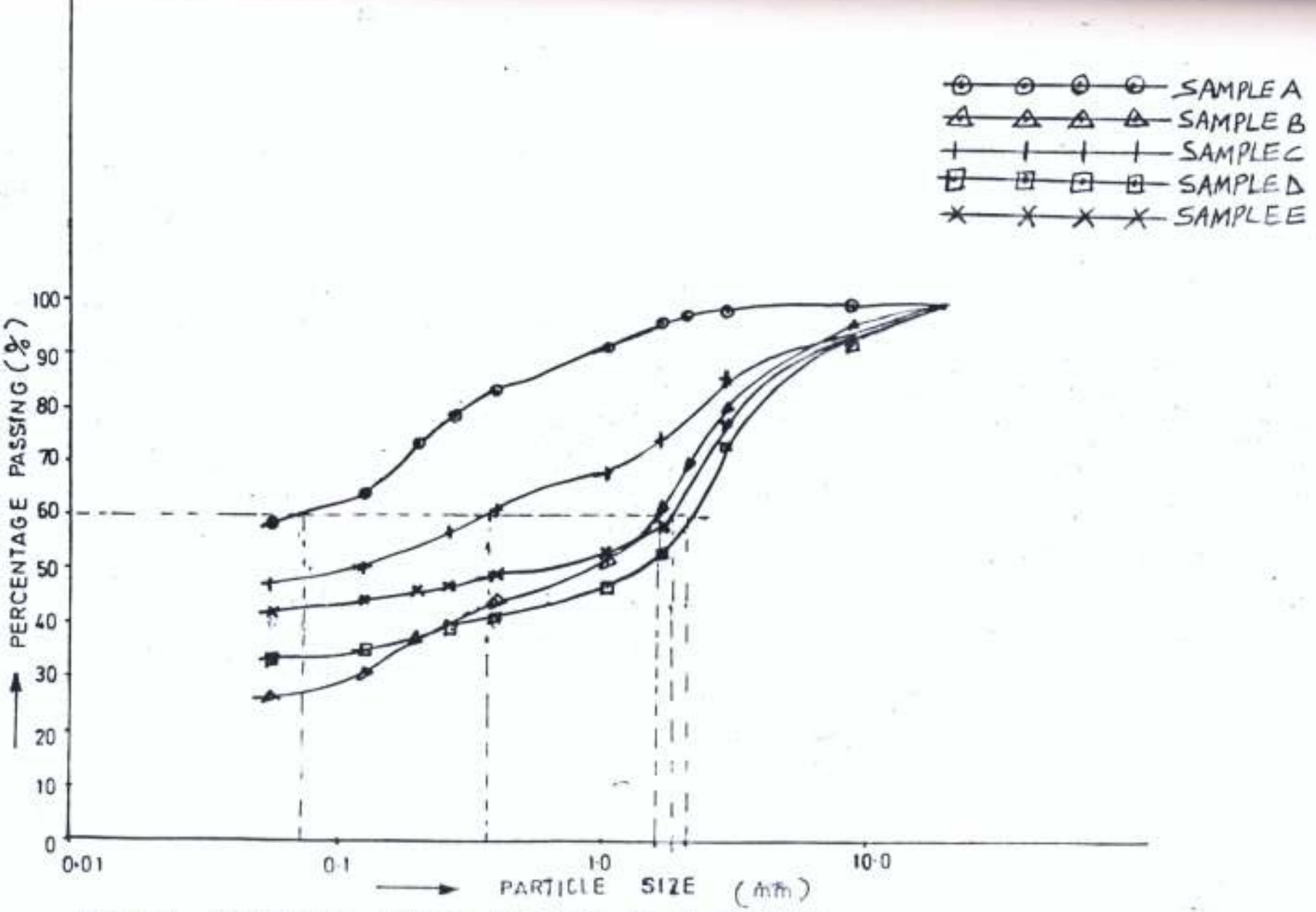


Fig. 4·1 PARTICLE SIZE DISTRIBUTION CURVE

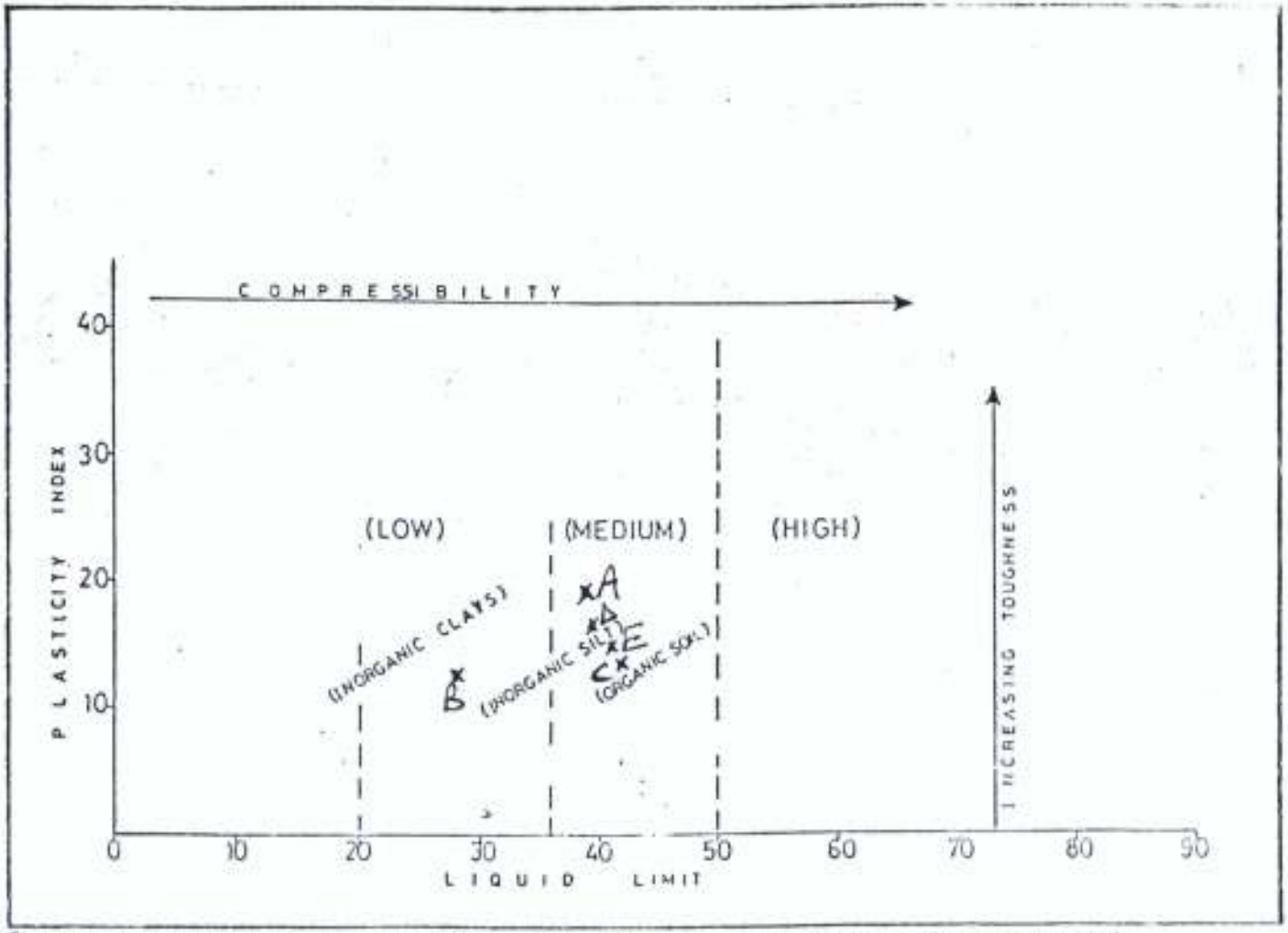


Fig. 4.2 Plasticity chart for Classification of Cohesive soils (after A. Casagrade 1932)

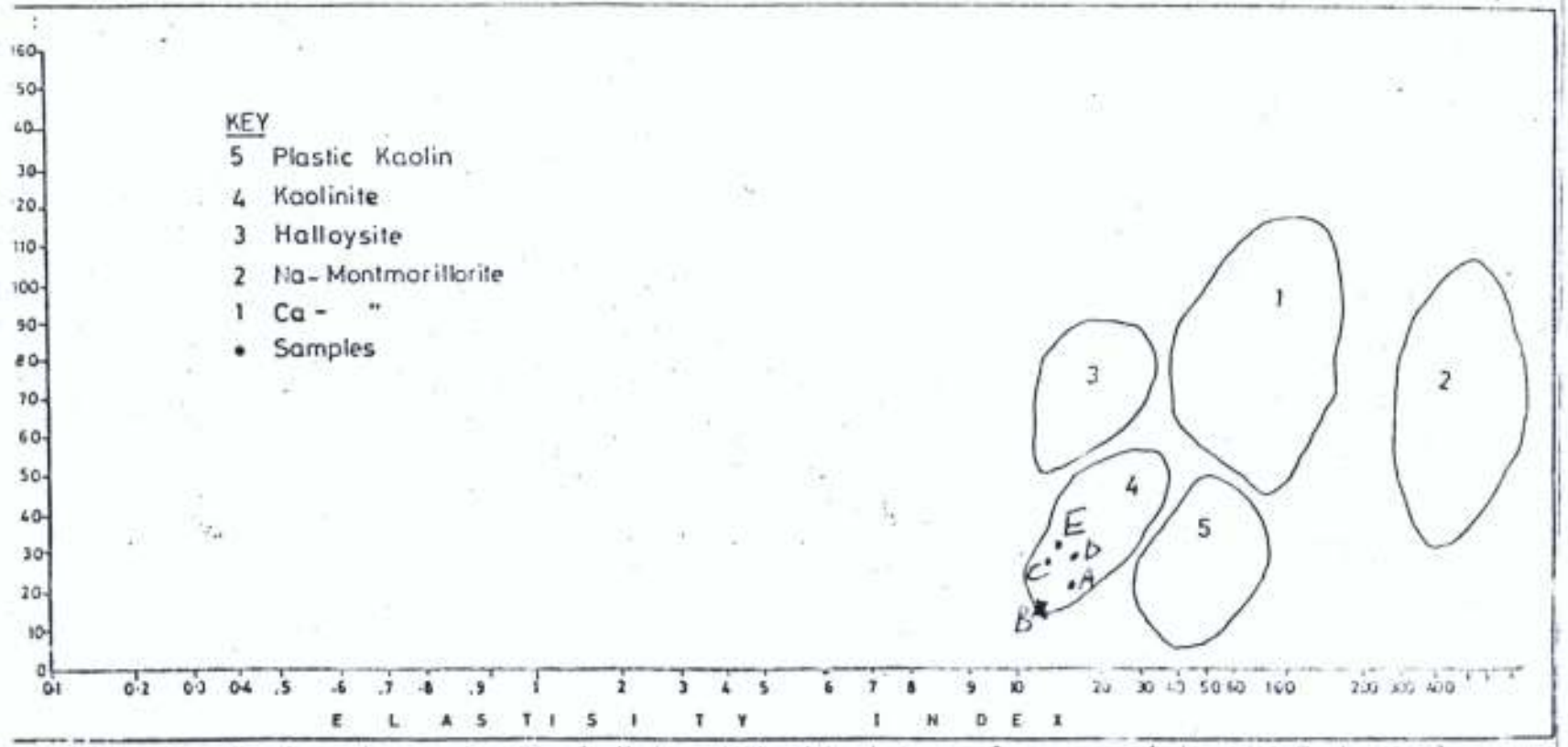


Fig 4.3 Clay identification chart using Plastic limit and Plasticity index as parameter (after J.A. Grain, 1971)

Soil type E comprises 57.8% coarse material and 42.2% fine with liquid limit, plastic limit, plasticity index and Group Index values of 40%, 26%, 14% and 2 respectively. The soil is classified as organic soil of medium compressibility according to Casangrade, SC (coarse grained - clayey sand soil) according to Unified Soil Classification system and as A-6 (clayey soil) of fair to poor rating as subgrade material according to AASHTO classification.

B. Chemical And Mineralogical Composition

In order to establish correlation between composition and chemical stabilization, the chemical and mineralogical composition of the samples were determined using the procedures as given in BS 1377. The results are as shown in Table 4.4

Table 4.3: Atterberg limits of sample

ple type	Liquid limit	Plastic limit	Plasticity Index	Group Index	Specific Gravity
A	39	22	17	8	2.57
B	28	16	12	0	2.53
C	41	27	13	4	2.35
D	39	24	15	1	2.46
E	40	26	14	2	2.56

Table 4.4: Chemical Analysis Result

nple type	SIO_2 %	Al_2O_3 %	Fe_2O_3 %	Lol %	U %	pH	$\frac{Sio_2}{(Al_2O_3+Fe_2O_3)}$
A	45.10	38.00	1.53	12.17	3.20	5.8	1.14
B	52.43	34.60	1.67	9.78	1.52	6.5	1.45
C	51.00	37.15	0.85	8.15	2.85	6.1	1.34
D	54.40	34.10	0.50	10.71	0.29	5.7	1.57
E	55.03	31.04	0.64	11.97	1.32	5.8	1.74

SiO_2 - Silica content

Al_2O_3 - Alumina content or Aluminium sesquioxide

Fe_2O_3 - Iron sesquioxide

LOI - Loss on Ignition

U - undetermined

For the five samples, the silica to iron and aluminium sesquioxides ratio is shown in Table 4.4. All the values so obtained are less than 2 indicating that they are mostly lateritic soils except for sample type A that is a true laterite. The pH values of the samples from the table indicates that they are acidic.

The result of the mineralogical composition of all the samples indicate that they mainly contain kaolinite minerals. This can be seen in Fig 4.3.

C_2 Compaction Test.

Results of compaction tests on the natural samples are shown in Figure 4.4 for sample types as representative of the laboratory procedures. Sample type A has a maximum dry density (MDD) of 1.8g/cc with optimum moisture content (OMC) of 13%, sample type B has a MDD of 2.18g/cc with OMC of 11%, sample type C has a MDD of 1.77g/cc with OMC of 15.8%, sample type D has a MDD of 1.96/cc with OMC of 10.8% sample type E has a MDD of 1.82g/cc with OMC of 15.8%. From the results samples types A, C and E have low dry densities which will lead to low strength of the samples if used naturally as construction materials, especially as subgrade materials but samples types B and D have high dry densities which if used naturally as construction materials especially as subgrade materials will produce high strength. Therefore the need arises to improve the properties of samples type A, C and E. Finding also correlates with the rating based on the soil clasifications earlier

presented.

D. California Bearing Ratio (CBR)

The suitability of the samples in their natural forms for use as highway materials was investigated using the CBR test. The results gave CBR values of 2%, 14% and 24% for samples A, C and D respectively. Using the table for typical rating of performance of soils, the CBR values for sample types A and C fall into the category of poor soil which needed its properties to be improved upon for use as base and subbase materials while sample type D falls into the category of good soil for use as base and subbase material

E. Unconfined compressive Strength, UCS

Unconfined compressive strengths of the three natural samples were determined using remoulded samples. The tests were carried out at 7, 21 and 28 days.

The remoulded samples had strengths that increased with age of curing from 1.41MN/m^2 7 days strength to 1.67MN/m^2 28-day strength for sample type A and from a 7-days strength of 2.23MN/m^2 to 2.52 MN/m^2 28-day strength for sample type C and from a 7 day strength of 3.6MN/m^2 to 4.11MN/m^2 28-day strength for sample type D. The low result for sample types A anc C indicate that the materials are unsuitable as highway construction materials. Hence the need to improve their engineering properties.

F. Compressive Strength

The compressive strength test for the natural samples does not give any appreciable result because it cannot withstand the crush strength of the crushing machine. The values are all zeros.

G. Shear Strength parameters (C-Ø)

The shear strength of the samples is another factor or characteristic of the samples

that can be used to determine the suitability of a soil for highway construction. The cohesion and adhesion property of the soil, the frictional resistance to sliding between soil particles are important. To investigate these properties of the soil, the unconsolidated undrained test was carried out on each of the samples. The results gave cohesion values of 17.5kN/m^2, 30kN/m^2 and 65kN/m^2 for cured sample types A, C and D while the angle of friction values are 8^0, 10^0 and 23^0 for samples type A, C and D respectively. From the values, it is evident that the soils are cohesive soils (C - O). Sample types A and C have low angles of friction: hence there is need for improvement.

2 CHARACTERISTICS OF STABILIZED MATERIALS

Since the properties of sample types B and D are similar and sample types C and E are similar, the stabilized samples were limited to sample types A, C and D respectively.

The three sample types were treated with lime as additive to improve their engineering properties. For proper investigation and comparison of results, varying proportions were used. The following tests were carried out on the treated samples.

a. Compaction tests

b. California Bearing Ratio (CBR) tests

c. Unconfined compression strength tests

d. Triaxial Compression tests

e. Compressive strength tests

2.1 Compaction Characteristics

Compaction tests were carried out on treated samples to establish relationship between strength and density for treated soil mixtures compacted at different moisture contents and using varying proportions of the additive.

Results of compaction test on samples treated with lime are as shown on Table 4.5 The variations of dry density and moisture content are shown in the Appendix for all samples treated with lime contents. The results show that adding lime to the samples increased the optimum moisture content and decreased the maximum dry densities. For sample type A, the OMC and MDD changed from 13% and 1.8g/cc at 0% lime content to 22.8% and 1.57g/cc at 10% lime content. For sample type C, the OMC and MDD changed from 15.8% and 1.77g/cc at 0% lime content to 23% and 1.62g/cc at 10%. The slight decrease in OMC at 2 and 6% additive may be due to error in the laboratory. For sample type D the OMC and MDD changed from 10.8% and 1.96g/cc at 0% lime content to 17% and 1.78/cc at 10% lime content.

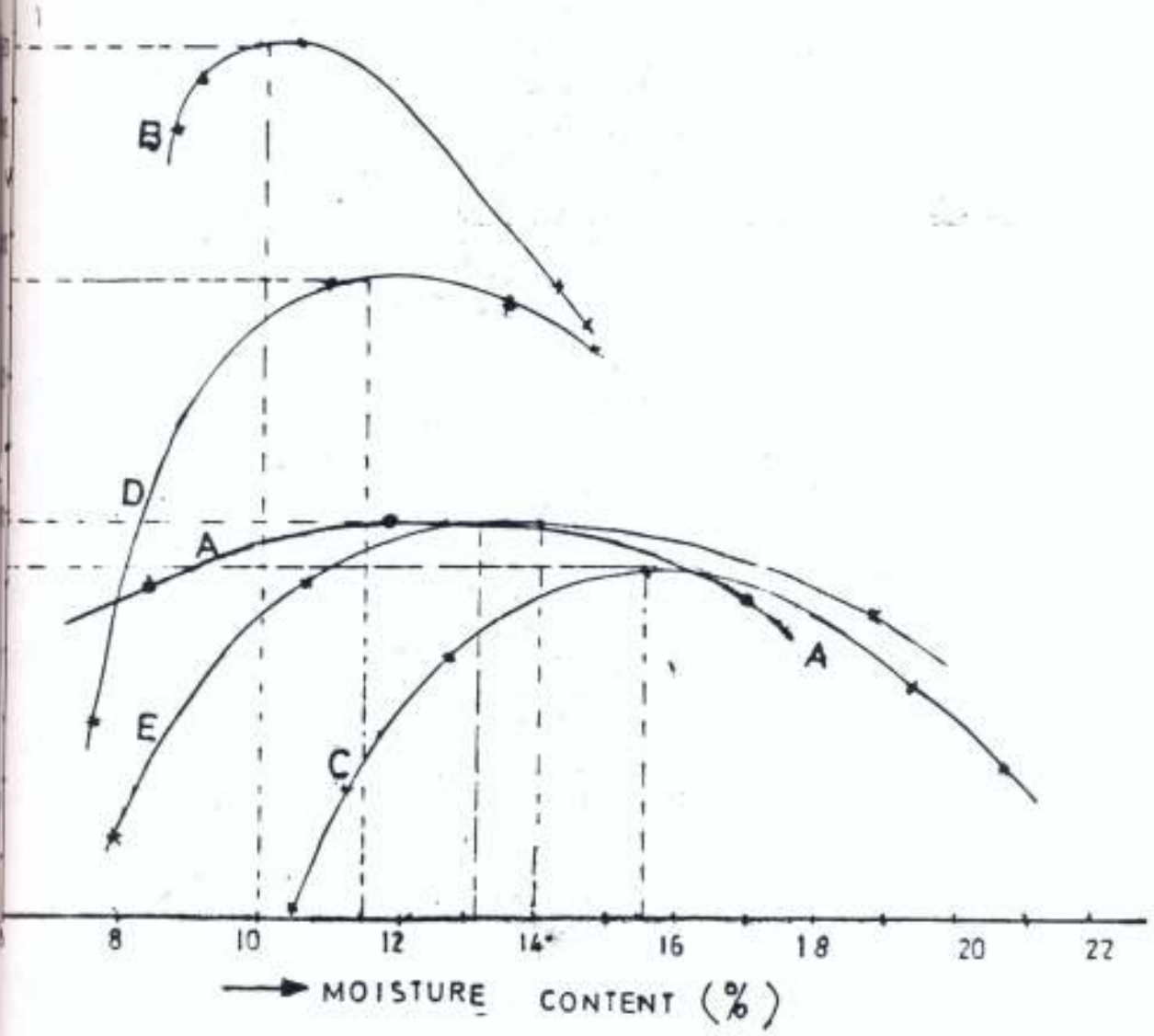
Sample type D has the highest MDD in all the percentages of lime content because of the presence of high coarse materials in the samples which resulted into high compacted strength while sample type A has low MDD due to its high plasticity which caused low compaction.

The increase in optimum moisture content as lime content increases is caused by the increase in amount of water needed for full hydration developed within the soil as a result of the chemical reactions with the lime; while the reduction in maximum dry densities is as a result of flocculation of soil fines and creation of loosely bonded aggregates in treated samples caused by the added lime.

Table 4.5: Summary of Test Results for Lime Treated Samples

Lime Content %	Atterberg Limits %				OMC %	MDD g/cc	pH value	CBR %			UCS MN/m²			Cohesion (C)KN/m²			Angle of friction (Ø)°			Avg. Compressive Strength N/mm²		
	LL	PL	PI	SL				7 days	21 days	28 days	7 days	21 days	28 days	7 days	21 days	28 days	7 days	21 days	28 days	7 days	21 days	28 days
Sample-type A																						
0	39	22	17	12.9	13.0	1.80	5.8	-	-	-	1.41	1.58	1.67	-	-	-	-	-	-	0	0	0
2	54	40	13	11.4	14.0	1.67	10.4	2	2	2	3.22	3.35	3.38	117	60	-	3	37	-	0.58	0.66	0.77
6	48	43	5	11.7	14.5	1.70	11.1	17	50	97	3.60	4.63	5.21	82	130	158	11	7	16	0.69	0.85	0.88
10	56	47	10	9.3	22.8	1.57	11.3	3	5	7	2.65	3.76	5.78	125	205	130	20	8	35	0.55	0.74	0.85
Sample type C																						
0	41	27	13	9.3	15.8	1.77	6.1	-	-	-	2.23	2.61	2.52	-	-	-	-	-	-	0	0	0
2	48	31	17	8.6	15.0	1.77	10.8	3	3	4	4.55	6.29	6.76	-	50	60	-	29	46	0.88	1.02	1.07
6	44	33	11	7.9	14.8	1.70	11.0	18	45	93	4.79	5.17	7.37	73	-	153	12	-	34	0.91	1.05	1.18
10	48	34	13	7.1	23.0	1.62	11.4	13	44	64	6.52	7.22	8.37	78	-	-	9	-	-	0.96	1.02	1.10
Sample type D																						
0	39	24	15	10.0	10.8	1.96	5.7	-	-	-	3.60	3.90	4.11	-	-	-	-	-	-	0	0	0
2	47	37	10	9.3	16.6	1.79	10.1	3	4	6	4.07	4.53	4.76	35	-	-	23	-	-	0.83	0.99	1.13
6	40	38	2	7.9	12.5	1.80	11.1	10	44	90	4.61	5.14	4.84	60	-	-	20	-	-	0.88	1.07	1.24
10	43	41	2	5.7	17.0	1.78	11.4	19	48	98	6.14	6.60	7.06	45	-	-	25	-	-	0.99	1.13	1.30

NATURAL SAMPLES



Fig·4·4 VARIATION OF DRY DENSITY WITH PERCENTAGE MOISTURE CONTENT OF THE NATURAL SAMPLES

SAMPLE A

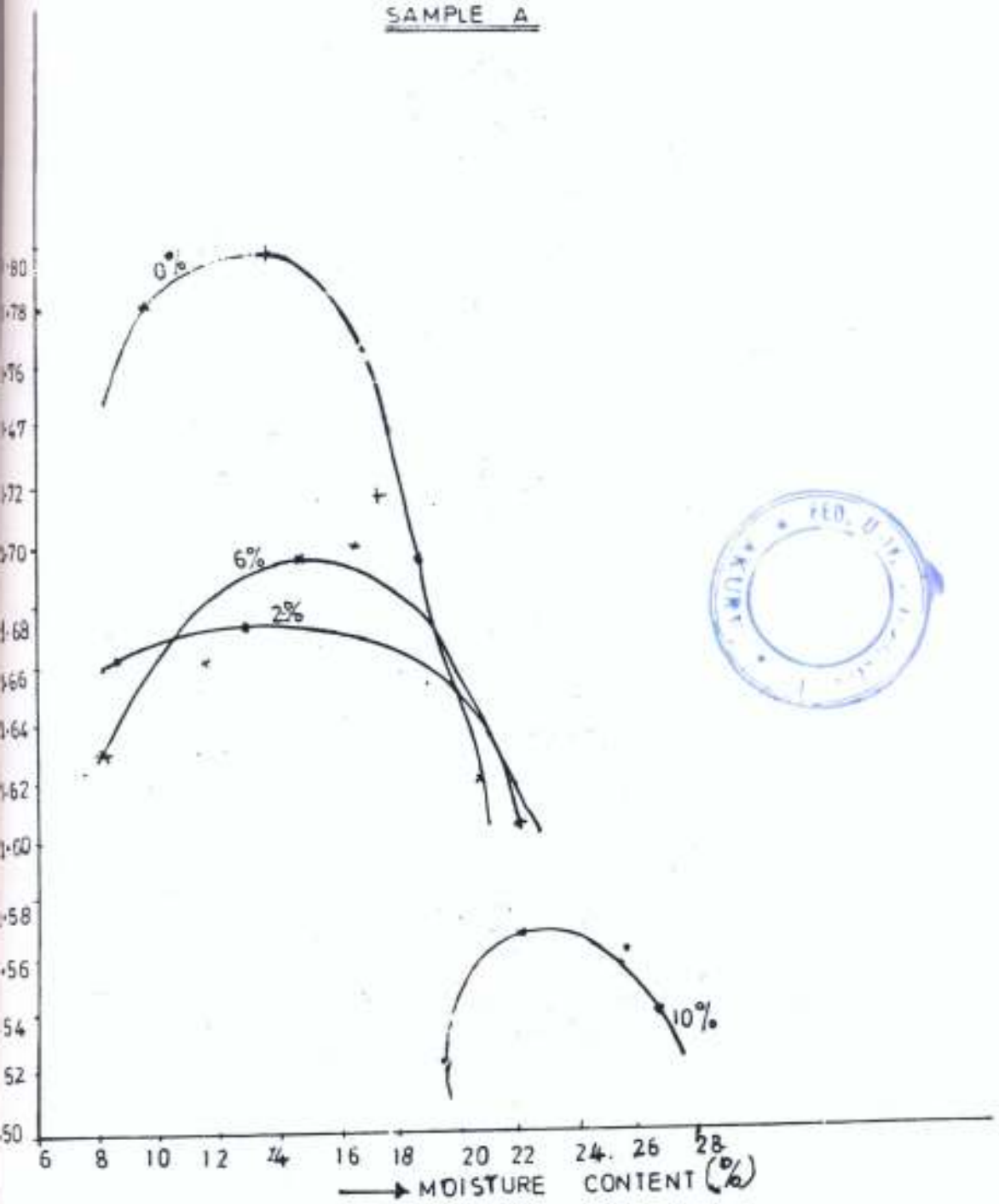
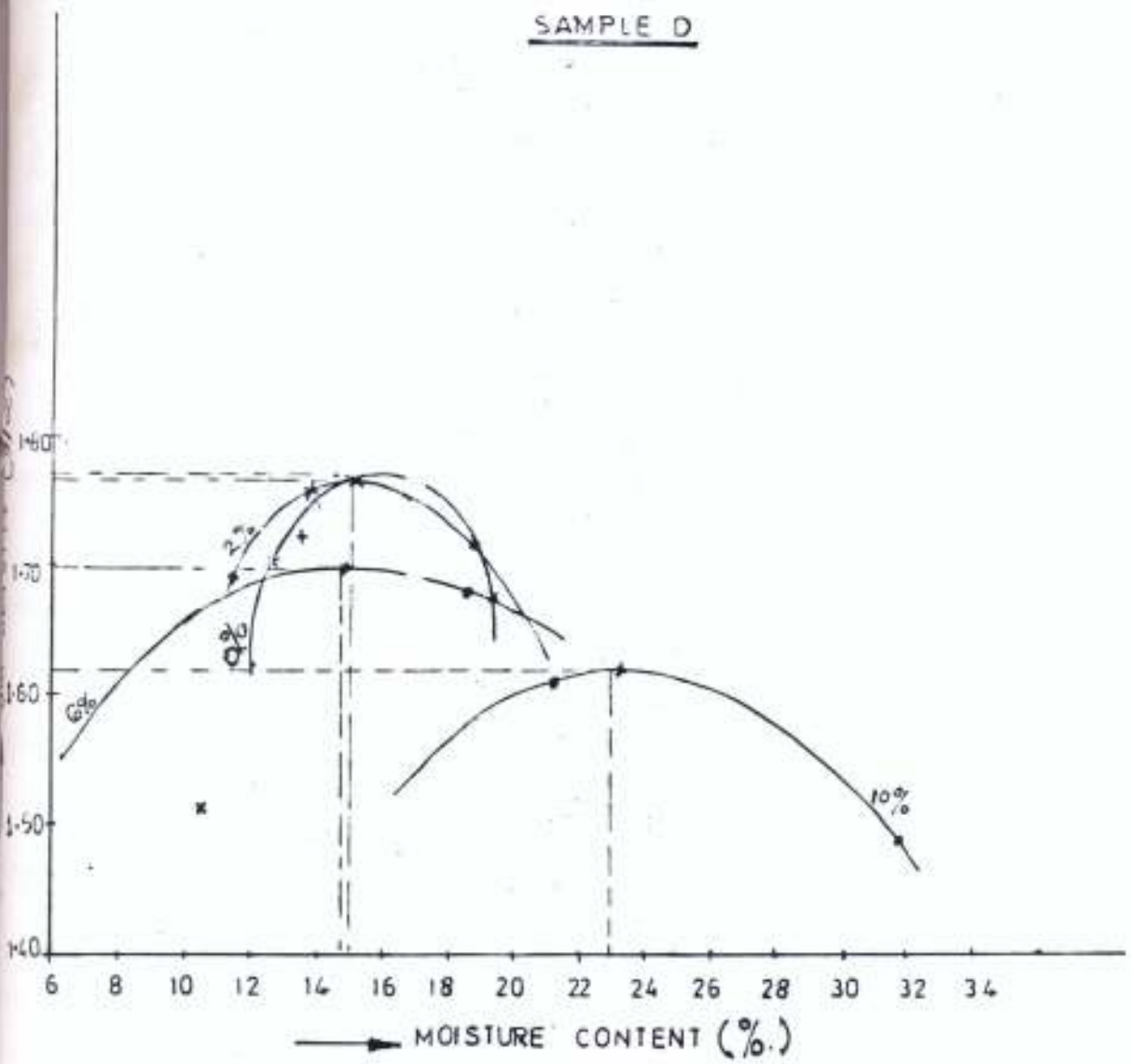


Fig.45 VARIATION OF DRY DENSITY AND MOISTURE CONTENT WITH LIME CONTENT



Fig·4·6 VARIATION OF DRY DENSITY AND MOISTURE-CONTENT WITH LIME CONTENT

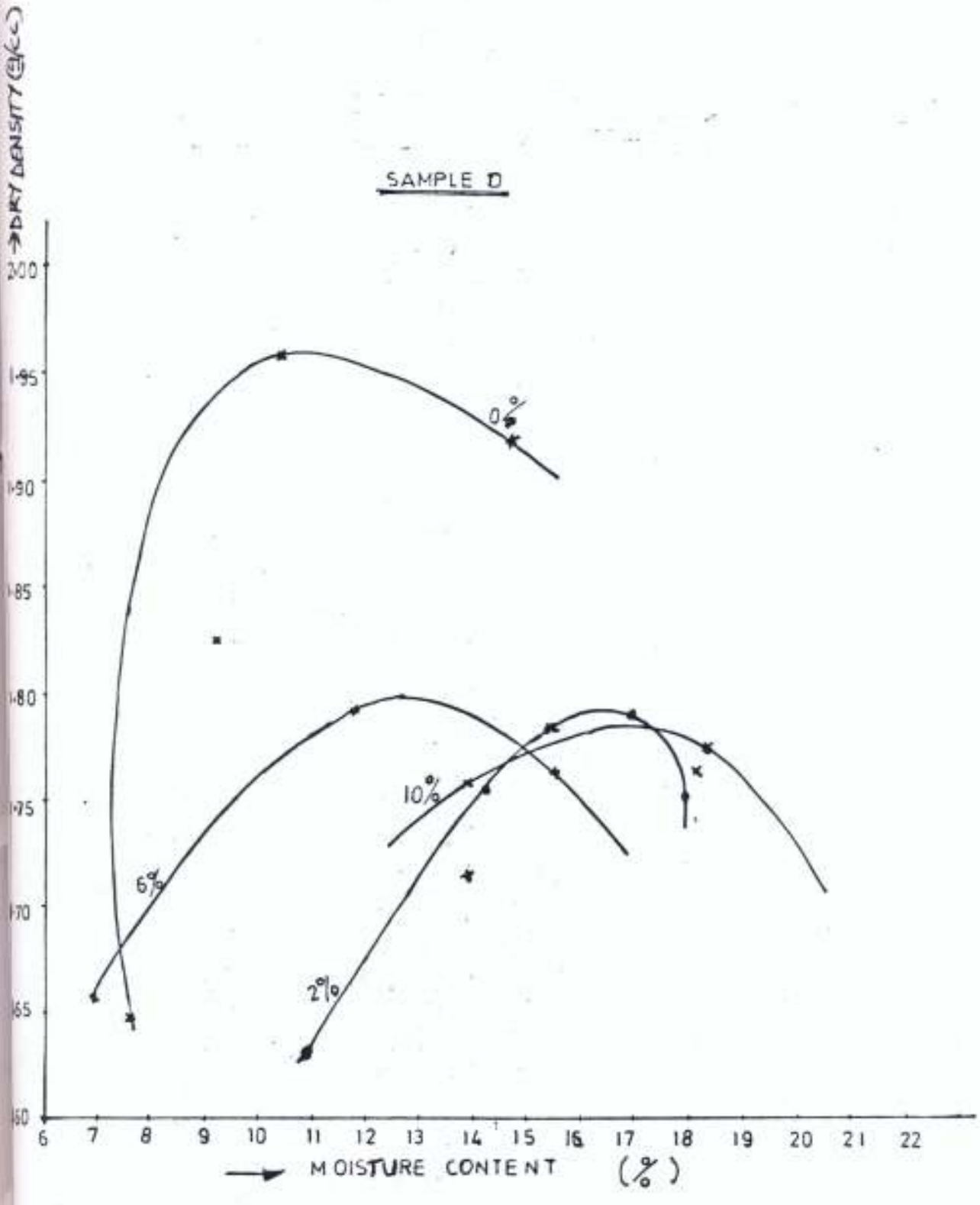


Fig.4.7 VARIATION OF DRY DENSITY AND MOISTURE CONTENT WITH LIME CONTENT

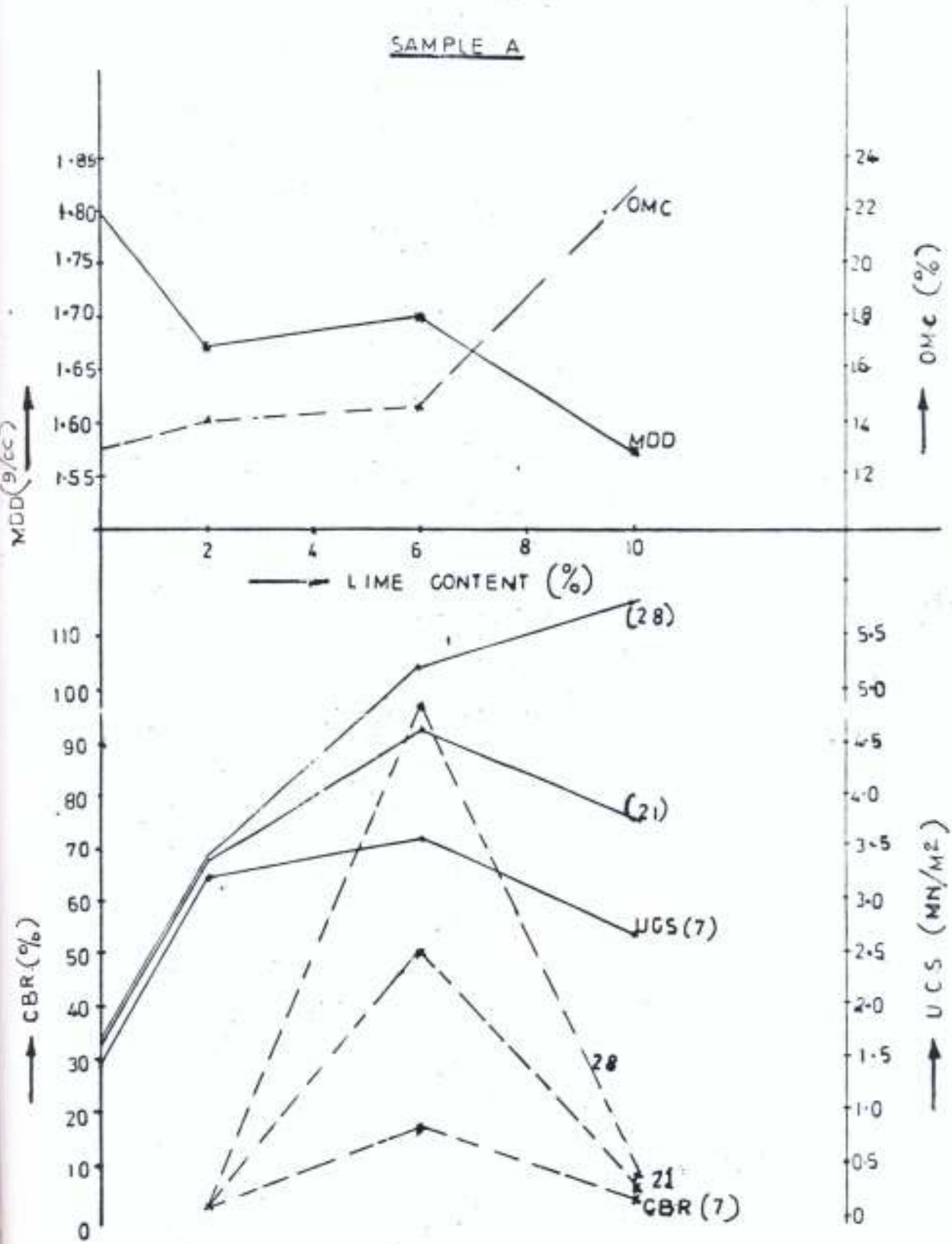
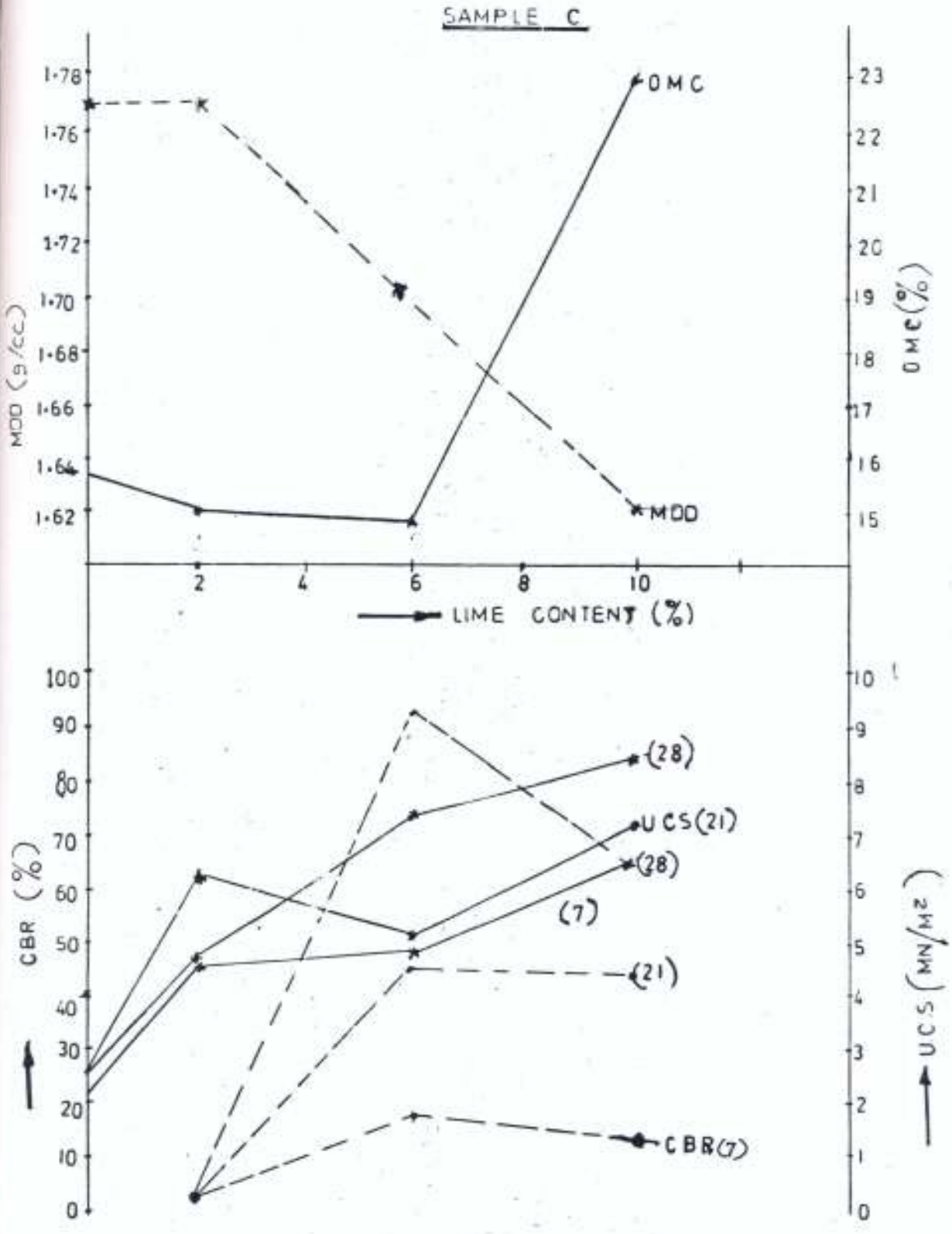
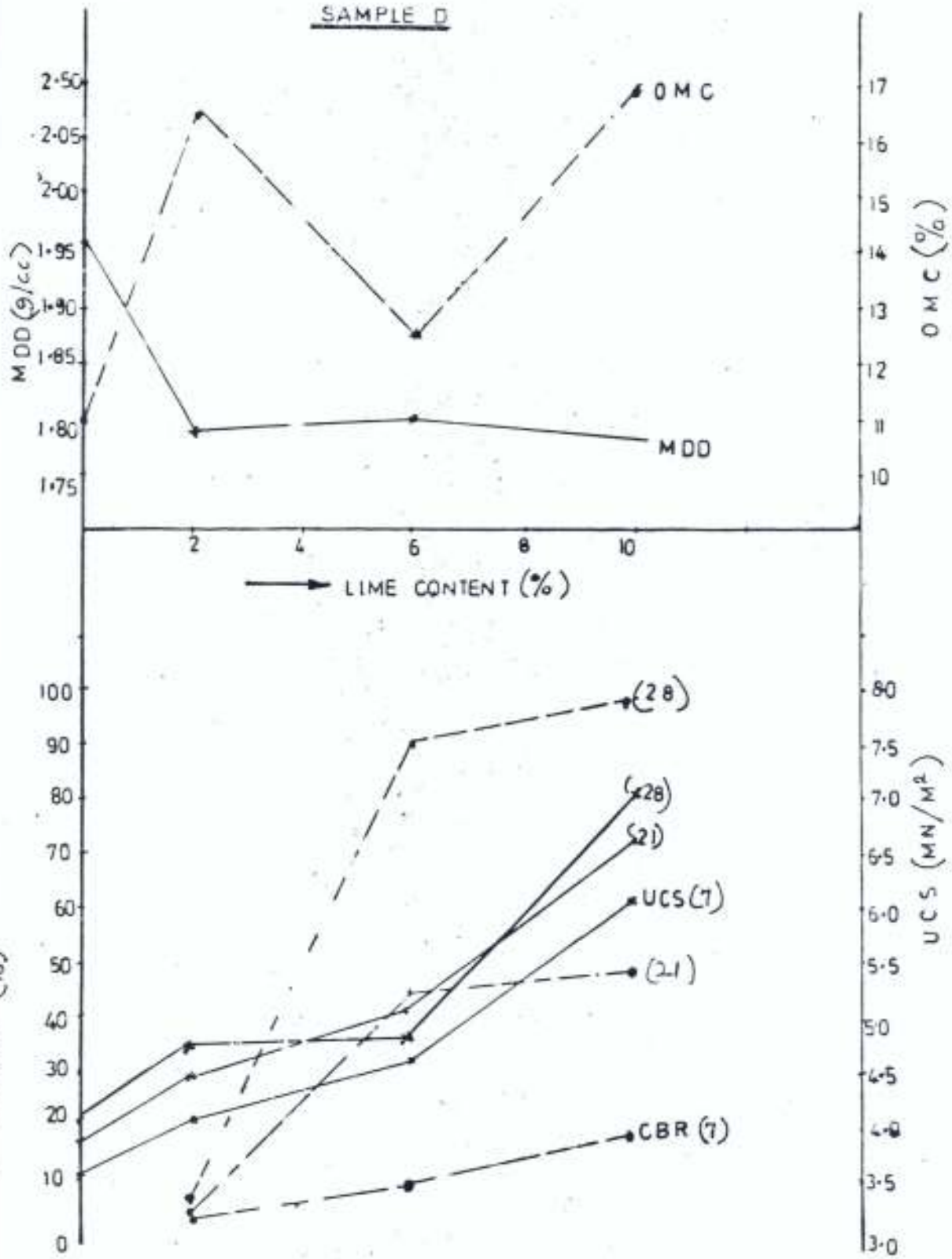


Fig 4.8 VARIATION OF MDD, CBR, OMC AND UCS WITH LIME CONTENT



Fig·4·9 VARIATION OF MDD, OMC, CBR AND UCS WITH LIME CONTENT



Fig·4·10 VARIATION OF MDD, OMC, CBR AND UCS WITH LIME CONTENT

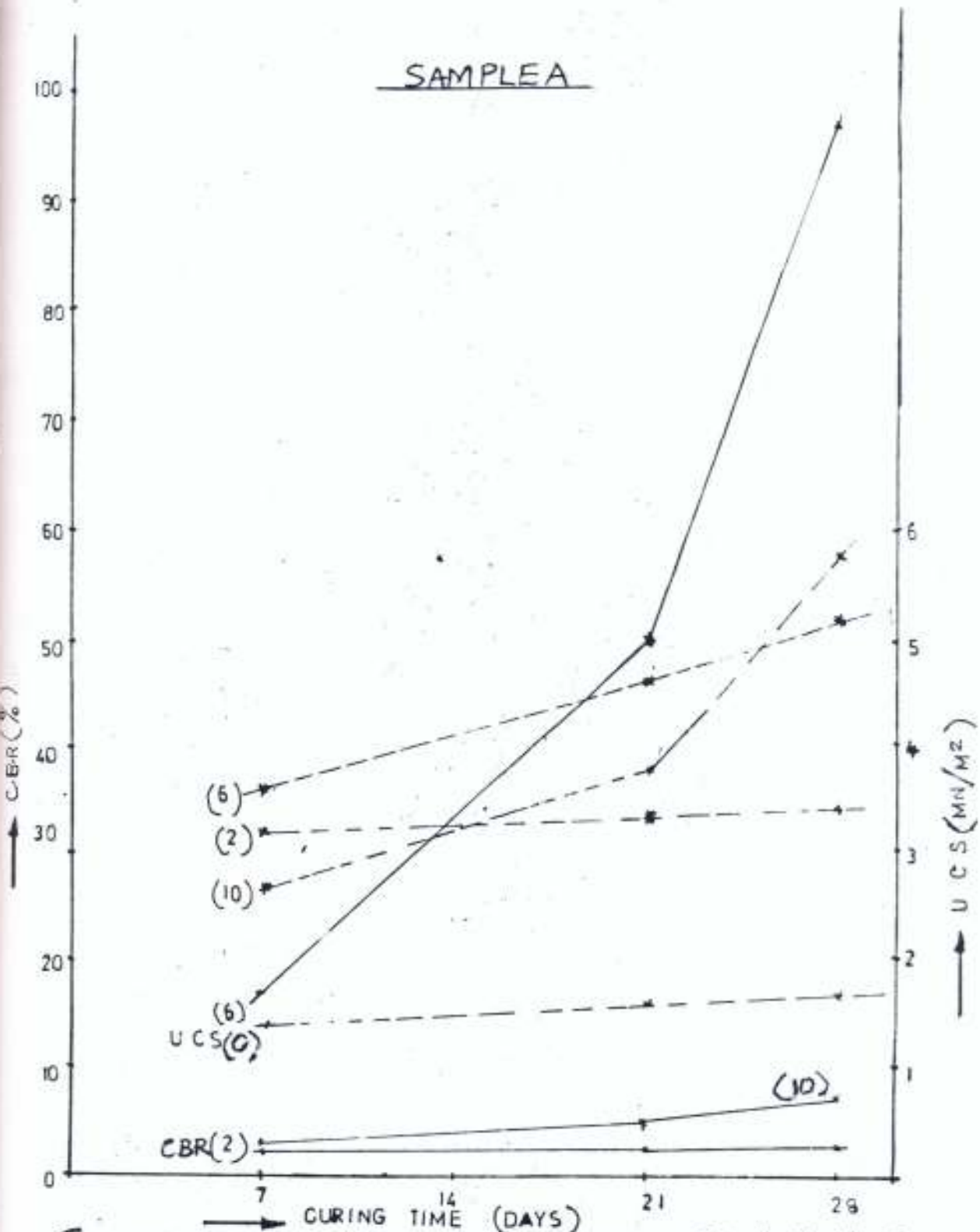


Fig. 4.11 INFLUENCE OF CURING ON CBR AND UCS OF LIME STABILISED SOIL

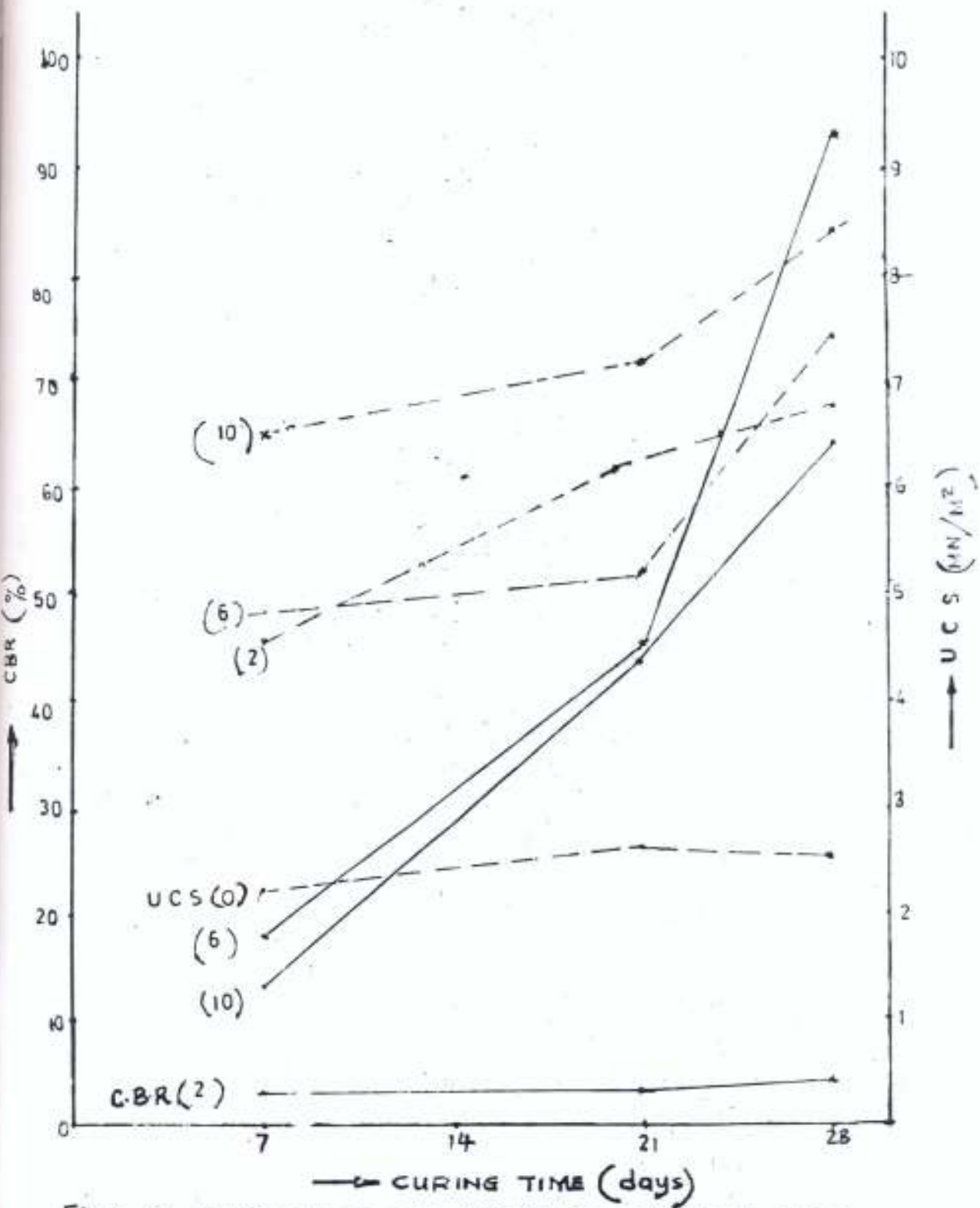


Fig.4.12 INFLUENCE OF CURING ON CBR AND UCS OF LIME STABILISED SOIL

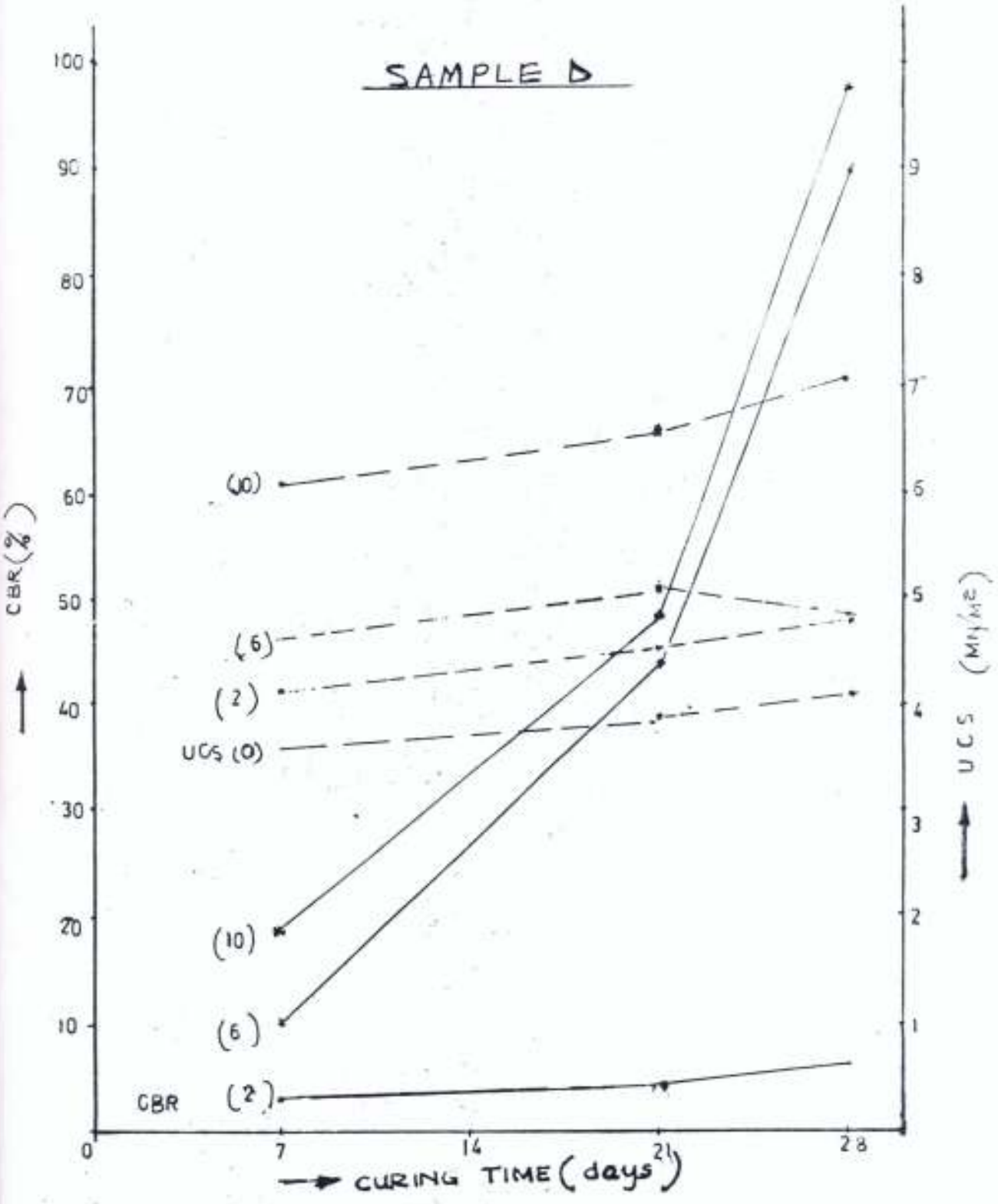


Fig4.13 INFLUENCE OF CURING ON CBR AND UCS OF LIME STABILISED SOIL

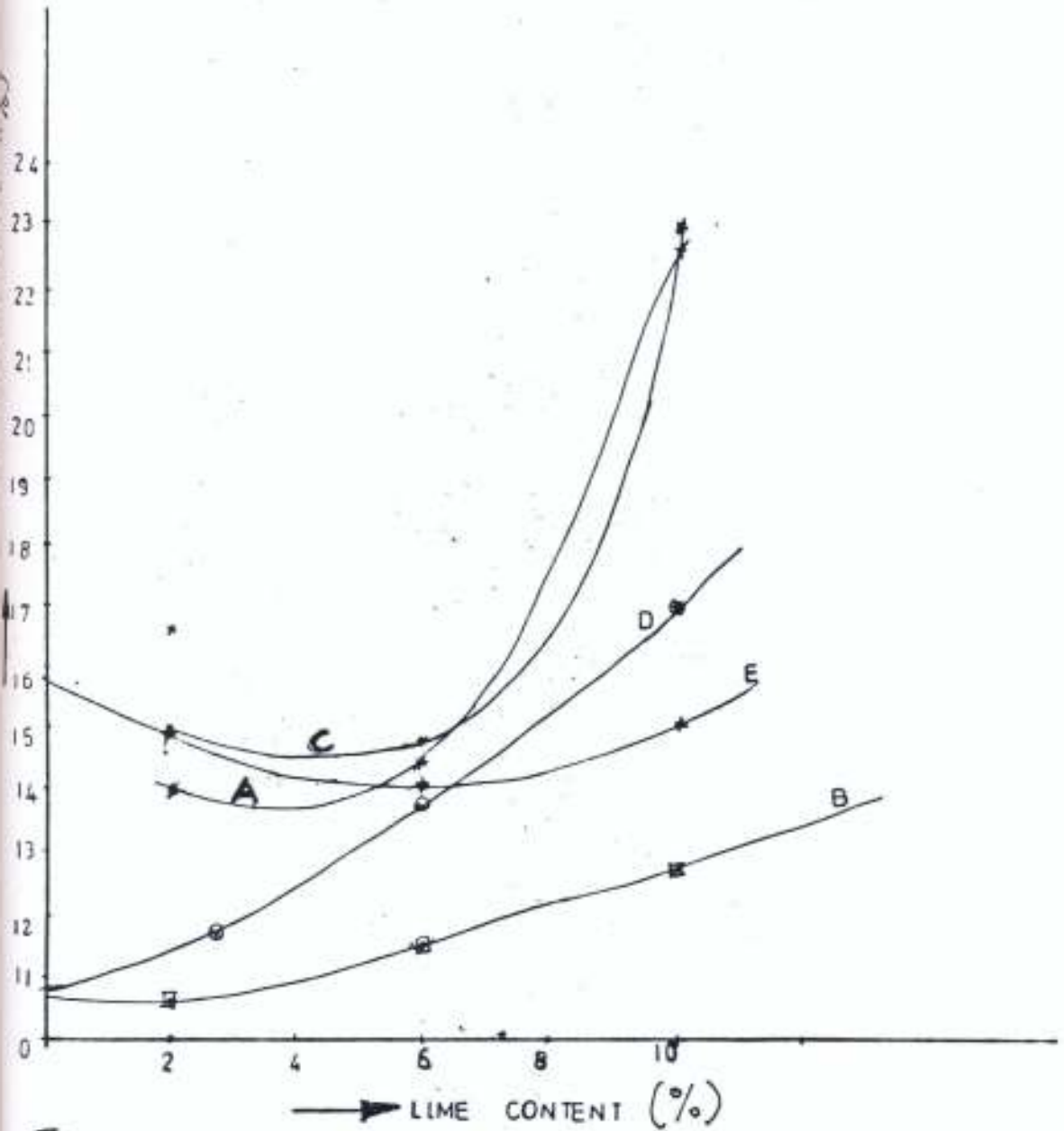


Fig. 4.14 VARIATION OF OPTIMUM MOISTURE CONTENT WITH LIME CONTENT

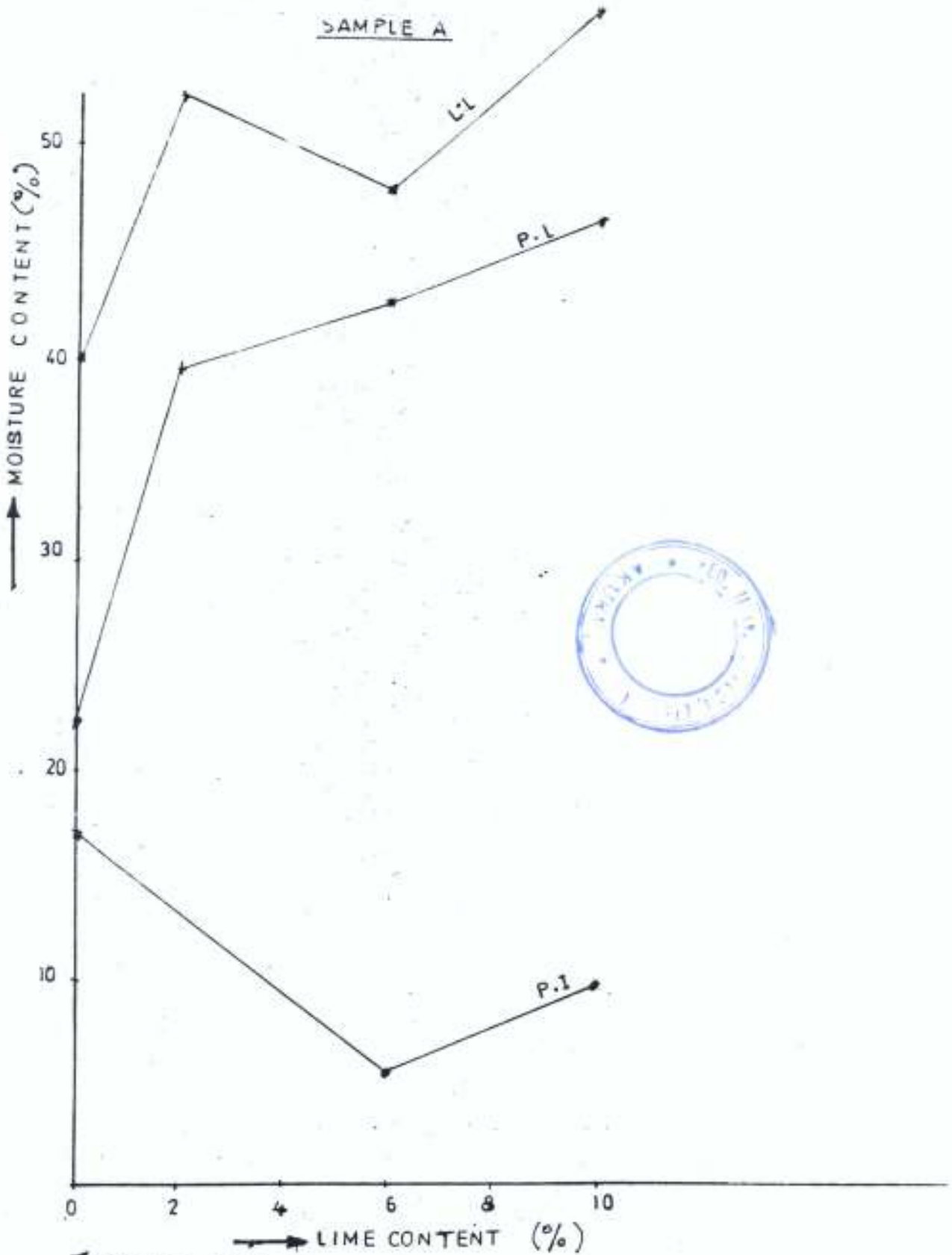


Fig 4.15 VARIATION OF ATTERBERG LIMIT WITH LIME CONTENT

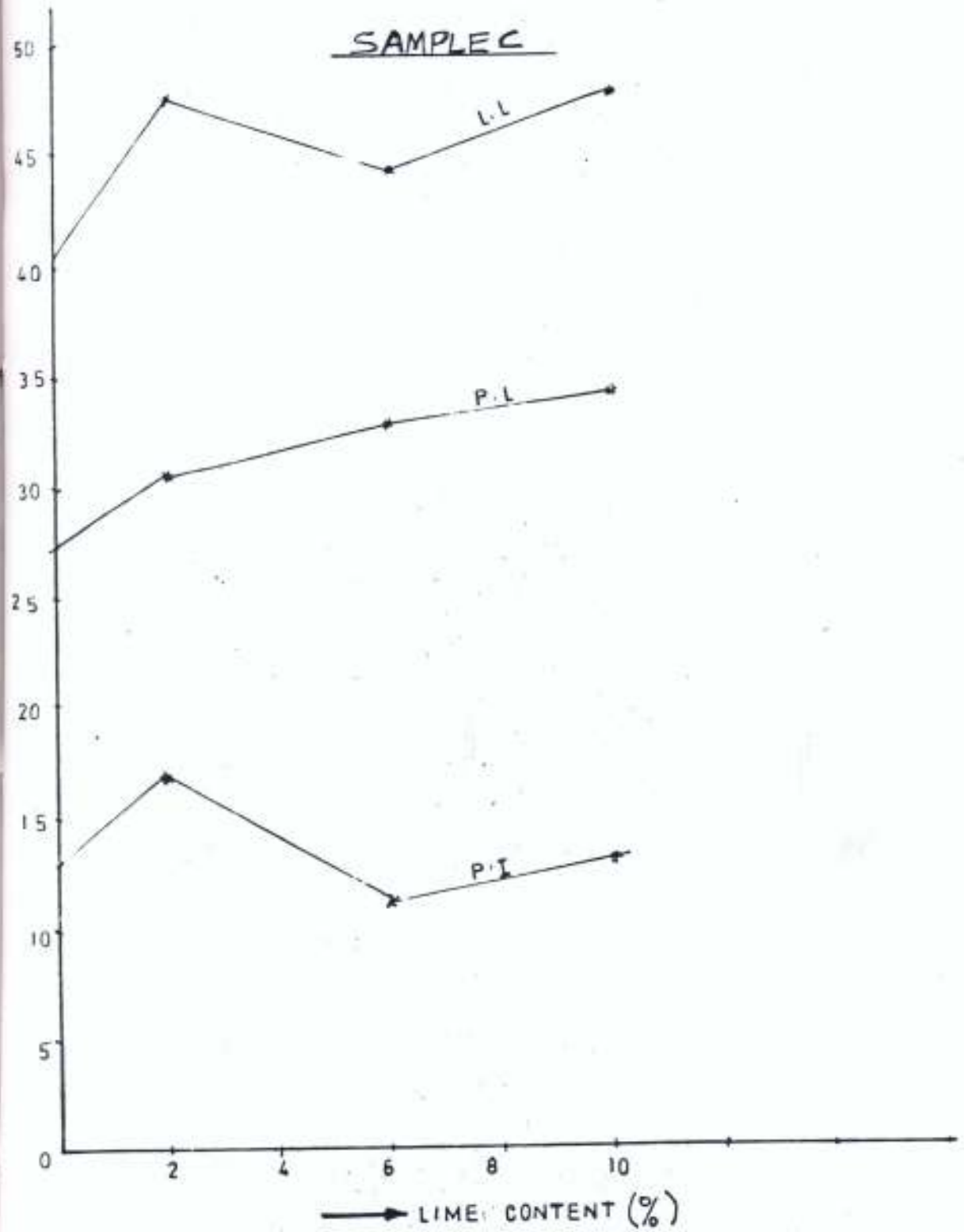


Fig. 4.16 VARIATION OF ATTERBERG LIMITS WITH LIME CONTENT

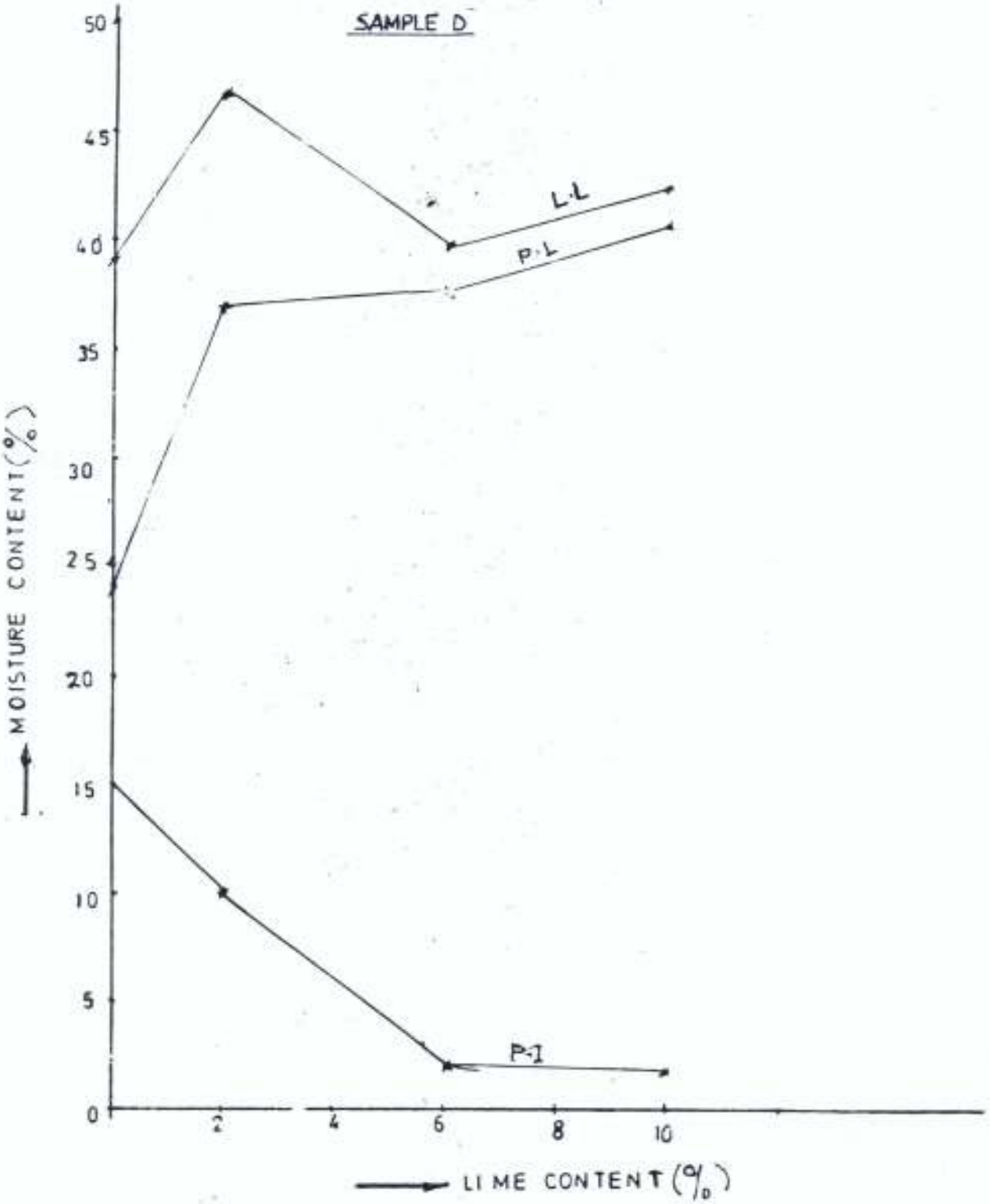


Fig 4.17 VARIATION OF ATTERBERG LIMITS WITH LIME CONTENT

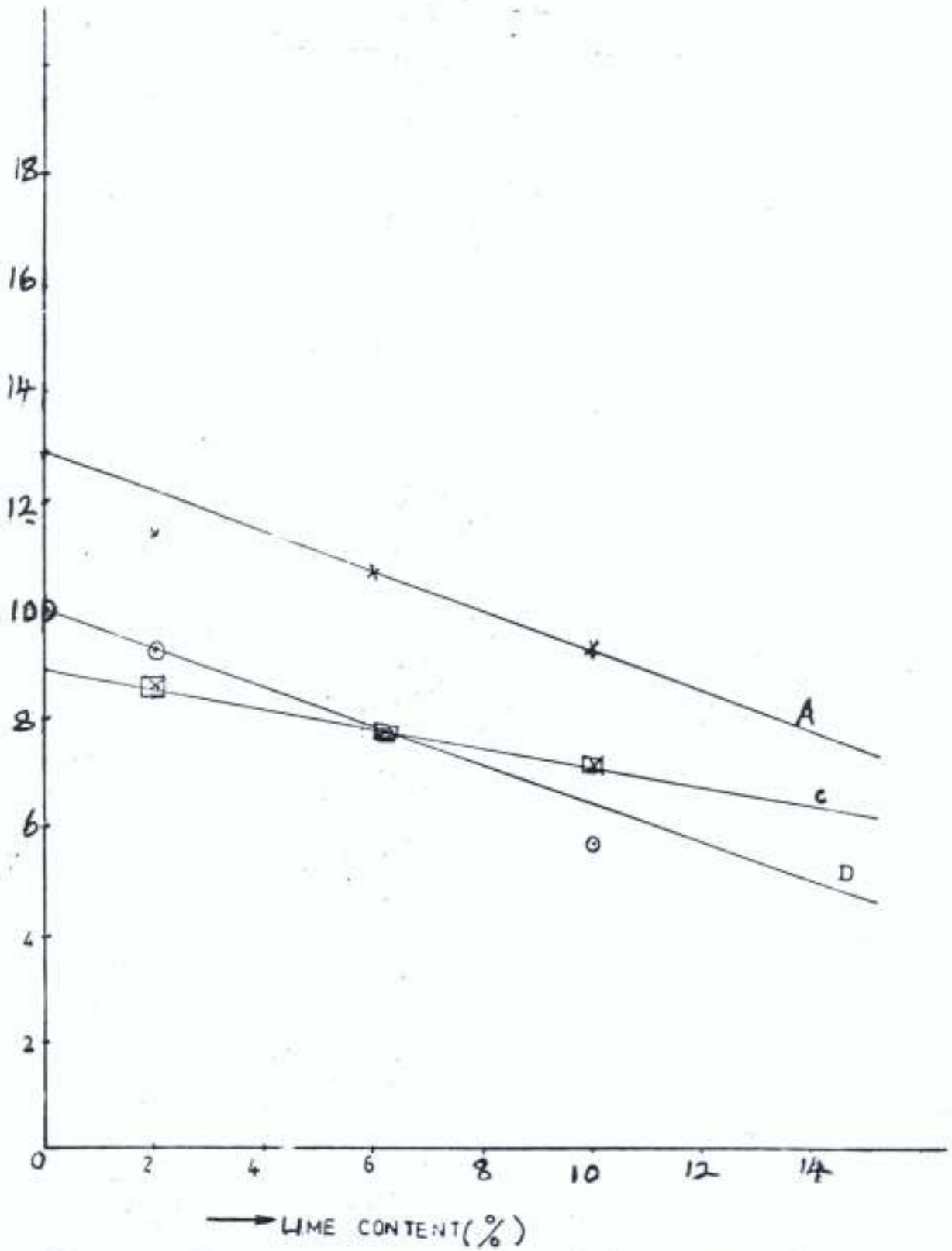


Fig4·18 VARIATION OF LINEAR SHRINKAGE WITH LIME CONTENT OF SAMPLES

SAMPLE . A

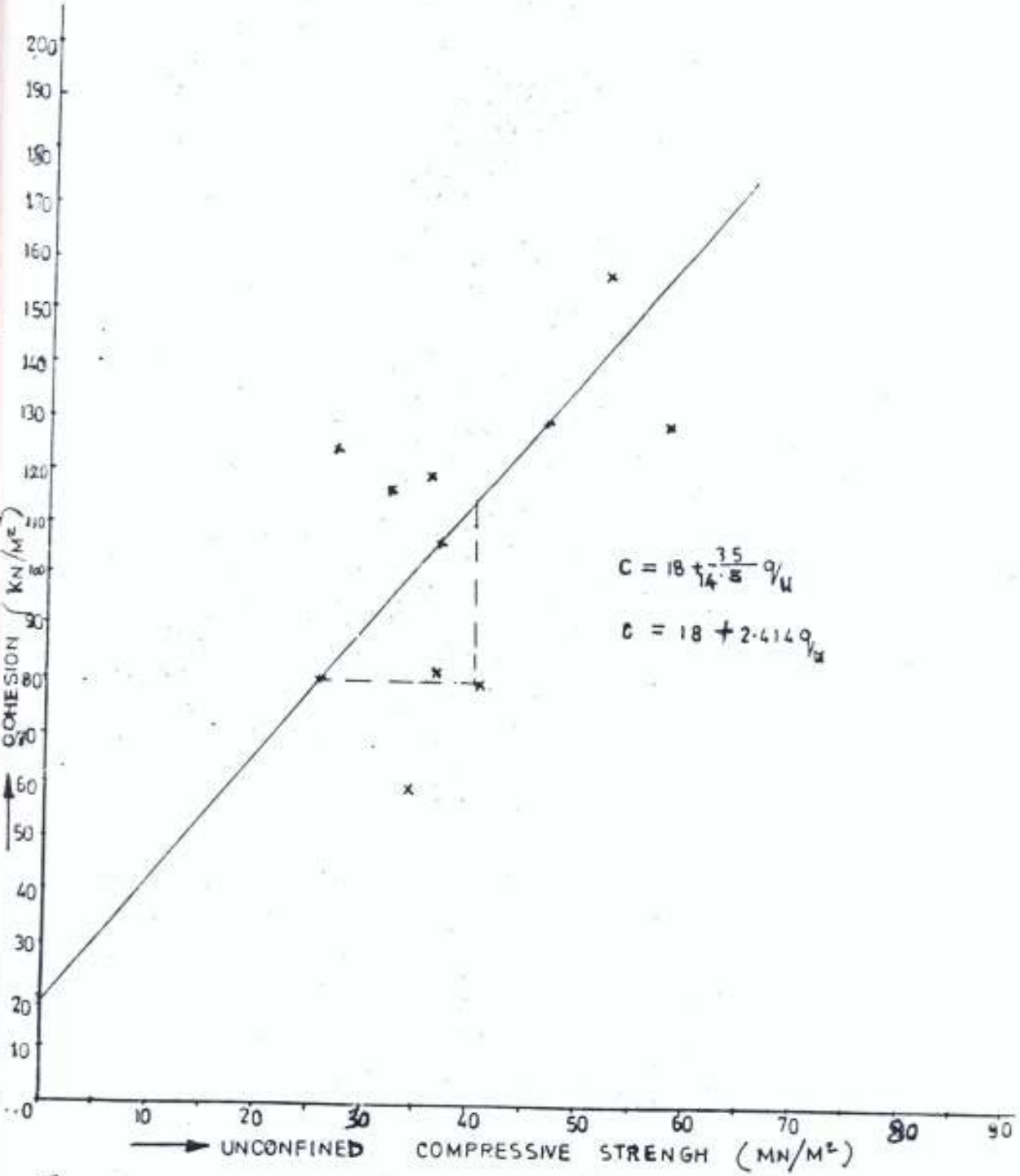


Fig 4·19 COHESION WITH UNCONFINED COMPRESSIVE STRENGTH

SAMPLE C

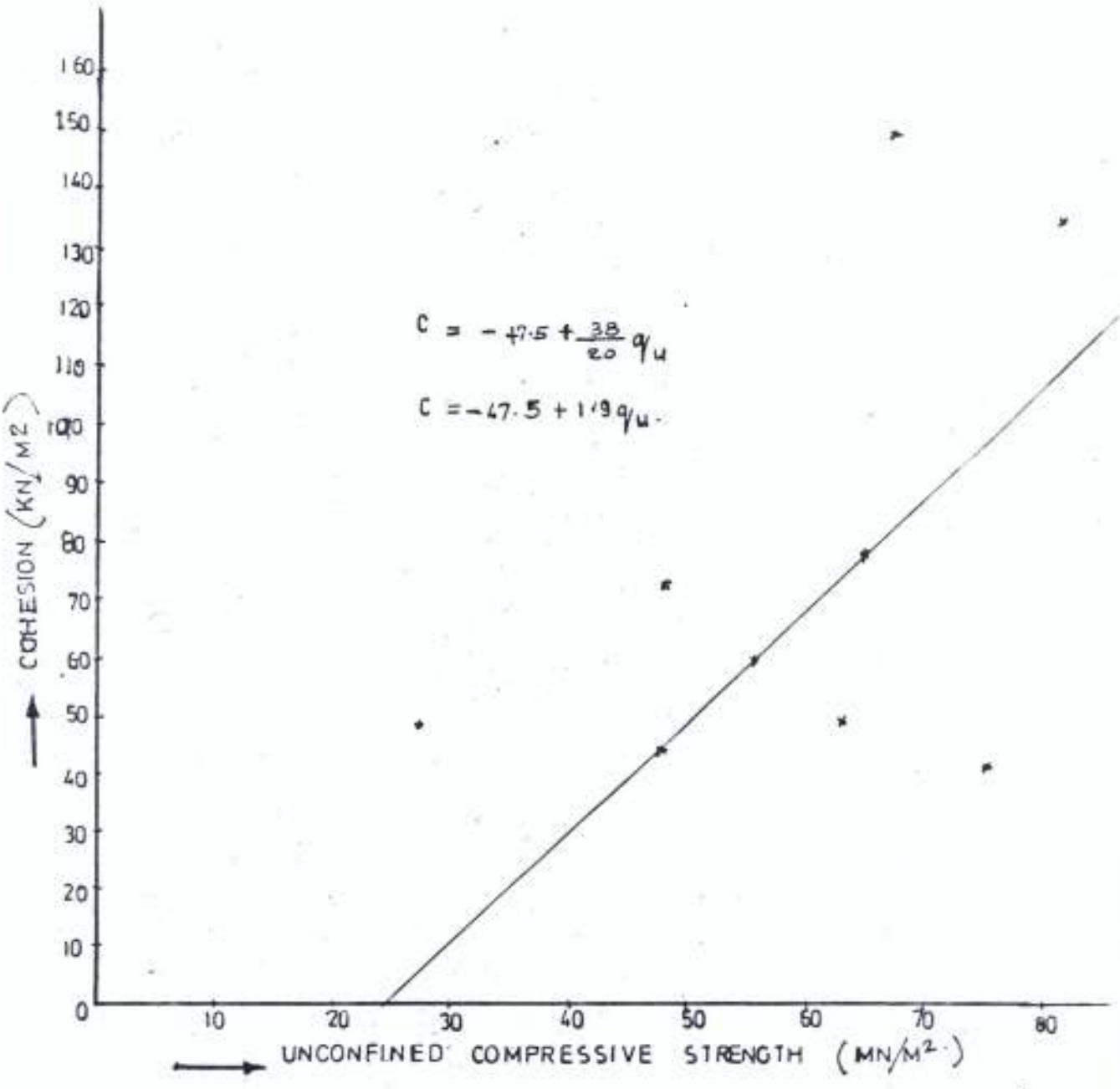


Fig. 420 COHESION WITH UNCONFINED COMPRESSIVE STRENGTH

SAMPLE D

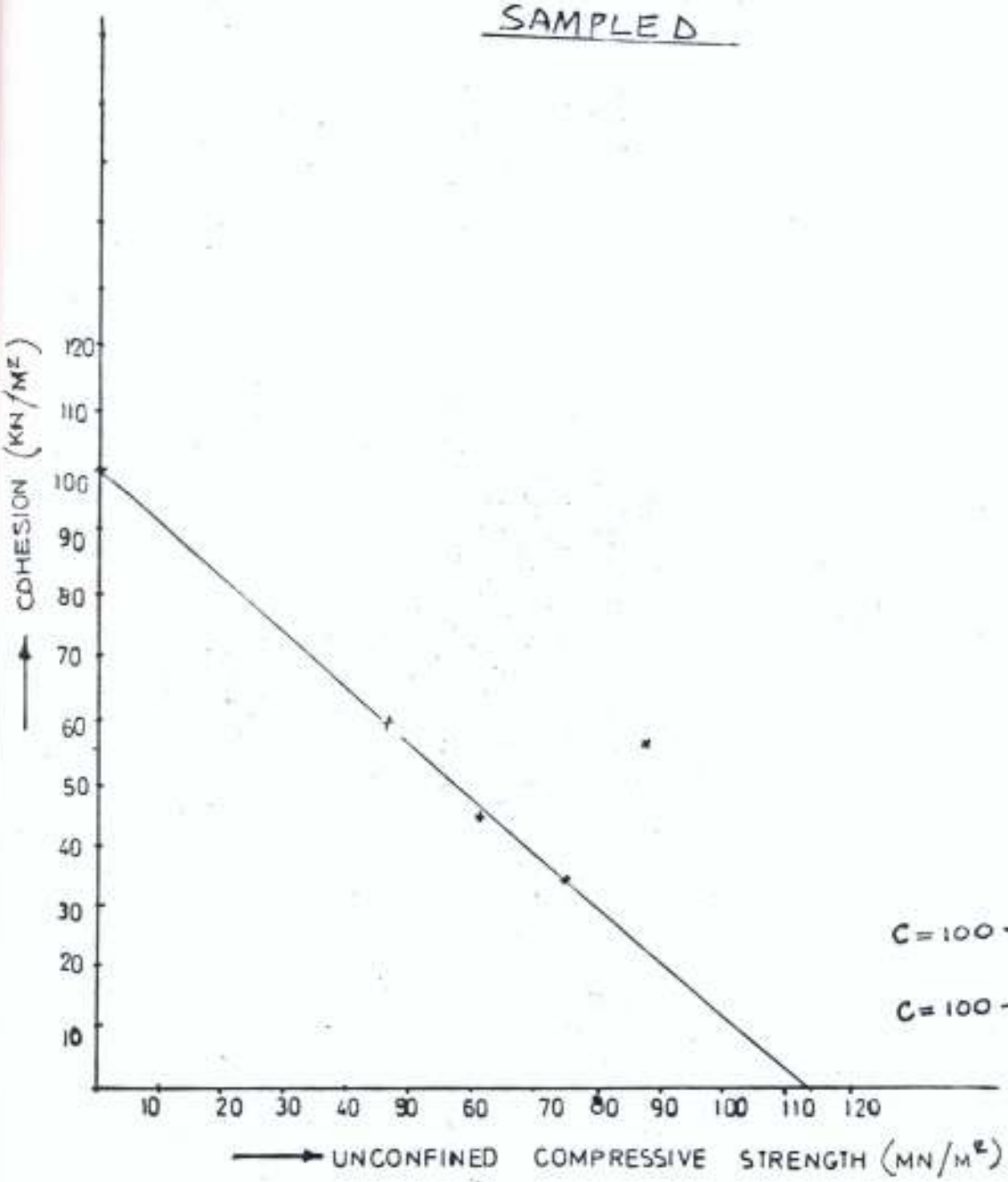


Fig 4.21 COHESION WITH UNCONFINED COMPRESSIVE STRENGTH

The variations of MDD and OMC with lime content are shown in figures 4.5 to 4.7 for samples A, C and D respectively. Figure 4.14 shows the variation of optimum moisture content with lime content.

Generally from the diagrams, it can be seen that initially the optimum moisture content decreases with increase in lime content but later increases with increase in lime content. The initial reduction in the optimum moisture content may be due to cation exchange, flocculation and agglomeration of the soil caused by the addition of lime which decreases the plasticity index of the soil.

4.2.2 Plasticity and Workability

The effect of lime on Atterberg limits of the clay soils can be seen as shown on Table 4.5 and figures 4.15-4.18.

For soil type A, the plasticity index reduced drastically from 17% for 0% lime content to 5% for 6% lime content. For soil type C, the reduction is not so appreciable but for soil D, the plasticity index reduced drastically from 15% for 0% lime content to 2% for 6% lime content. Also the plastic limit increased consistently for all the soil samples with increase in lime content. Linear shrinkage of all the soils decreases with increase in lime content thus increasing the shrinkage limit of the soils. The results obtained are as a result of flocculation effect when lime reacted with the soils.

It can also be seen that there is increase in plasticity index with increase in lime content from between 6% to 10%. This normally is explained by F.G. Bell (1993) that at times lime treatment of kaolinitic clay soil increases the plasticity index.

4.2.3 pH value

The treatment of the samples with lime content changes the pH of the samples from acidic to alkaline and the value increased with increasing lime content for all the samples. The pH value for sample A changes from 5.8 at 0% lime content to 11.3 at 10% lime content, that of sample C changes from 6.1 at 0% lime content to 11.4 at 10% lime content and that of sample D changes from 5.7 at 0% lime content to 11.4 at 10% lime content. These changes resulted from the change in chemical properties and composition of the sample due to their chemical reactions with the lime additive.

4.2.4 California Bearing Ratio (CBR)

California Bearing Ratio tests were carried out on samples A, C and D, using lime treatment at varied proportions at their various optimum moisture contents. The values of CBR are shown in Table 4.5 for cured samples and figures 4.8-4.10 show the effect of lime on CBR value of the samples. For sample types A and C, the CBR values increased drastically for cured samples and reached the maximum at 6% lime treatment; it then reduced slightly at higher values of lime treatment. For sample type A, the CBR value increased from 2% at 0% lime content at 4 days curing time to 17% at 6% lime content at 7 days curing time. At 28 days curing period the CBR value increased to 97% at 6% lime content.

For sample type C, the CBR value increased for 14% at 0% lime content at 4 days curing time to 45% at 6% lime content at 21days curing time. For sample type D, the CBR values increased drastically for cured samples. The CBR value at 10% lime content gives a better increase in strength. The CBR value increased from 24% at 0% lime content at 4 days curing time to 98% at 10% lime content for 28 days.

The CBR values increase with age for all the samples. Curing time is a strong factor in the development of strength of lime treated lateritic soils. Type of soil is also a factor in the development of strength of lime treated soils. There is always a sharp increase in strength of soil treated with lime in the first 7 days, then gradual increment for the subsequent days. These immediate effects which give a substantial improvement in strength and stability of the soil within the first 7 days can be expected for highly plastic cohesive soils especially sample type A. The effect of curing time on the samples treated with lime content is as shown in Figures 4.11-4.13. The CBR values increase with increase in curing time due to a pozzolanic reaction that takes place in the soil-lime mixture.

Unconfined Compressive Strength Test (UCS)

To determine the suitability of samples for treatment with lime content, unconfined compressive strength tests were carried out on the three samples as mentioned earlier. The samples were tested at 7, 21 and 28 days to determine the effect of age on strength of the samples. Table 4.5 shows results of unconfined compressive strength on the samples treated with varying percentages of lime. The variation of unconfined compressive strength with lime content for samples A, C and D are shown in Fig 4.8-4.10 while the effect of curing time on the samples treated with lime as shown in Figures 4.11-4.13.

The effects are similar to those obtained for CBR tests. For sample types A and C, the UCS values increased substantially for both cured and uncured lime treated soils and reached a maximum at 6% lime treatment; it then reduces slightly at higher values of lime treatment. For sample type A, the UCS increased from 1.41MN/m^2 at 0% lime content at 7 days curing time to 3.6MN/m^2 at 6% lime content at 7 day curing time. At 28-day curing

period, the UCS increased to 5.21MN/m^2 at 6% lime content. For sample type C, the UCS increased from 2.23 MN/m^2 at 0% lime content at 7 days curing time to 6.52MN/m^2 at 10% lime content at 7 days curing time. At 28 days curing period, the UCS increased to 8.37MN/m^2 at 10% lime content. For sample type D, the UCS increased substantially for cured samples. The UCS increased from 3.6MN/m^2 at 0% lime content at 7 days curing time to 6.14MN/m^2 at 10% lime content. At 28 days curing period the UCS increased to7.06MN/m^2 at 10% lime content.

The addition of lime to all the samples is beneficial in that it gives strength to the soils. The rate of increase in strength for sample type A is highest and is followed by that of sample type C. The difference between the unconfined compressive strength of the natural and lime treated samples has been used as indication of the degree to which the soil-lime pozzolanic reaction has proceeded. Substantial strength increase indicates that the soil is reactive with lime and can probably be stabilized to produce a good base and subbase material.

Shear Strength Parameters (C - Ø)

The cohesion values of natural samples are lower than the cohesion values of the lime treated soils. The angle of friction also follows the same trend.

For natural soils, the cohesion and angle of friction values for soil type D give the highest values while those of soil type A give the least values as shown in Table 4.5.

For sample type A, the cohesion increased form 17.5kN/m^2 at 0% lime content to 158MN/m^2 at 6% lime content at 28 days curing time. For sample type C, the cohesion increased from 30kN/m^2 at 0% lime content to 153 kN/m^2 at 6% lime content at 28days

curing time. For sample type D, cohesion decrease from 65kN/m^2 at 0% lime content to 60kN/m^2 at 0% lime content at 7 days curing time. For sample types A and C, the cohesion of the mixtures was substantially increased compared to the natural soils and cohesion continued to increase with increased unconfined compressive strength. Using the linear regression equation shown in Figures 4.19 to 4.21, cohesion values can be estimated from unconfined compressive strength results. For soil type D, the cohesion of the mixtures was substantially reduced compared to the natural soils but the linear regression equation shown in Figure 4.21 shows that cohesion reduced with increased unconfined compressive strength.

For sample types A and C, large shear strengths can easily be developed when they are treated with lime.

4.7 Compressive Strength (Crushing)

Compressive strength tests were carried out on treated samples to determine the compressive strength of the treated samples with varying proportions of the additive.

Results of compressive strength test are given on Table 4.5. While the samples at 0% lime content gave negligible values, lime treated samples gave appreciable results. For sample type A, the average compressive strength values increased from zero at 0% lime content to 0.88N/mm^2 at 6% lime content for 28 days curing period. For sample type C, the average compressive strength values increased from zero at 0% lime content to 1.18N/mm^2 at 6% lime content for 28 days curing period. For sample type D, the average compressive strength values increased from zero at 0% lime content to 1.30N/mm^2 at 10% lime content for 28 days curing period.

Sample type D gives the highest values of compressive strength while type A gives

the least values.

Ola (1983) stated that the minimum acceptable compressive strength for stabilized soil as base material is 1.03N/mm^2 which indicates that sample type A should not be used as base material even after being stabilized with lime. Sample type C could be used if allowed to cure for at least 28 days while sample type D could be used after curing for 28 days with a reasonable lime content of 2%.

Stress-Strain Characteristics

Stress-Strain properties are essential for properly analyzing the behavioural characteristics of a pavement structure containing a soil-lime structural layer. The marked effect of lime on the compressive stress-strain properties of fine grained soil are shown in Fig. 4.22-4.24: the failure stress is increased, and the ultimate strain is decreased for soil-lime mixtures relative to the natural soils. For all the lime-treated and untreated samples, the failure stress for uncured samples are higher than for cured samples. But the ultimate strain does not follow a particular pattern. It either decreases or increases. The results of the tests as shown in Table 4.6.

Table 4.6: Summary of Test Results Containing Stress and Strain of both Lime Treated and Untreated Samples

Strain x 10^{-4}	Average Stress KN/m²											
	Sample Type A with Lime Content				Sample Type C with Lime Content				Sample Type D with Lime Content			
	0% Cured	*0% Uncured*	*10% Cured*	*10% Uncured*	*0% Cured*	*0% Uncured*	*2% Cured*	*2% Uncured*	*0% Cured*	*0% Uncured*	*10% Cured*	*10% Uncured*
32.9	21.1	42.2	179.3	184.6	24.6	52.7	87.9	43.9	56.2	52.7	43.9	254.9
65.79	29.8	63.1	219.0	324.1	31.5	73.6	140.1	113.9	94.6	77.1	56.1	471.0
98.68	34.9	80.3	253.2	523.8	38.4	101.3	202.5	164.1	132.7	90.8	66.4	635.6
131.58	38.3	88.7	304.5	629.9	43.5	121.8	234.9	217.5	161.8	100.9	74.8	713.4
164.47	41.6	97.1	381.6	662.6	46.8	142.2	249.8	253.2	187.3	111.0	86.7	756.2
197.37	43.2	100.2	466.6	698.2	50.1	159.0	271.3	293.8	207.4	119.2	96.8	756.9
230.26	46.5	106.8	551.1	733.6	53.4	175.7	292.8	318.6	230.8	127.4	103.3	773.2
263.16	48.0	113.3	652.1	751.6	58.3	192.2	317.5	329.5	243.7	137.3	111.5	779.1
296.05	49.6	116.3	718.1	755.7	59.8	201.8	328.3	357.4	259.9	147.0	116.3	783.1
328.95	51.1	121.0	766.7		63.0	211.3	330.5	366.3	269.2	156.7	126.1	787.1
361.84	52.6	123.9	791.0		66.2	220.7	331.0	373.4	278.4	161.2	134.1	791.0
394.74	54.2	128.7	808.0		67.8	228.7	335.4	381.1	288.0	166.0	138.9	794.1
427.63	57.3	131.4	808.8		69.1	232.5	337.0	385.8	293.2	171.9	148.3	
460.53	58.7	134.3	807.3		68.8	238.3	337.4	387.7	298.8	176.2	154.4	
493.42	58.5	137.1			70.2	242.5	337.8	389.6	304.3	182.3	157.2	
526.32	60.1	138.7			73.5	247.3	342.5	398.9	307.4	187.1	162.1	

559.21	61.6	141.5			76.6	249.7	343.0	389.6	319.7	189.8	166.5	
592.11	61.4	142.7			79.6	253.8	348.4	389.9	325.2	192.5	169.2	
625.00	62.8	143.8			81.0	256.2	348.7	390.0	327.2	198.3	171.9	
657.90	64.3	145.0			82.4	258.6		390.4	331.1	201.0	173.0	
690.79	65.7	146.1			82.1	262.7		397.3	333.3	201.9	177.3	
723.68	67.1	147.2			83.4	263.4		400.8	333.7	202.8	180.0	
736.59	68.5	150.0			84.8	264.1			334.1	205.4	184.2	
789.47	68.2	150.9			84.4	266.2			334.3	206.1	188.3	
822.37	69.6	152.1			85.8	267.1			336.7	205.6	191.0	
835.26	69.3	153.2			85.5	267.7			338.7	206.4	191.9	
888.16	70.7	154.3			86.8				340.6	207.3	192.8	
921.05	72.0	155.3			89.7				341.0	208.1	193.7	
953.95		156.3			90.9				343.0		196.2	

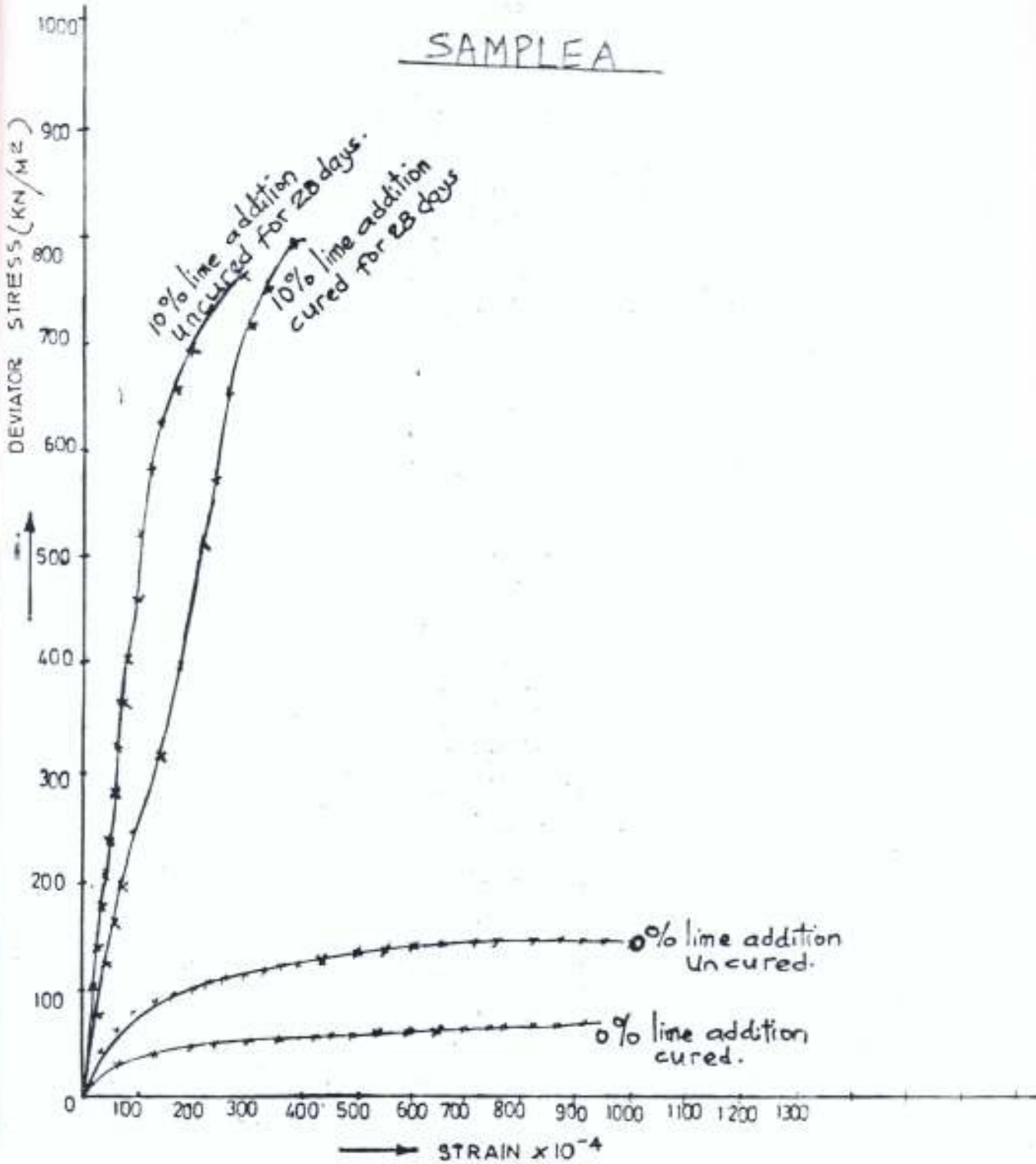


Fig 4.22 STRESS-STRAIN CURVE

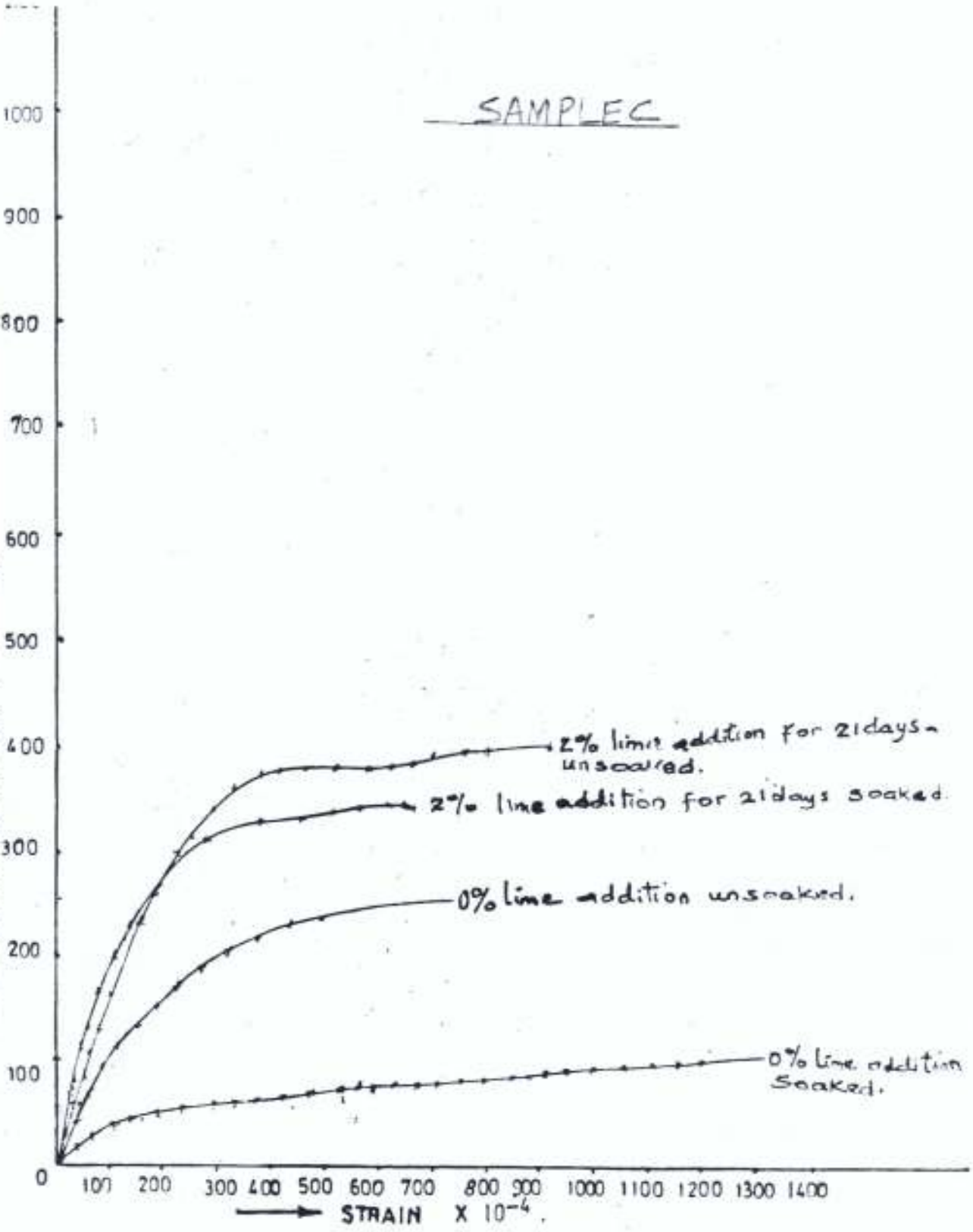


Fig4·23 STRESS – STRAIN CURVE

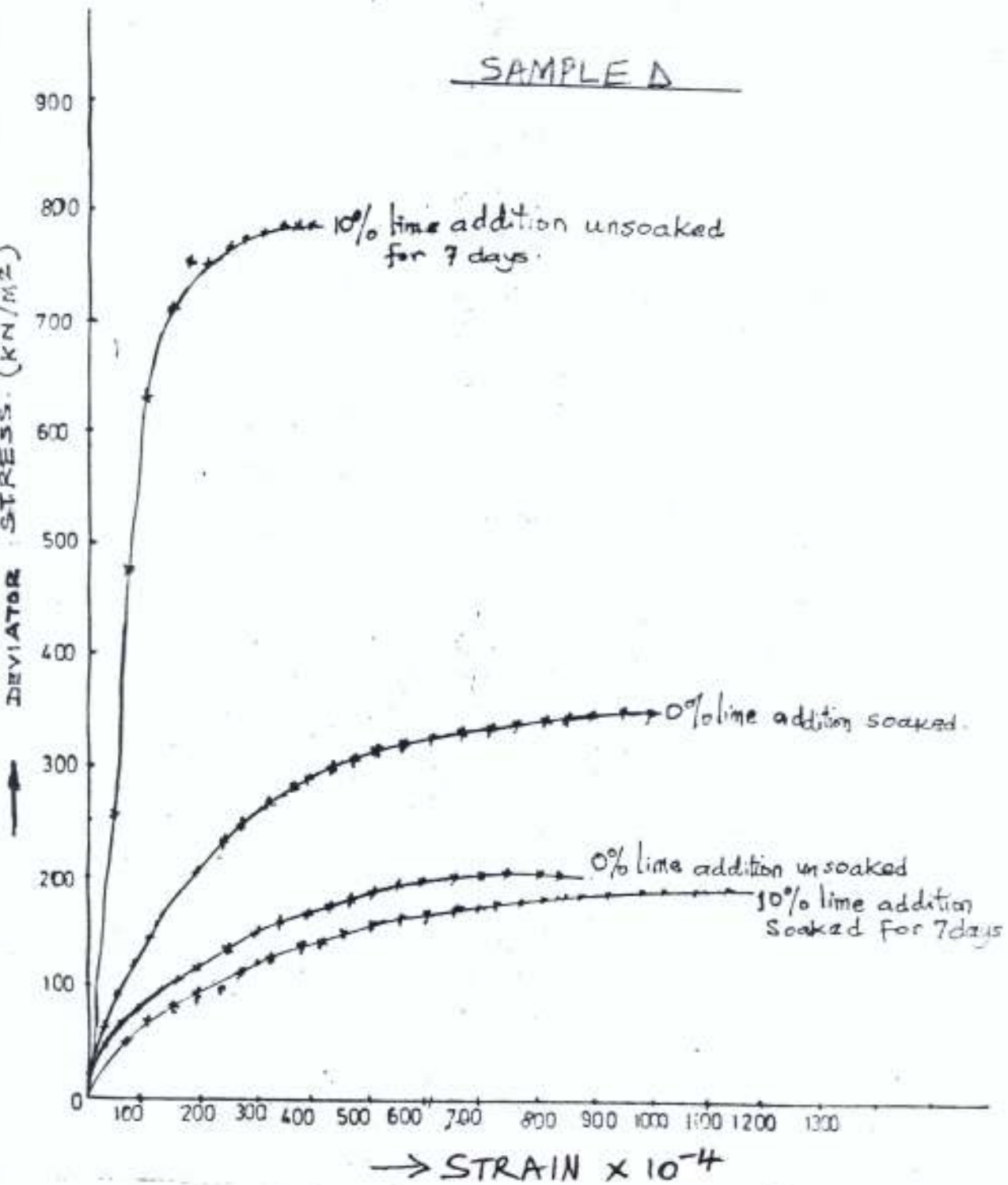


Fig4.24 STRESS – STRAIN CURVE

CHAPTER FIVE

DISCUSSION OF RESULTS

OBSERVATIONS ON THE RESULTS

Results of the various tests carried out on the samples show that the addition of lime to natural samples resulted in increased strength over the natural samples. This is in accordance with the works of other investigators like **Murphy (1991), Ola (1983)** and **Lundy Jr.** and **Greenfield (1968).**

The physical inspection of the samples revealed that their colour varies from brownish red clay to brownish gravelly clay. Sample type A is yellowish brown clay, sample type B is brownish gravelly clay, sample type C is brownish red clay, sample type D is brownish red clay and sample type E is brownish red clay.

The natural samples have the following properties, the percentage of fine grains ranges from 26% to 58.4%; their plastic limit ranges from 16% to 27%; their plasticity index ranges from 12% to 17%; their specific gravity ranges from 2.35 to 2.59; their pH values ranges from 5.7 to 6.5; their maximum dry density ranges from 1.8g/cc to 2.2g/cc; their optimum moisture content ranges from 11 to 16; their group index ranges from 0 to 8. The soils are A-2-6 and A-6 types bases on AASHTO Classification system.

From the clay identification chart; using plastic limit and plasticity index as parameters (after **Brain 1971**), all the soil samples have kaolinite type of minerals. From the plasticity chart for classification of cohesive soils (after Casangrade 1932), soil types A, C, D and E are inorganic silty clay and have medium compressibility while soil type B is in

organic clay and has low compressibility. Based on AASHTO classification system rating as subgrade material, soil types B and D are rated between excellent and good while soil types A, C and E are rated between fair and poor. This is an indication that soil types A, C and E are not good for use as subgrade material except they are stabilized.

The chemical composition show that the silica to sesquioxides ratio ($SiO_2/Al_2O_3 + Fe_2O_3$) for soil type A is less than 1.33 which is indicative of laterites while other types of soil are lateritic soils because their silica to sesquioxides ratio is greater than 1.33 and less than 2.

CBR, UCS, cohesion and angle of friction values are highest for sample type D and least for sample type A. Soil type D has the highest strength while soil type A has the least.

The addition of lime to the samples generally increases the optimum moisture content and reduces the maximum dry densities as lime content is increased with the rate of reduction more pronounced between 0% and 2% lime content. In the lime treated samples, the plasticity index also reduced drastically: For sample type A, PI value reduced from 17% to 5% at 6% lime content; for sample type C, the value reduced from 13% to 11% at 6% lime content and for sample type D, the value reduced from 15% to 2% at 6% lime content. The plastic limit increased consistently for all treated samples with increase in lime content. Linear shrinkage of all the soils decreases with increase in lime content thus increasing the shrinkage limit of the samples. As explained by **Davidson** and **Handy (1960)**, calcium ions from lime cause a reduction in the plasticity of cohesive soils which make them to become more silty and friable thus causing a significant improvement in workability and this expedites subsequent manipulation and working of the lime treated soils.

The CBR of treated samples increased drastically from 2% to 97% for sample type A at 6% lime content which makes it to satisfy the CBR criterion of 80% based on FMW highway manual. The soil after treatment with lime will become useful as highway construction material. CBR values increased with increasing age of the treated sample. Age is a strong factor in the development of strength of lime treated lateritic soils. The sharp increase in strength of soil treated with lime in the first 7 days is as a result of chemical reaction taking place between the lime and the soil which impacts increased strength and stability into the soil especially the highly plastic cohesive soils like type A.

The UCS of treated samples increased also drastically from 1.41MN/m^2 to 3.6MN/m^2 for sample type A within the first 7 days which makes it to satisfy the UCS criterion of 1.7MN/m^2 thereby making the soil suitable as highway construction material. The strength increase is as a result of soil-lime pozzolanic reactions.

The cohesion and angle of friction parameters of shear strength both increase with increasing lime content. The strength developed by the mixture would be adequate to prevent shear failure.

OTHER BENEFITS OF LIME TREATED SOILS

The main benefits obtained when lateritic soil is treated with lime are improved workability, increase in strength and volume stability. Lime increases the optimum moisture content for compaction which is an advantage when dealing with wet soils. It reduces the variability of the density produced, while path rutting and pot holing are reduced resulting in low maintenance costs. It forms an effective moisture barrier to downward percolation or capillary action.

Lateritic soils stabilized with lime have other uses e.g. for providing low cost housing units. It can be used as load bearing walls and partition walls.

In the construction of stabilized soil roads, cost of construction can be brought down considerably by stabilizing locally available poor soils with lime. Lime being a cheap stabilizer which is effective for stabilizing clayey soils with high plasticity reduces the thickness of water film surrounding individual soil particles and impacts strength and makes the mixtures more durable and stable. As much as 50% of saving can be effected by using lateritic soils treated with lime over the costly stone aggregates for road construction **(Vazirani and Chandola, 1984)**.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The properties and characteristics of lime treated lateritic soils vary significantly depending on the type of soil, amount of additive, type of additive and curing conditions including time, temperature and moisture.

The results of this investigation have showed that beneficial effects are obtained by the addition of small amounts of lime to lateritic soils and specifically the results lead to the following conclusions:

(i) The dry density of the mixture decreases with increase in lime content with the rate of reduction of maximum dry density being more between 0% and 2% lime content while the optimum moisture content increases with increase in lime content.

(ii) The plastic limit and the liquid limit of the mixture increase with increase in lime content while the plasticity index of the mixture reduces consistently with increase in lime content from 0% to 6% but increased slightly from 6% to 10%.

(iii) The linear shrinkage and swell of the mixture decrease markedly.

(iv) Tests on both the cured and uncured mixes produced increases in CBR value with increasing curing time and lime content until it reaches the maximum at 6% lime content for soil types A and C and subsequently decreased with lime content but for soil type D, it increased up to 10% lime content.

(v) The unconfined compressive strength of the mixture increases with increase in

curing time and lime content until it reached the maximum and subsequently decreased with increasing lime content.

(vi) The shear strength of the mixture increased with increase in curing time and lime content.

(vii) The failure stress of the mixture increased with increase in lime content while the ultimate strain reduced with increasing lime content. The Young's modulus of the mixture is thereby increased with increasing lime content.

(viii) Soil types C and D with 6% lime treatment can be used as base material in pavement construction while soil type A with 6% lime treatment can be used as sub-base material according to Federal Ministry of Works of Nigeria highway standard.

(ix) Lime treatment of lateritic soils improve their characteristics but at different rates. It is best suited for highly plastic clayey soils.

(x) The minimum lime content required for treatment of the lateritic soils for use as pavement construction material is 6%.

RECOMMENDATION

On the basis of the test results, observations and conclusions, the following recommendations are being made:

(i) The use of lime as a soil improvement technique for laterites and lateritic soils should be encouraged for heavy clay soils like soil sample type A.

(ii) This type of investigation should be extended to other types of lateritic soil deposits in Nigeria.

(iii) Lime processing company should be established by the necessary governmental agency to reduce the cost of lime. Since the lime used in this investigation is the imported white hydrated high calcium lime. There is also a large deposit of lime stone in Nigeria.

(iv) Field tests on roads with lime stabilized laterite base should be encouraged.

REFERENCES

deyeri, J. B. (1988): Engineering Characteristics of some Nigerian residual soils for Highway Construction.Transportation Research Records, No 1192 pages 39-50.

deyeri, J. B., Iloghalu, O. A. And Eruchalu, O. E. (1994): Stablization of some lateritic soils with cement. Geotechnical Lab. Report. GLR02, Civil Engr. Dept. OSUA.

nifowose, A. Y. B. (1989): "The performance of some soils under stabilization in Ondo State, Nigeria". Bulletin of the International Association of Engineering Geology No 40 Paris.

shworth, R. (1979): "Highway Engineering" Heine man Education Books Ltd.

aker, Ryrd and Mickle (1975): "Handbook of Highway Engineering" Van NoS trand Reinhold Ltd.

ell, F. G. (1989): "Lime stabilization of clay soils" Bulletin of the International Association of Engineering Geology No 39 Paris.

ell, F. G. (1993): "Engineering Treatment of soils" E & EN SPON publishers.

rry, P. L. and Reid, D. "An Introduction to soil Mechanics" Butterworth, London

wles, J. E. (1984): "Physical and Geotechnical properties of soils" McGraw - Hill Inc.

rthy, D. F. (1993): "Essentials of Soil Mechanics and Foundations Basic Geotechnics", Regents Prentice Hall Inc.

s, B. M. (1990): "Principles of Foundation Engineering" PWS-Kent publishing Company.

Gaft Johnson, J. W. S. And Bhatia, H. S. (1969): Engineering properties of laterites, proc. 7th Int. Conf. On soil mechanics and foundation. Eng, Vol. 2 specialty session.

rucher, K. N. and Korfiatis, G. P. (1988): "Materials for civil and Highway Engineers" Prentice Hall Englewood Cliffs, New Jersery.

nlop,W. A., Gallaway, B. M. and Grubbs, E. C. (1962): "Recent investigations on use of a fatty Quaternary Ammonium chloride as a soil stabilizing agent". Highway Research Board Bulletin 357.

nsmann, M. R. (1990): "Engineering principles of Ground Modification". Regents Prentice Hall Inc.

Harris, F. (1994): "Modern Construction and Ground Engineering Equipment and Methods" Longman Scientific and Technical.

Harris, V. A. P. (1987): "Materials - sources and uses" Developing world land Transport Grosvenor Press International Ltd. London.

Hemwall, J. B., Davidson, D. T. and Scott, H. H. (1962): "Stabilization of soil with 4-Tert-Butylpyincatechol" Highway Research Board Bulletin 357.

Hunt, R. F. (1986): "Geotechnical Engineering Analysis & Evaluation" McGraw - Hill Inc.

Idigasu, M. D. (1967): Laterite soil Engineering pedogenesis and Engineering Principles, Elsevier scientific publishing Company, Amsterdam.

Iuiseppe, F. K. (1969): Engineering Classification of characteristics of laterite soils, Proc. Of the speciality session of Engr properties of laterite soils vol 1.

Ingles, O. G. and Metcalf, J. B. (1973): Soil stabilization principles and practices, Butterworth, London.

Izdi, A. (1979): Stabilized Earth Roads - Development in Geotechnical Engineering" Elsevier Scientific publishing Company.

e, I. K., White, W. and Ingles, O. G. (1983): "Geotechnical Engineering" Butterworth, London.

ttle, A. L. (1969): Engineering properties of Lateritic soils (vii) Int. Conf. Soil Mechanics Fdn. Engg Mexico.

ndy, H. L. Jr. and Greenfield, R. J. (1968): "Evaluation of Deep in-situ soil stabilization by high pressure lime slurry injection" Highway Research Record No 235 Swindell-Dressler Company, Pittsburgh Pa.

rtins, C. F and Doyne, M. A. (1930): Laterite and lateritic soils in Sierra-leone, Jornal of Agric. Science vol. xvii.

rritt, F. S. and Max kurtz, P. E. (1983): "Civil Engineering Reference Guide" McGraw - Hill Book Company.

rthy, V. N. S. (1991): "Soil Mechanics and Foundation Engineering vol. 2" Sai Technical Consultants.

Flaherty, C. A. (1975): "Highway Engineering vol. 2". Edward Arnold publishers, London.

Oglesby, C. H. (1975): "Highway Engineering" John Wiley and Sons

Ola, S. A. (1983): "Tropical soils of Nigeria in Engineering Practice" Balkema A. A.

Oluyemi - Ayibiowu, B. D. (1999): "Characteristics of Cement - Modified Asphalt - Emulsion Laterite Mixes" Unpublished M. Eng. Thesis, Department of Civil Engineering, Federal University of Technology, Akure.

Paquette, R. J. and Mcqcee, J. D. (1961): "Evaluation of Strength properties of several soils treated with Admixtures" Highway Research Board Bulletin 282.

Penrifoy, R. I. (1982): "Construction planning, Equipment and Methods" MC Graw-Hill International Book Company.

Singh, G. and Singh, J. (1983): "Highway Engineering" Standard publishers Distributions.

State of the Art: Lime Stabilization - Reaction, properties, Design, Construction" Transportation Research circular No. 180 (Sept. 1976) prepared by Transportation Research Board Committee on lime and lime flyash stabilization.

Uzirani, V. N. and Chandola, S. P. (1984): "Highway and soil Engineering" Romesh chander Khanna Delhi.

Watson, J. (1991): "Highway Construction and maintenance" John Wiley and Sons Inc. New York.

Winterkorn, H. F. and Fang, H. (1975): "Foundation Engineering Handbook" Van. Nostrand Reinhold Company.

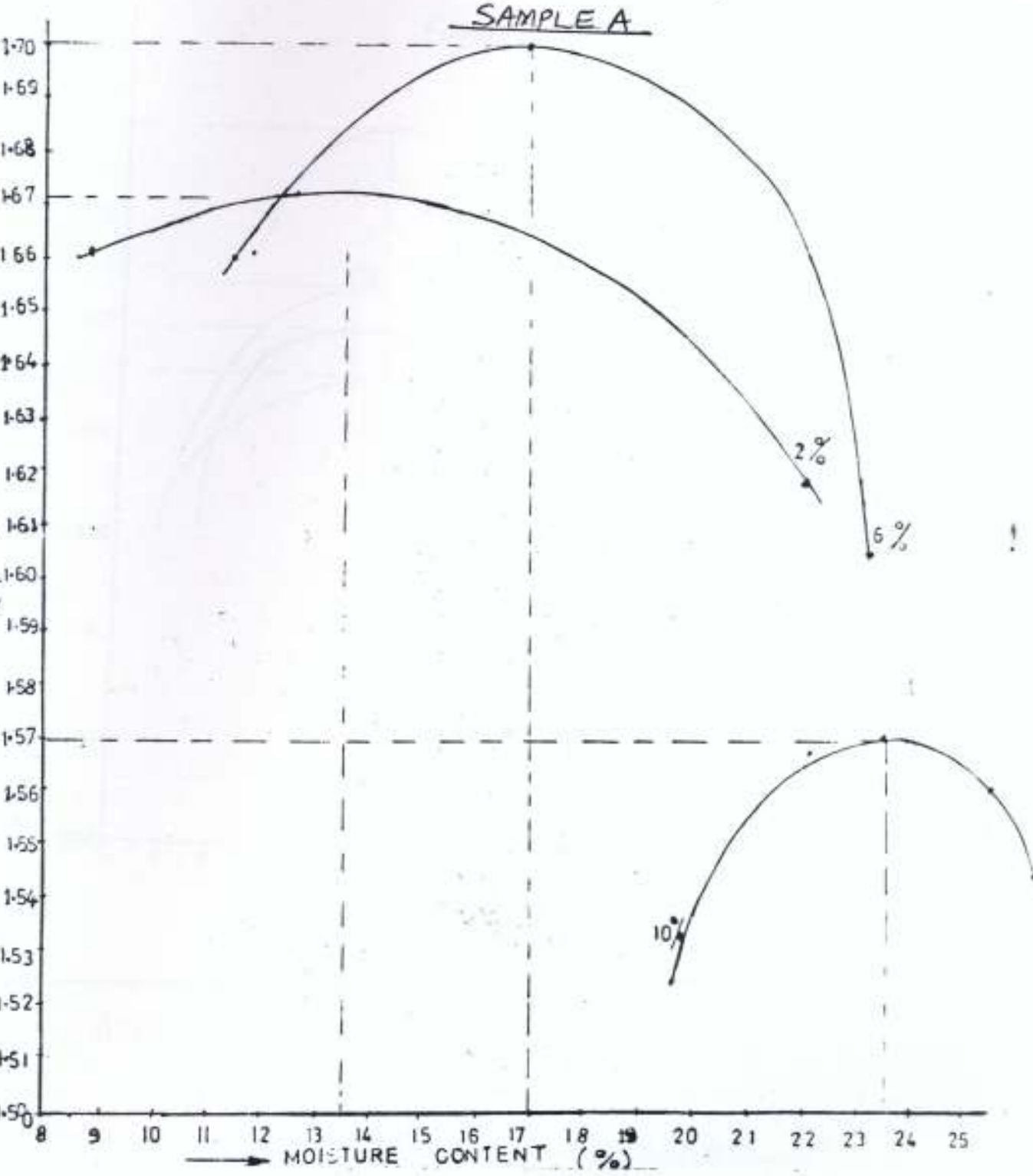
Woods, K. B. (1960): "Highway Engineering Handbook" McGraw - Hill Inc.

Wright, P. H. and Paquette, R. J. (1987): "Highway Engineering" John Wiley and Sons Inc.

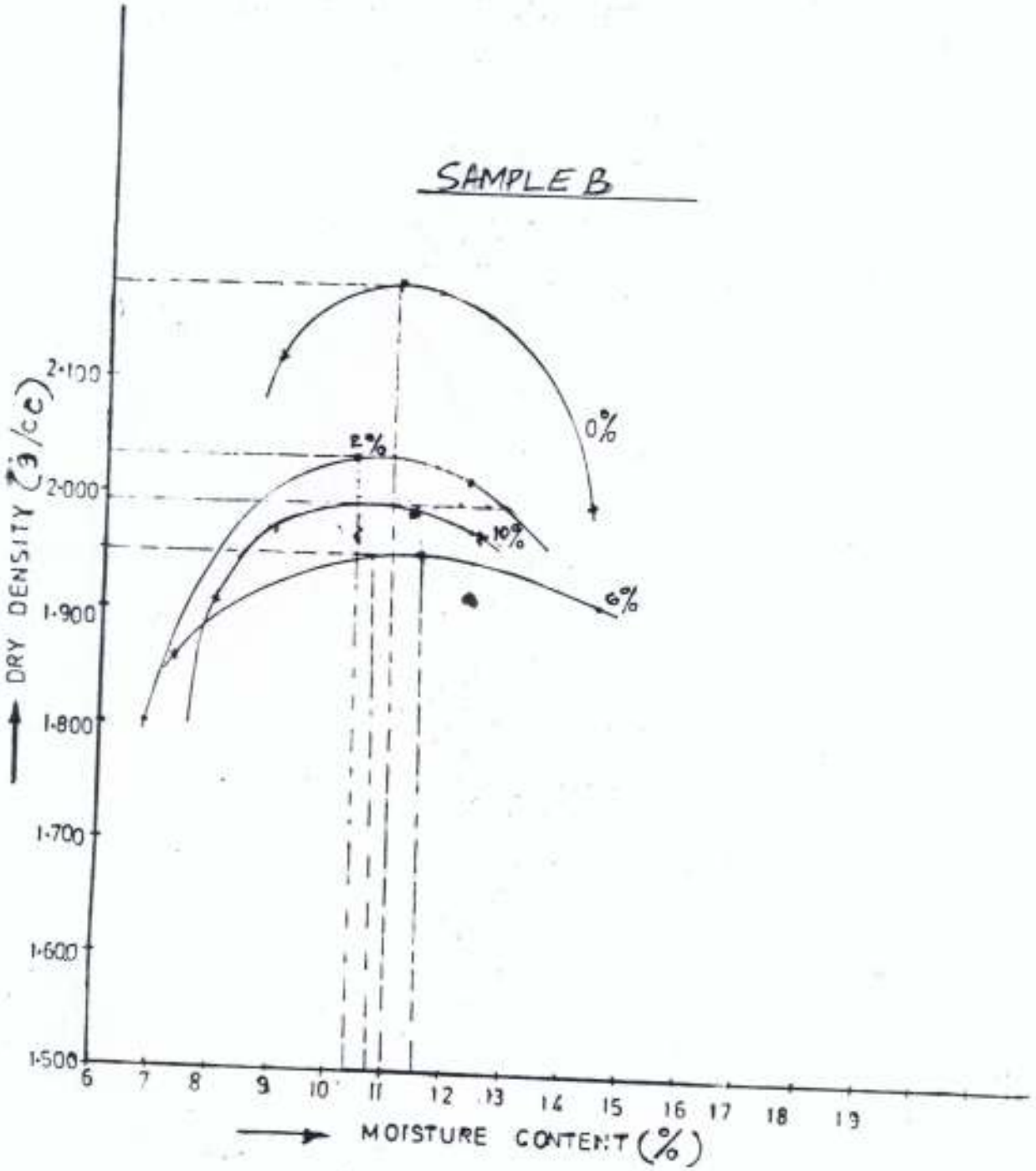
APPENDIX

Compaction Table for both Lime Treated and Untreated Samples

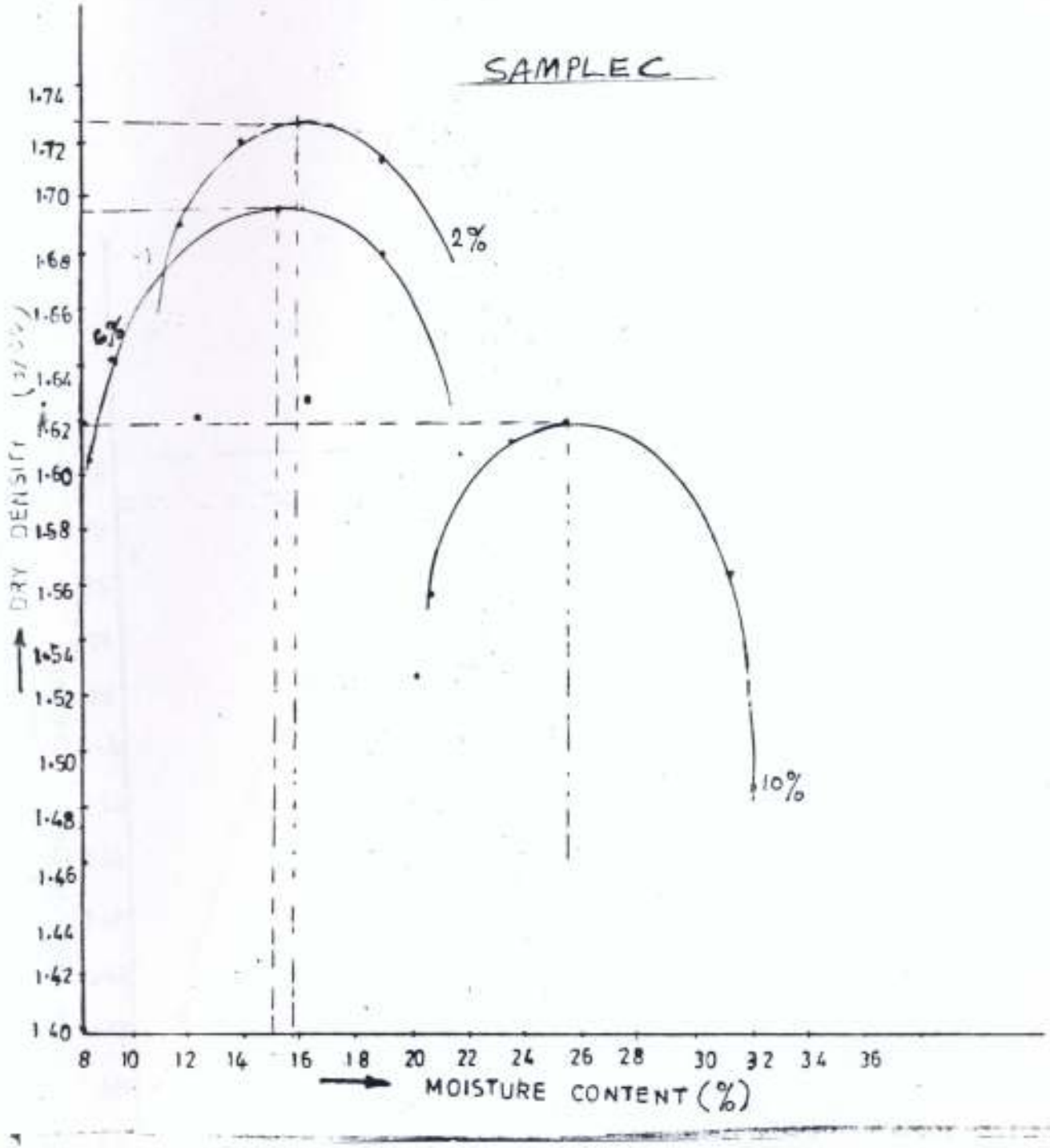
SOIL TYPE	TYPE A				TYPE B				TYPE C				TYPE D				TYPE E			
Test No.	*1*	*2*	*3*	*4*	*1*	*2*	*3*	*4*	*1*	*2*	*3*	*4*	*1*	*2*	*3*	*4*	*1*	*2*	*3*	*4*
0% Lime Content																				
Moisture Content (m)	10.0	14.0	17.6	20.8	8.7	9.0	11.0	4.6	10.5	12.7	15.2	19.4	7.6	9.2	10.4	14.7	10.6	7.9	17.6	18.5
Dry Density fd (g/cc)	1.78	1.80	1.72	1.62	2.08	2.12	2.03	1.94	1.51	1.70	1.76	1.67	1.65	1.82	1.96	1.92	1.52	1.75	1.79	1.73
2% Lime Content																				
Moisture Content (m)	8.8	13.2	19.1	22.1	6.8	8.6	12.3	14.2	11.5	13.6	14.2	18.8	10.9	14.2	16.9	17.9	8.9	11.5	14.7	16.9
Dry Density fd (g/cc)	1.66	1.67	1.65	1.62	1.80	1.94	1.99	1.98	1.69	1.72	1.76	1.71	1.63	1.75	1.79	1.75	1.72	1.84	1.88	1.80
6% Lime Content																				
Moisture Content (m)	11.8	15.0	16.8	23.2	7.3	10.8	14.6	16.6	8.3	12.1	14.8	18.8	6.9	13.9	15.4	15.5	8.2	10.5	13.6	22.4
Dry Density fd (g/cc)	1.66	.70	1.70	1.61	1.85	1.95	1.90	1.51	1.60	1.62	1.70	1.68	1.58	1.77	1.79	1.76	1.70	1.81	1.89	1.71
10% Lime Content																				
Moisture Content (m)	19.6	22.1	25.6	26.6	8.0	8.9	11.3	12.5	20.6	21.5	23.4	31.8	13.9	18.1	18.3	20.3	10.7	12.7	21.9	23.2
Dry Density fd (g/cc)	1.52	1.57	1.56	1.54	1.92	1.97	1.99	1.97	1.56	1.61	1.61	1.49	1.76	1.76	1.78	1.72	1.73	1.82	1.69	1.64



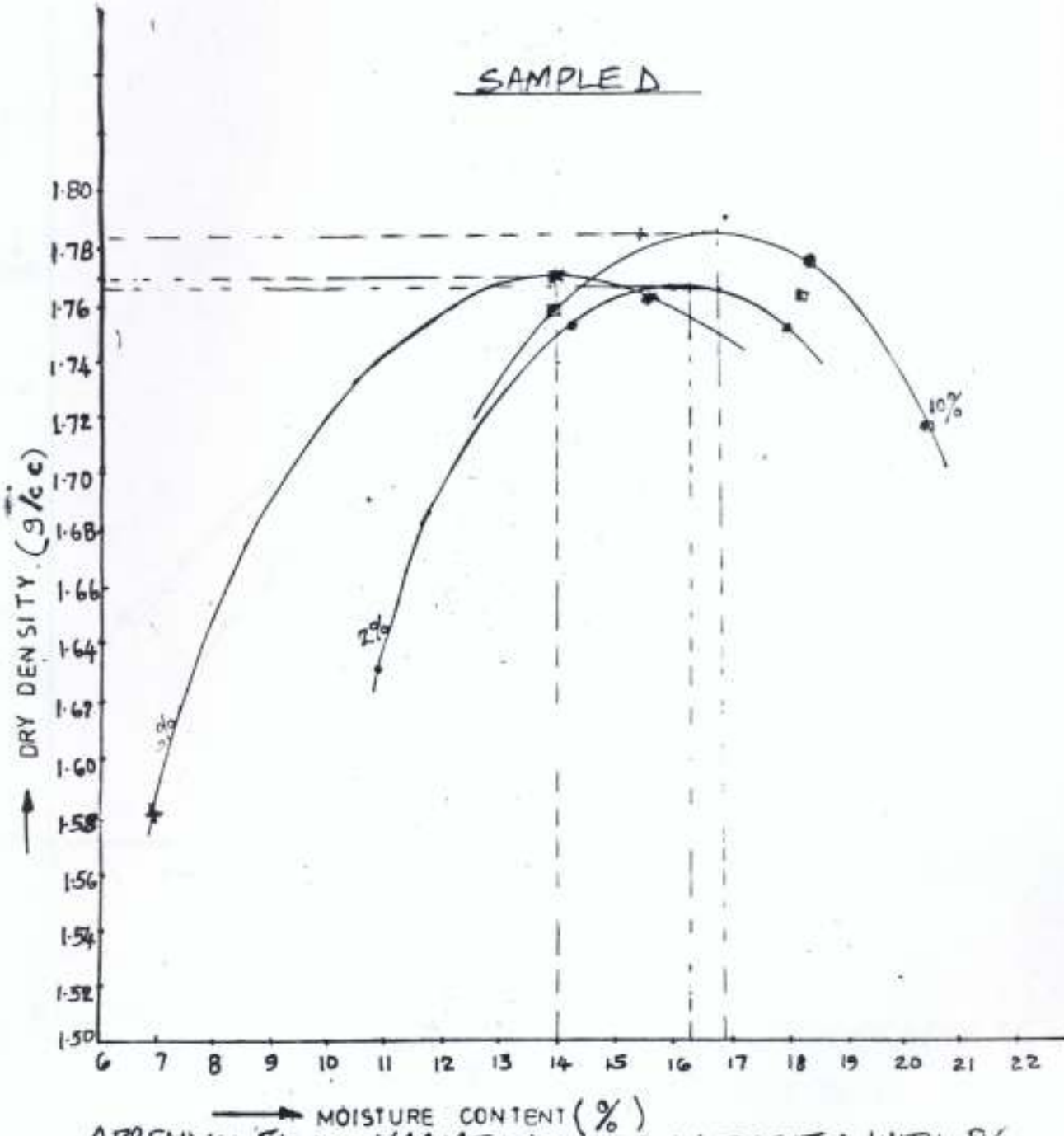
APPENDIX Fig1 VARIATION OF DRY DENSITY WITH PERCENTAG
MOISTURE CONTENT FOR LIME TREATED SO



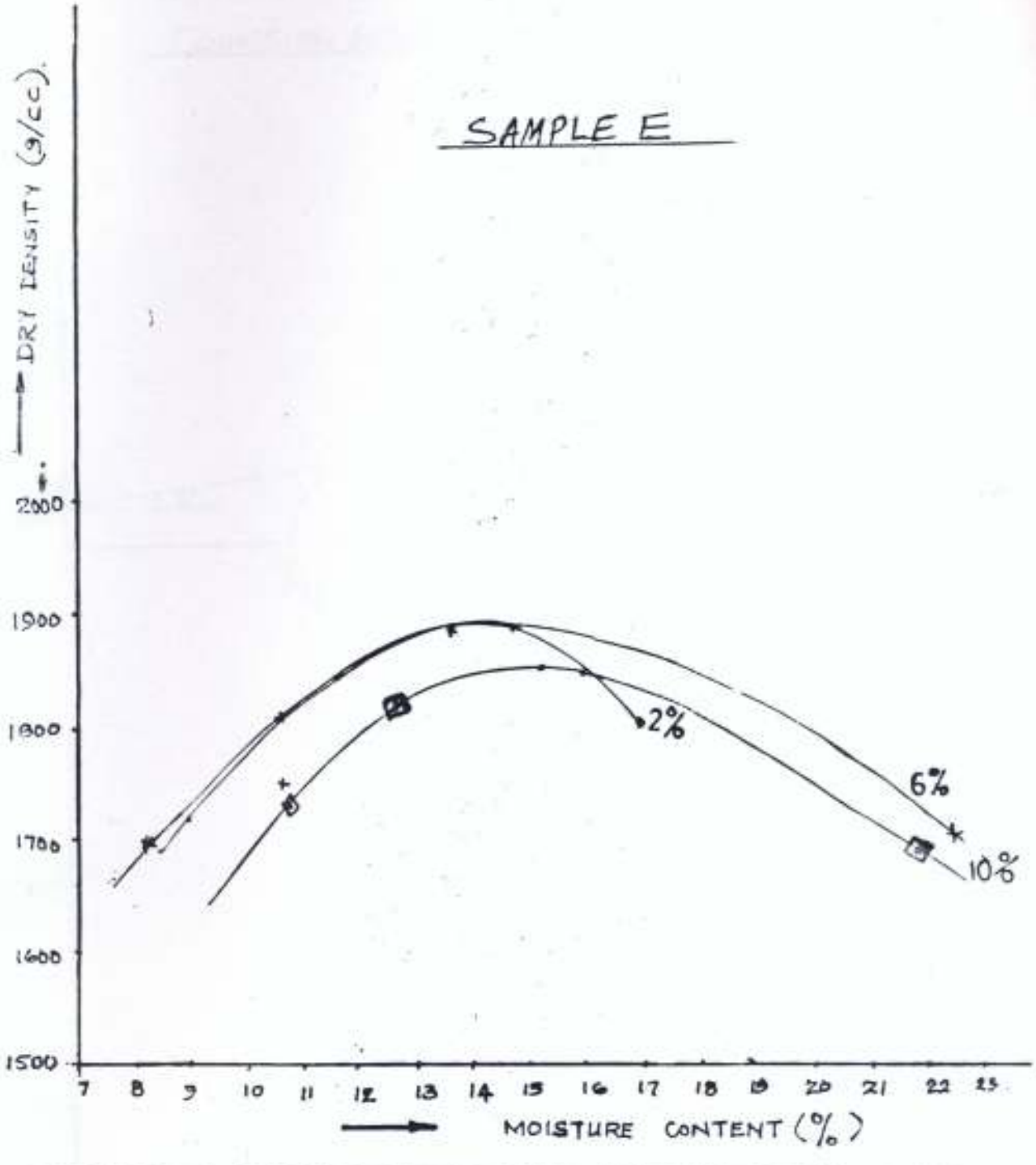
APPENDIX Fig2 VARIATION OF DRY DENSITY WITH PERCENTAGE MOISTURE CONTENT FOR LIME TREATED SOIL



APPENDIX Fig.3 VARIATION OF DRY DENSITY WITH PERCENTAGE MOISTURE CONTENT FOR LIME TREATED SOIL

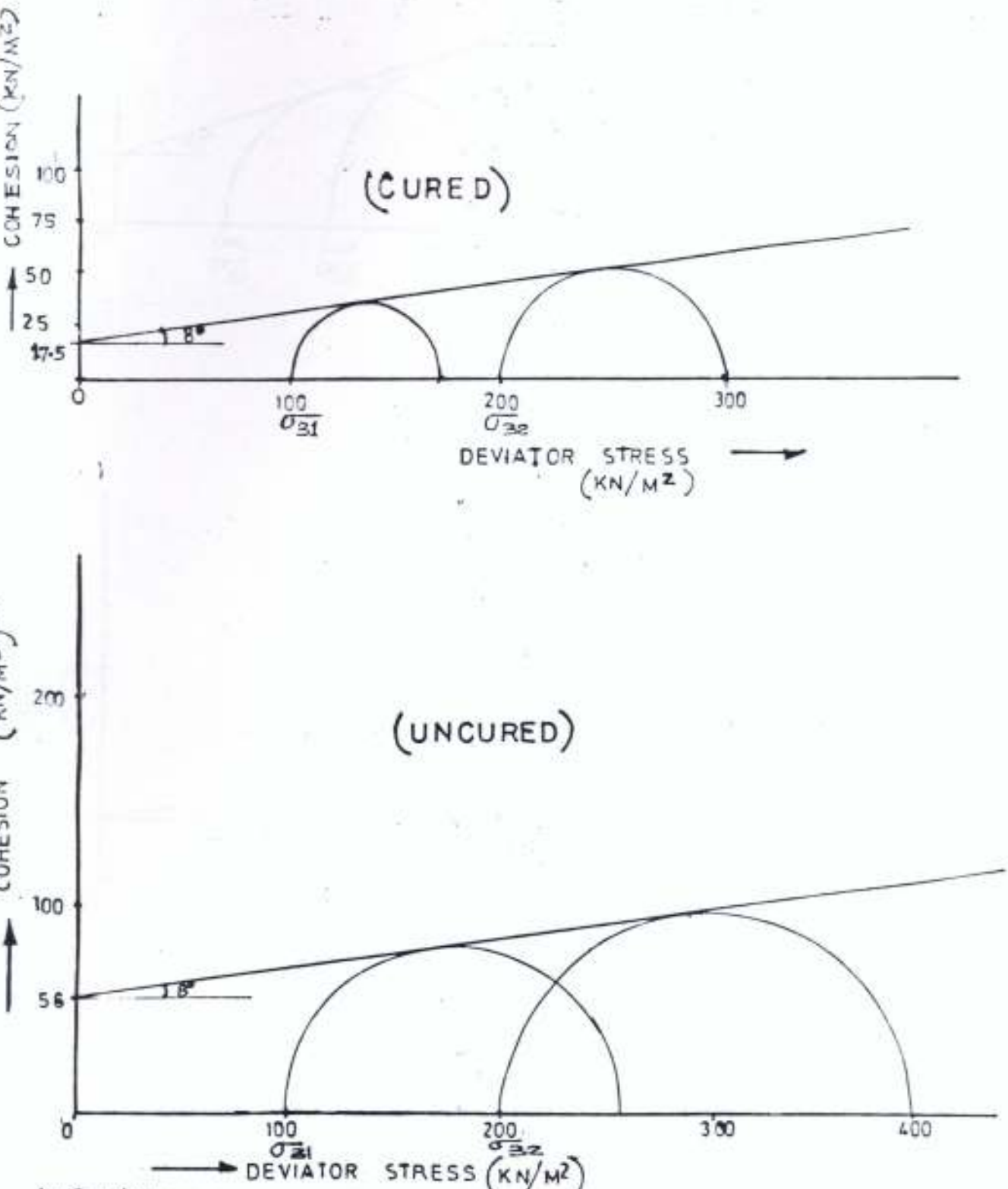


APPENDIX Fig·4 VARIATION OF DRY DENSITY WITH % MOISTURE CONTENT FOR LIME TREATED SOIL

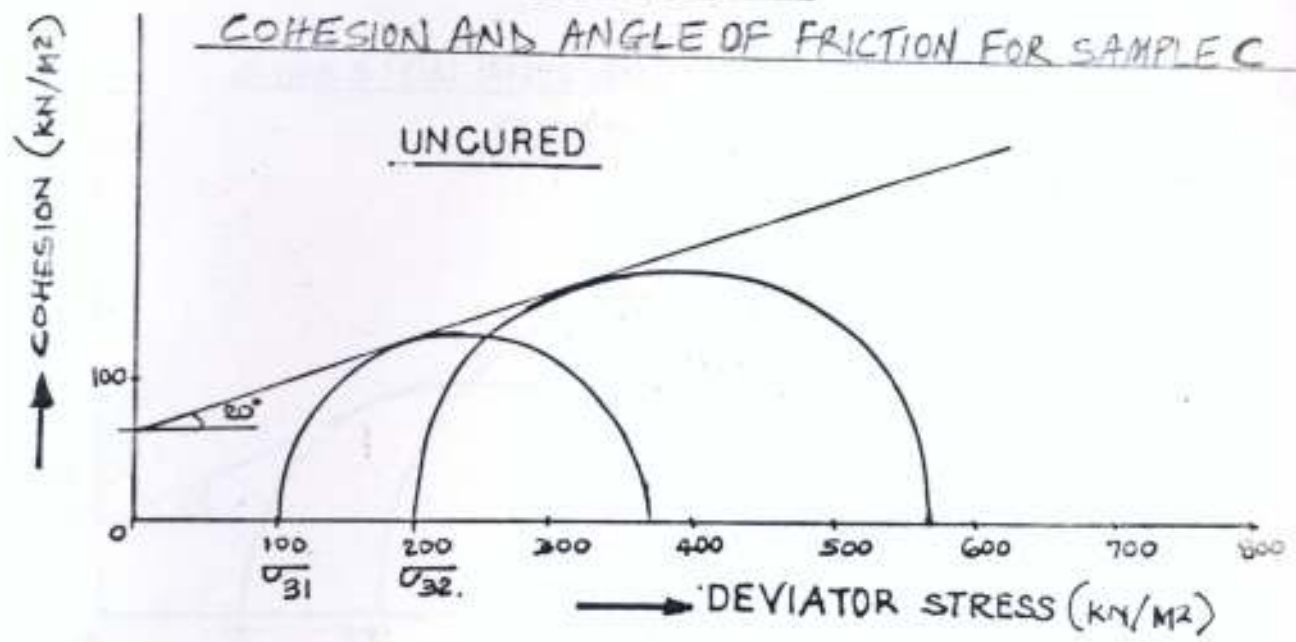


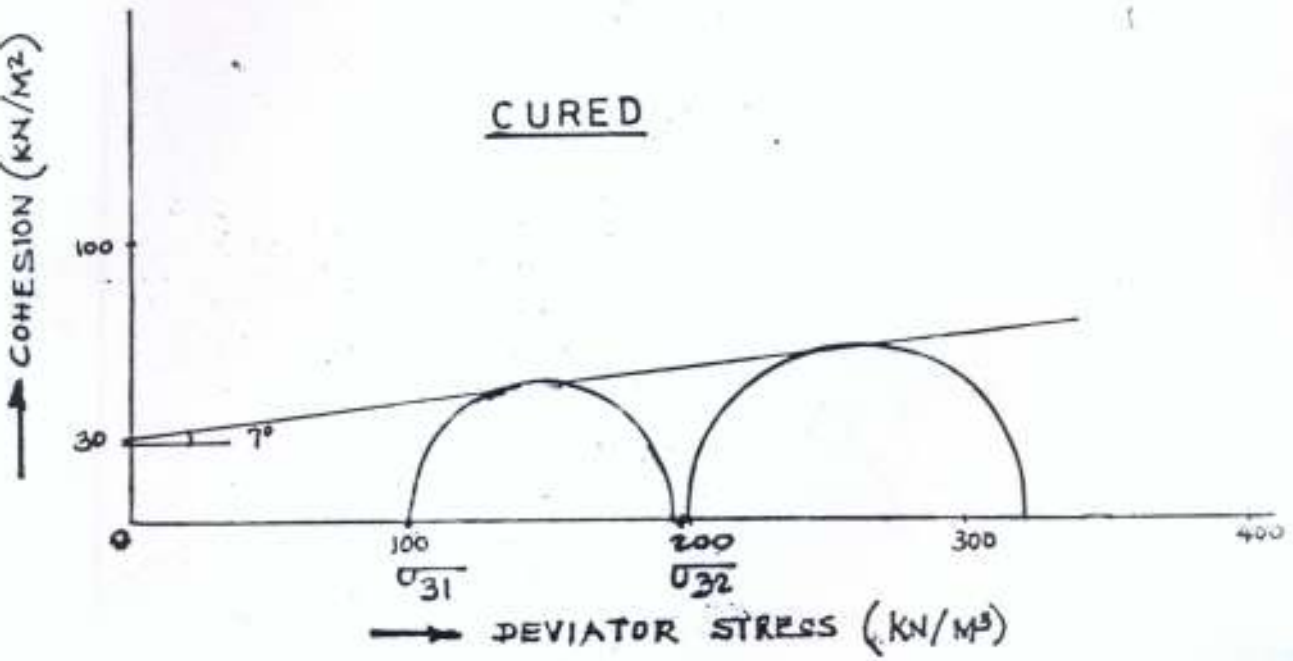
APPENDIX Fig5 VARIATION OF DRY DENSITY WITH % MOISTURE CONTENT FOR LIME TREATED SOIL

COHESION AND ANGLE OF FRICTION FOR SAMPLE A

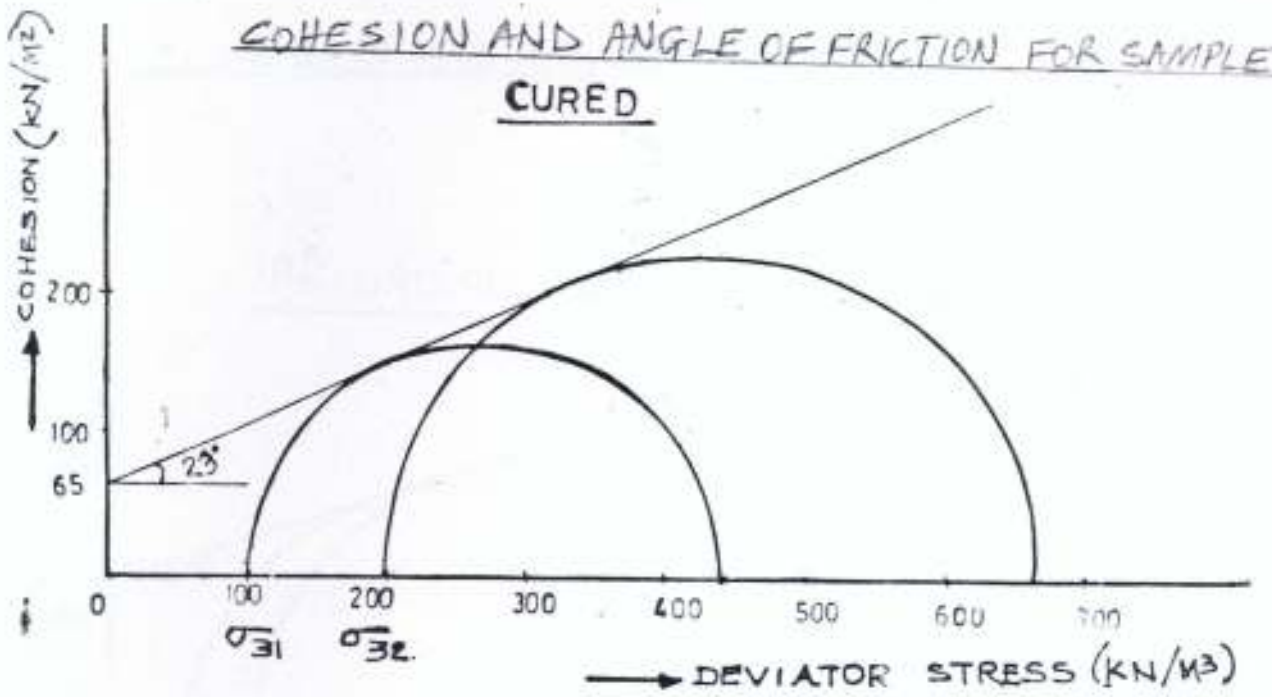


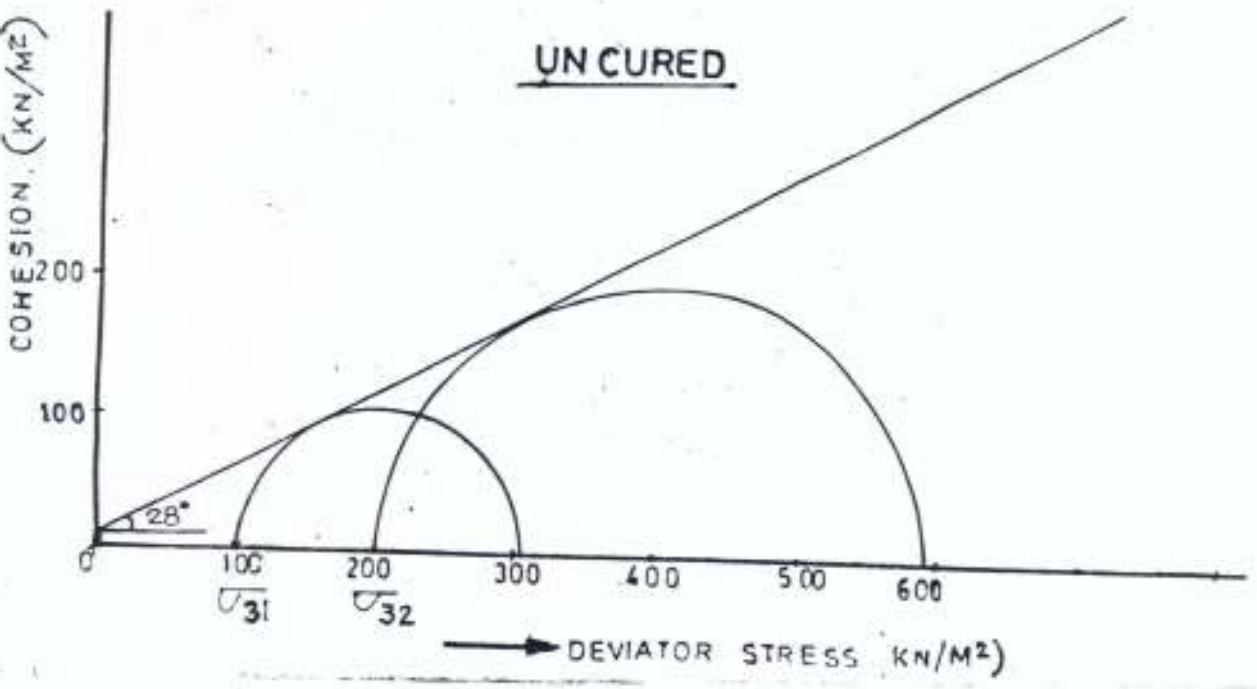
APPENDIX Fig·6 MOHR CIRCLES FOR NATURAL SOIL





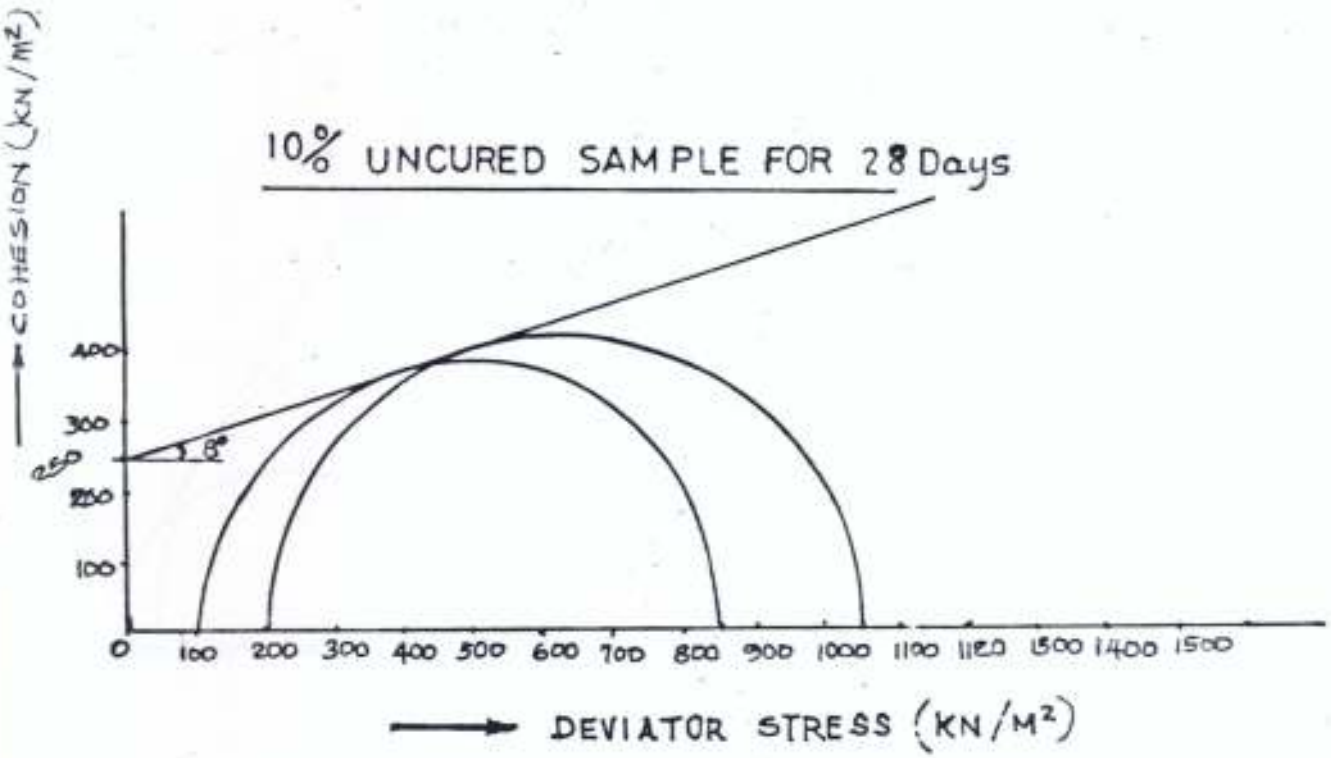
APPENDIX Fig·7 MOHR CIRCLES FOR NATURAL SOIL

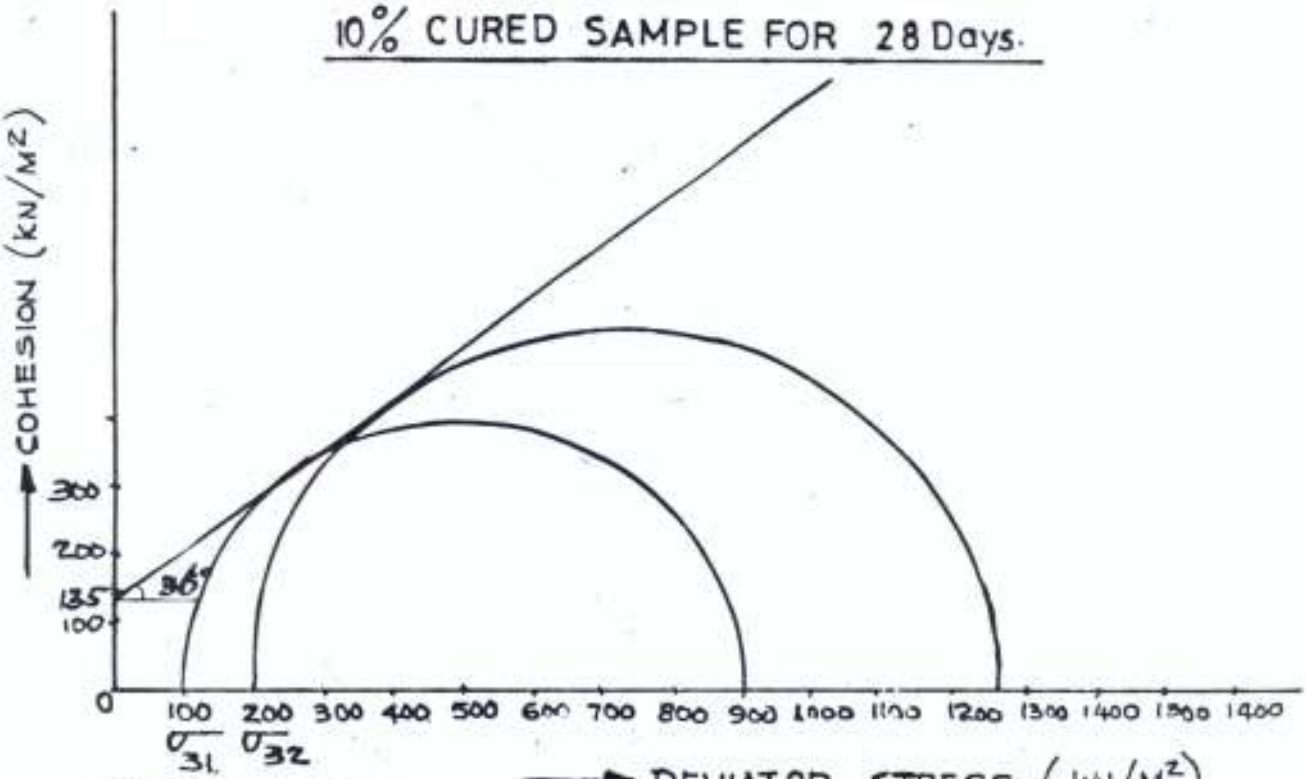




APPENDIX Fig.8 MOHR CIRCLES FOR NATURAL SOIL

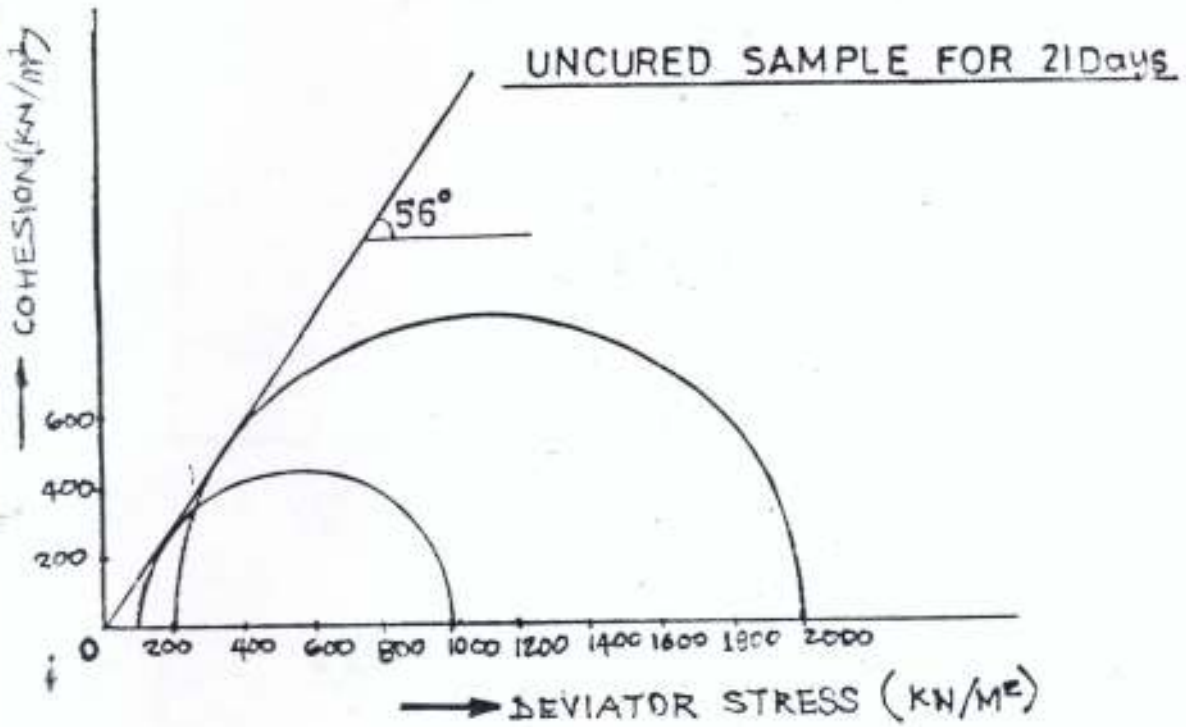
COHESION AND ANGLE OF FRICTION FOR SAMPLE A.

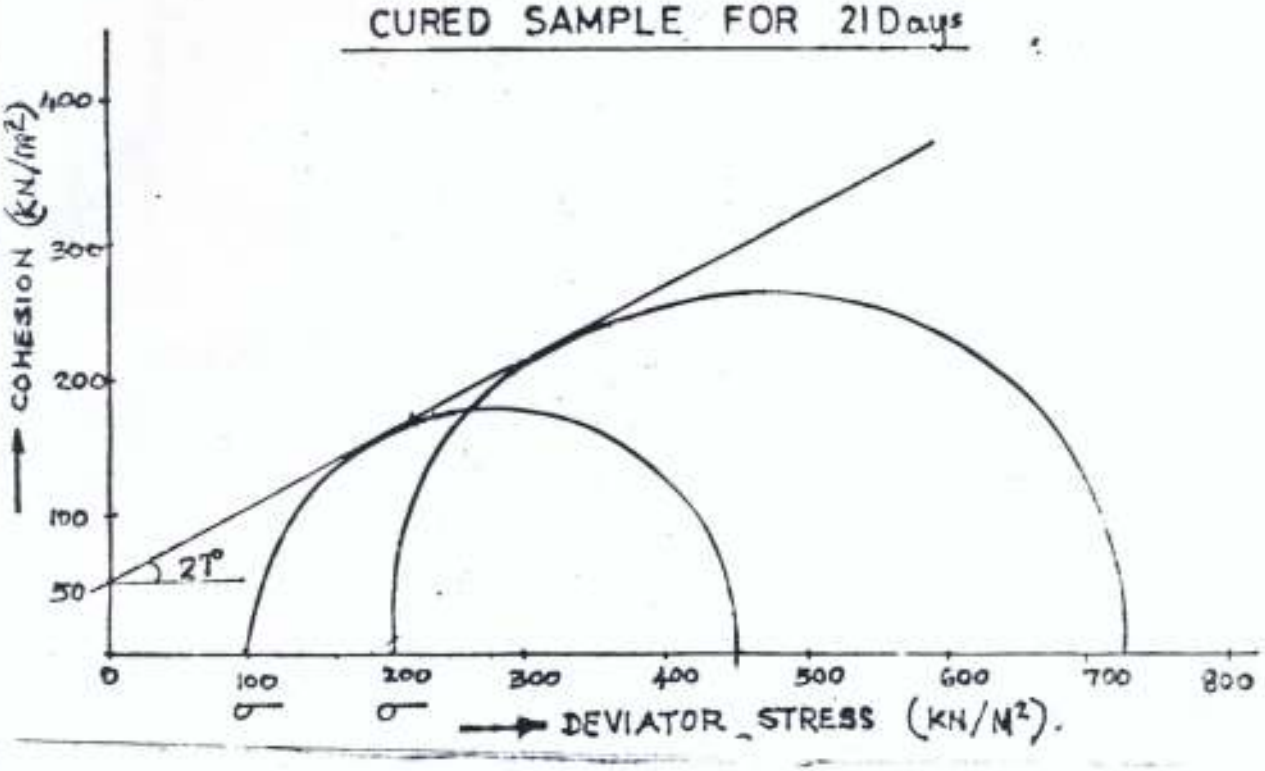




APPENDIX Fig. 9 TYPICAL MOHR CIRCLES FOR LIME TREATED SOIL

COHESION AND ANGLE OF FRICTION FOR SAMPLE. C

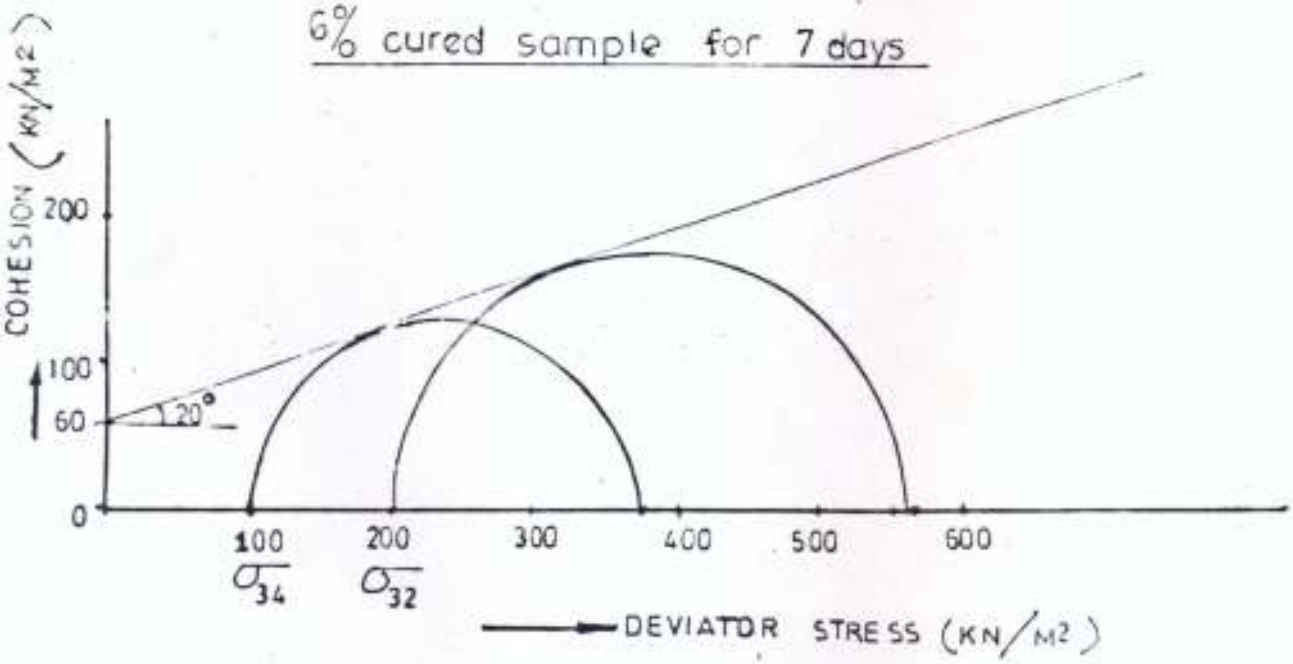




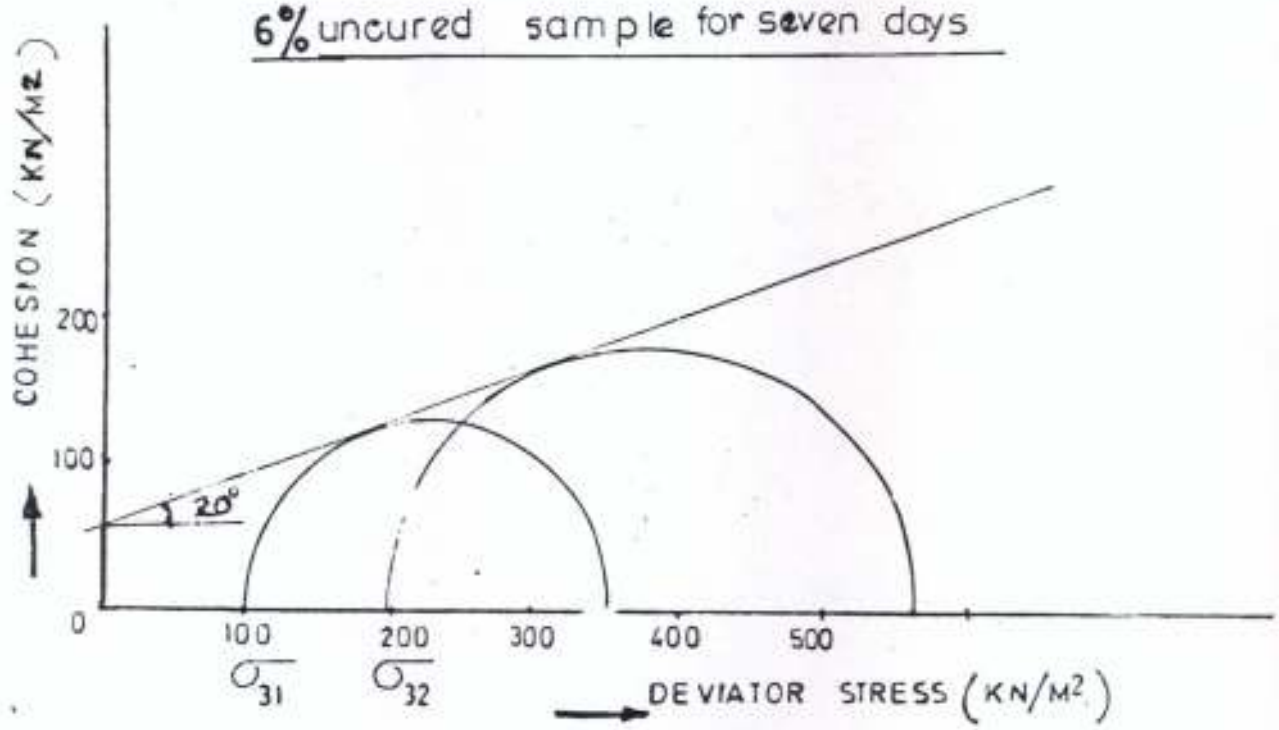
APPENDIX Fig·10 TYPICAL MOHR CIRCLES FOR LIME TREATED SOIL

COHESION AND ANGLE OF FRICTION FOR SAMPLE D

6% cured sample for 7 days



6% uncured sample for seven days



APPENDIX Fig·11 TYPICAL MOHR CIRCLES FOR LIME TREATED SOIL