

## GEOTECHNICAL ENGINEERING PROPERTIES OF CLAY SOIL IN IBADAN, UNIVERSITY OF IBADAN

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### ABSTRACT

This project reports the outcome of an investigation into the effect of lime on the some geotechnical properties of clay soil in Ibadan, to ascertain its suitability for use as a modifier or stabilizer in clay treatment. Three samples were collected from three different locations namely, Odo-ona, Ibadan Grammar School area and Ogunpa all in Ibadan South-West, Ibadan North-East and Ibadan North-West Local

Governments respectfully. Varying percentages of 2,4,8,10 and 12%lime were used in the treatment of clay. Classification, Atterberg, Compaction, and California bearing ratio (CBR) tests, were conducted on the specimens. Results of tests showed that sample 'A' (Odo-ona) was an A-7-6 soil, according to the AASHTO classification system. Liquid limit reduced from 66 % at 0 % lime to 48 % at 12% lime content on sample 'A', 47.5 % at 0 % lime to 32% at 12% lime content on sample 'B', and while sample 'C' drops from 36% at 0% lime to 25% at 12% lime by dry weight of the clay samples, the plastic limit increment and plastic index reduction of the three samples are tabulated on table 4.2. The maximum dry density of sample 'A' reduces from 1.51 Mg/m<sup>3</sup> to 1.38 Mg/m<sup>3</sup>, sample 'B' increases from 2.05 Mg/m<sup>3</sup> to 2.18 Mg/m<sup>3</sup>, while sample 'C' increases from 3.50 Mg/m<sup>3</sup> to 3.59 Mg/m<sup>3</sup> on a range of 0% - 12 % lime. At the optimum lime content, a maximum CBR value of 38 % was achieved for sample 'A', 50% for sample 'B' and 48% for sample 'C'. In conclusion, the improvements noticed in some of the geotechnical properties of sample 'A' were not

adequate for its use in roadwork but samples B and C were adequate, 40% CBR is recommended by Nigeria General Specification for highway and bridges. However, lime can be used as a modifier in the stabilization of sample A with cement or other additives but for samples B and C, lime is a stabilizer, likewise, test could be performed in a wider range across the country to provide data to enhance the relevant equation.

**KEYWORDS:** This project reports the outcome of an investigation.

## CHAPTER ONE

### 1.0 INTRODUCTION

Infrastructural development in areas where problem of soils are identified has been a major source of concern to the engineers. As such, infrastructures like roads, buildings, bridges to mention but a-few within such areas normally undergo foundation problems, which lead to a reduction in the life-span of such facilities. Sound knowledge of geotechnical and engineering properties must be well known in order to commence the design and construction of infrastructures earlier mentioned and the geotechnical parameters needed to be determine includes; the strength, the maximum dry density (MDD), the amount of dominant clays and silts, compaction state or performance of the foundation of the structure concerned.

### 1.1 Geology of Ibadan in South-West Area of Nigeria

Ibadan, city in south-western Nigeria, capital of Oyo State, located about 110 km (about 70 mi) northeast of Lagos. Ibadan lies between the Latitude of  $7^{\circ}$  and  $7^{\circ}30'$  North of the Equator, and Longitude of  $3^{\circ}30'$  and  $4^{\circ}$  East of the Greenwich meridian. Ibadan is underlain by gneisses and migmatite rocks of the undifferentiated basement complex and the older granites. These are crystalline rocks, which form part of the lithology of the crystalline basement complex of Ibadan, Oyo State of Nigeria. The basement is a crystalline (hard) rock basement, which is composed largely of migmatite and granitic gneisses, quartzite and meta-igneous rocks of Pre-Cambrian to Lower Paleozoic. Odo ona and Ogunpa Rivers are some of the major rivers recognized in Ibadan.

### 1.2 Clay and its Associated Engineering Problems

Clay, an abundant geological material accounting for approximately half the strati-graphic column (Kuenen, 1941) is frequently encountered in road cuts and other construction sites where economic and environmental considerations often recommend its use in the

construction of embankments Agbede 1998 and O' Flaherty (1974) clayey material which is very likely to break down in the presence of moisture and frost, it is subjected to swelling during the rainy season and shrinking during the dry season.

Clay as a notoriously unpredictable material, in which a number of failures have been reported by engineers involving settlement and shear failure of compacted clay embankments.

Clays, silts and sands or a mixture of all the three fractions of soil derived from the weathering of rocks. These fractions of soil are deposited in sea or riverbeds in layers and subjected to high overburden pressures, which lead to consolidation. De Graft–Johnson et al (1973).

### **1.3 Objective and Scope of the Study**

The aim of this study is to obtain some geological properties of clay soil and to derive a means whereby it can be utilized in projects construction.

The objectives include the following:-

- (1) Classify and also determine the compaction characteristic of the soil.
- (2) Determine the bearing capacity.
- (3) Determine the physical properties and chemical components of clay soil and also determine the various types of existing clay.
- (4) Proffer a means of modification or stabilization of clay soil.
- (5) Determine the percentage of lime that will be suitable for clay classification
- (6) Determine the percentage of lime suitable for clay stabilization.

Essentially, the work is executed in two stages. The first stage is the field work which entailed the collection of soil samples from about three locations in Ibadan Oyo state.

This stage was thereafter follow by the laboratory works which include tests such as Natural Moisture Content determination, Sieve analysis, Compaction test, Physical and Chemical composition of clay.

### **1.4 Climate, Rainfall and Vegetation**

The study area is situated within the tropical climate environment with prominent wet and dry seasons. The rainfall which is almost confined entirely to the wet season occurs from the month of April to October while the dry season runs from November to March. During this

period, the dust-laden Harmattan wind which blows in from the north-east brings with it the cool winds which moderate the climate at this period. The mean annual rainfall is about 1150mm. All the streams in the area dry up completely while the volume of water in the major existing rivers reduced drastically. At the peak of the rainy season, the rivers and streams of the area overflow their banks. The relative humidity of the project area is generally high (Ola and Raheem 2002). The vegetation cover is tropical rain forest which is characterized by tall grasses and shrubs with many tall trees. The vegetation is generally dense in all the area concerned. The stream and river valley, plain lands and even the mountains contained tall trees.

The average day time temperature is between 35°C and 38°C, but this falls to about 20°C and 22°C at night, most especially during the cold and dry harmattan (dry) season.

### **1.5 Method of Study**

The method of study employed in this research was by extensive desk study, that is, consultation of relevant text books and journals for literature review, and collection of information online.

The last aspect is field and laboratory works. The former is for the collection of samples and the latter for laboratory tests.

## **CHAPTER TWO**

### **2.0 LITERATURE REVIEW**

#### **2.1 Origin and Nature of Soils**

Soil can be said to be as old as the universe itself because its formation has been traced to the outer crust of the earth, which extends 10 to 15 kilometres in depth. Derucher (1999) said that the outer crust is composed of eight predominant chemical elements. These elements are found in the soil and rock of the crust and these elements are assembled in solid homogeneous chemical compounds that form minerals. The minerals in turn form rocks that also produce soil when they are exposed to weathering. The weathering processes were classified into two by Scott (1974) as mechanical and chemical. Soil was defined from an engineering point of view as "any un-consolidated material composed of discrete solid particles with gases and liquids between."

## 2.2 Soil Constituents and their Properties

Soil consists largely of mineral matter formed by the disintegration or decomposition of rocks. Any sample of soil will be found to contain some or all the following: solid phase, liquid phase and gaseous phase (Scott, 1974).

### 2.2.1 Solid Phase

The solid phase consists of four major components, namely: the primary rock minerals, clay minerals, inter-granular cement and organic matter.

The primary rock mineral are pieces broken from the parent rock. They are generally relatively large, being seldom less than 0.002mm in diameter, although some soils of glacial origin contain very small particles of rock floes. The particles are generally rounded or angular in shape. Where particles of this type form the principal part of the minerals, as in gravels and sands, the engineering properties depend mainly on the grading and on the closeness of packing.

A densely packed material has a low compressibility and high resistance to shearing on account of the interlocking of the particles. The shape and texture of the particles also have some effect on the properties.

Clay minerals present in a soil sample also affect the engineering properties of such soil. They are mainly the products of chemical weathering. The particles are very small; their dimension is around 0.002mm.

### 2.2.2 Liquid Phase

The liquid phase consists of water and dissolved salts. Change in water content of the soil is the cause of variation in the engineering properties. Shear strength, compressibility and permeability are all, directly or indirectly related to the water content. The dissolved salts are most times detrimental to engineering properties of soils. Most important of these are the sulphate, such as, calcium sulphate, which is moderately soluble but sodium and magnesium sulphates are less common, but are more dangerous because of their greater solubility.

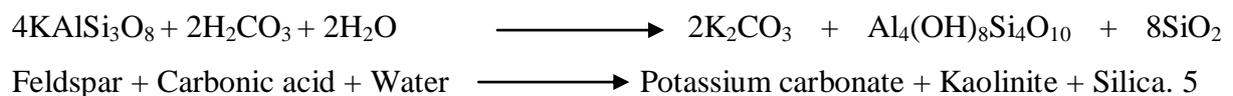
### 2.2.3 Gaseous Phase

It comprises the air and water vapour. Since most soils are not fully saturated, the pores are filled by air, which may cause excessive settlement of such soil when compacted. This therefore, is an essential property that alters the behaviour of soil when in service. Likewise,

if the soil is partially saturated, air will fill the pore spaces thereby increasing the relative humidity of such soil. If the degree of saturation is low, so that the air spaces are mainly continuous, there may be considerable migration of water in form of water vapour.

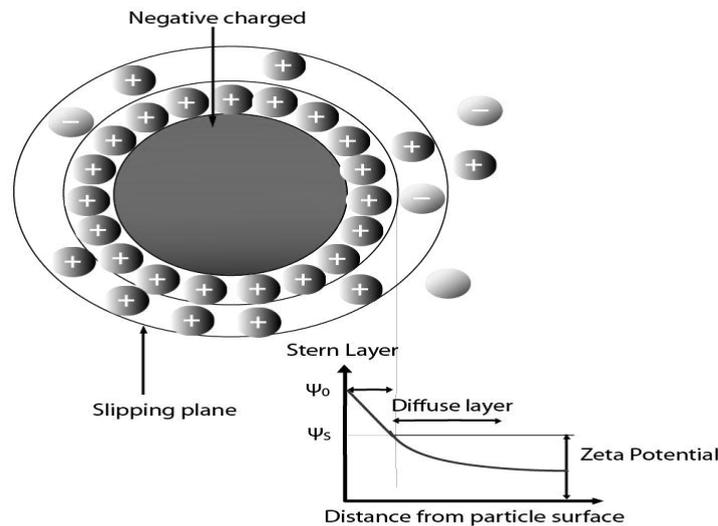
### 2.3 Clay Formation

Clay minerals are layer silicates that are formed usually as products of chemical weathering of other silicate minerals at the earth's surface; their new materials formed are quite different from the parent rock. Unlike silt, sand and gravel that retain much of its chemical structure of their parent rock. A chemical weathering process changes orthoclase feldspar (mineral in granite and many other rocks) into a mineral called Kaolinite. During raining period, water and carbon di-oxide in atmosphere seep through soil, forming a weak carbonic acid ( $H_2CO_3$ ) solution. This acid reacts with feldspar according to the equation bellow;

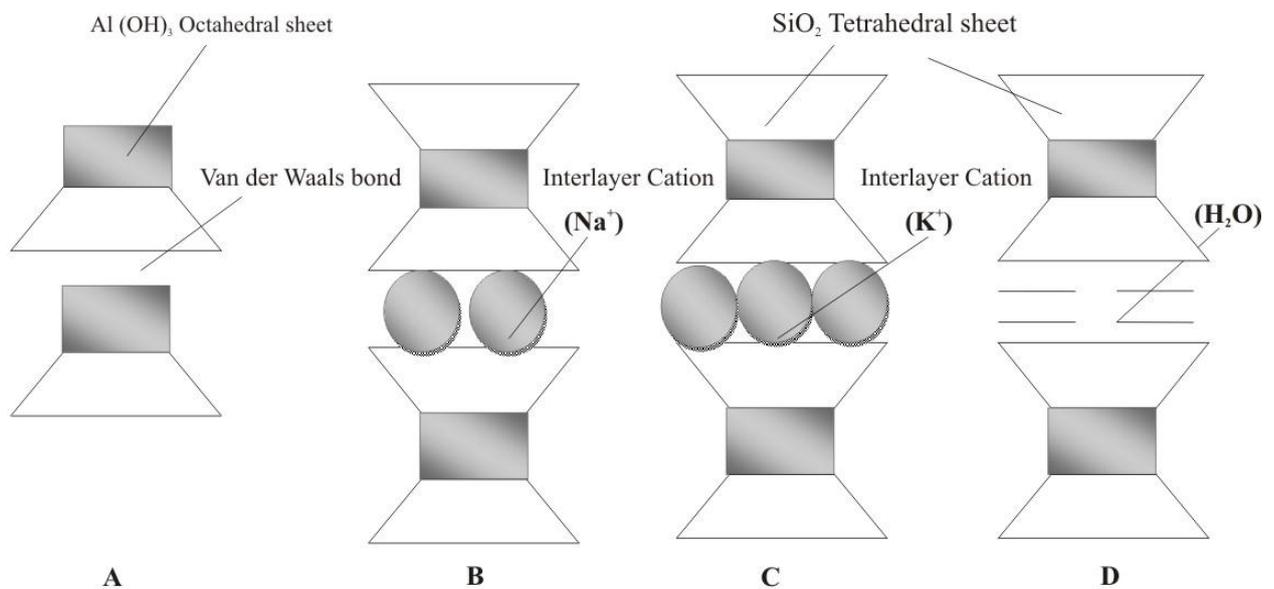


They are found most often in shale, the most common type of sedimentary rock. Clay minerals act as "chemical sponges" which hold water and dissolved plant nutrients weathered from other minerals. This results from the presence of unbalanced electrical charges on the surface of clay grains, such that some surfaces are positively charged (and thus attract negatively charged ions), while other surfaces are negatively charged (attract positively charged ions) as in figure 2.1.

Clay minerals also have the ability to attract water molecules, because this attraction is a surface phenomenon, it is called adsorption. Water is drawn from the surface of the grains, both because their molecules tend to saturate the negative charges and because it interacts with the oxygen atoms. For better understanding of the behaviour of clay minerals, it is necessary to examine their crystal structure in detail (Scott, 1974), see figure 2.2.



**Fig. 2.1:** *Schema of a clay particle with adsorbed water layers and diffuse ions.*



**Fig. 2.2:** *Schematic representation of the clay minerals. (A) Kaolinite, (B) Na-smectite (C) Illite and (D) Montmorillonite*

## 2.4 Physical and Chemical Properties of Clay

The properties of illite and montmorillonite (also known as smectite or bentonite) can be used to represent clay due to their swelling and shrinkage characteristics which are very challenging to engineers and technologists in the area of construction. The physical and chemical properties are shown in table 2.1 and 2.2 respectively. Sivapullaiah and Lakshmikanthay (2005).

**Table 2.1: Physical properties of bentonite and illite.**

Property	Bentonite	Illite
Specific gravity	2.77	2.45
Liquid limit: %	310	134
Plastic limit: %	49	79
Plasticity index: %	261	55
Shrinkage limit: %	10	45
Base exchange capacity; meq/100 g	100	16
Specific surface: m <sup>2</sup> /g	800	90
Proctor's		
Maximum dry density: kN/m <sup>3</sup>	11.9	9.2
Optimum water content: %	42.5	60
Unconfined compressive strength: kPa	220	170

**Table 2.2: Chemical composition of bentonite and illite.**

Property: %	Bentonite	Illite
Silica, as SiO <sub>2</sub>	43.6	32.0
Alumina, as Al <sub>2</sub> O <sub>3</sub>	18.6	15.0
Ferric oxide, as Fe <sub>2</sub> O <sub>3</sub>	13.1	14.0
Calcium, as CaO	2.7	1.2
Magnesium, as MgO	3.3	9.6
Titanium, as TiO <sub>2</sub>	2.6	1.5
Potassium, as K <sub>2</sub> O	0.3	18.0
Sodium, as Na <sub>2</sub> O	2.6	1.1
Loss on ignition	13.3	7.5

## 2.5 Clay Minerals and their Economic Importance

### 2.5.1 Kaolinite

This clay mineral is the weathering product of feldspars. It has a white, powdery appearance. Kaolinite is named after a locality in China called Kaolin, which invented porcelain (known as china) using the local clay mineral. The ceramics industry uses it extensively. Because kaolinite is electrically balanced, its ability of adsorb ions is less than that of other clay minerals. Still, kaolinite is used to make pottery, an important ingredient in paper; paint was used as the main ingredient for the original formulation of the diarrhea remedy (that is Kaopectate).

### 2.5.2 Illite

It is the weathering product of feldspars and felsic silicates. It is named after the state of Illinois, and is the dominant clay mineral in mid-western soils. It is more suitable as a liner material for waste disposal facilities when mix with bentonite.

### 2.5.3 Smectite

This clay mineral is the weathering product of mafic silicates, and is stable in arid, semi-arid, or temperate climates. It was formerly known as montmorillonite. Smectite has the ability to adsorb large amounts of water, forming a water-tight barrier (preventing groundwater contamination). It is used extensively in the oil drilling industry, civil and environmental engineering (where it is known as bentonite), and the chemical industry. Here are two main varieties of smectite; (1) Sodium Smectite, (2) Calcium smectite.

### 2.5.4 Vermiculite

This clay mineral has the ability to adsorb water, but not repeatedly. It is used as a soil additive for retaining moisture in potted plants, and as a protective material for shipping packages.

### 2.5.5 Attapulgite

This mineral actually resembles the amphiboles more than it does clay minerals, but has a special property that smectite lacks - as a drilling fluid, it is stable in salt water environments. When drilling for offshore oil, conventional drilling mud falls apart in the presence of salt water. Attapulgite is used as a drilling mud in these instances. Incidentally, attapulgite is the active ingredient in the current formula of Kaopectate.

### 2.5.6 Chlorite

This clay mineral is the weathering product of mafic silicates and is stable in cool, dry, or temperate climates. It occurs along with illite in mid-western soils. It is also found in some metamorphic rocks, such as chlorite schist.

## 2.6 Soil Investigation Techniques

Soil investigation is an essential preliminary to the execution of a civil engineering project. Sufficient information must be obtained to enable a safe and economic design to be made and to avoid any difficulty during construction. It was reported by Craig (1992) that principal objectives of soil investigation are three, namely;

1. To determine the sequence, thickness and lateral extent of the soil strata
2. To obtain representative samples of the soils for identification and classification and where necessary, for use in laboratory tests to determine relevant soil parameters.
3. To identify the groundwater condition.

Sowers (1979) identified three major procedures for complete investigations. They are reconnaissance, exploratory and intensive investigation. In accomplishing the above, Craige (1992) gave about eight techniques for exploratory and intensive investigations. They are Trial pits, Shaft and headings, Percussion boring, Mechanical augers, Hand and portable augers, Wash boring and rotary drilling. For the purpose of this research, the trial pits method will be explained further.

### **2.6.1 Trial Pits**

The excavation of trial pit is a simple and reliable method of investigation but exhibited to a maximum depth of 4.5m. The soil is generally removed by means of the back shovel of a mechanical excavator. The sides may be supported if it is not stepped or at a safe shape. In trial pitting, large samples can be taken for laboratory testing, or test may be carried out on site. The major setback under this technique is the need for timbering which makes the work increasingly slow and expensive as the depth increases and therefore, trial pits are rarely used for deep investigation. In the course of this project, trial pit ranges between 1m to 2m was dug before attaining clay material needed as shown in fig. 3.1 (a) and fig. 3.1(b).

## **2.7 Soil Tests**

After the collection of soil samples from trial pits or boreholes depending on which is applicable, tests are carried out on the samples in the laboratory. However, some test carried out on the field depending on the nature of the work or the property of interest. Various types of laboratory tests include:

### **2.7.1 Grain Size Distribution**

The size distribution of a soil sample is determined using the processes of sieve analysis and sedimentation. The former is used for coarse-grained soil while the later is used for fine-grained soil.

According to BS1377(1975), sieve analysis process involves sieving a soil sample through a set of sieves kept one over the other, the largest size being kept at the top and the smallest size at the bottom. The soil retained on each sieve is weighed and expressed as a percentage of the total weight of sample. The percentage passing each sieve can be plotted against the sieve size. This will give an S or Ogive curve.

Sedimentation process is adopted for soils passing through 75micron sieve and is based on the principle that the larger grains settle faster. Hydrometer or pipette methods are the methods use in common.

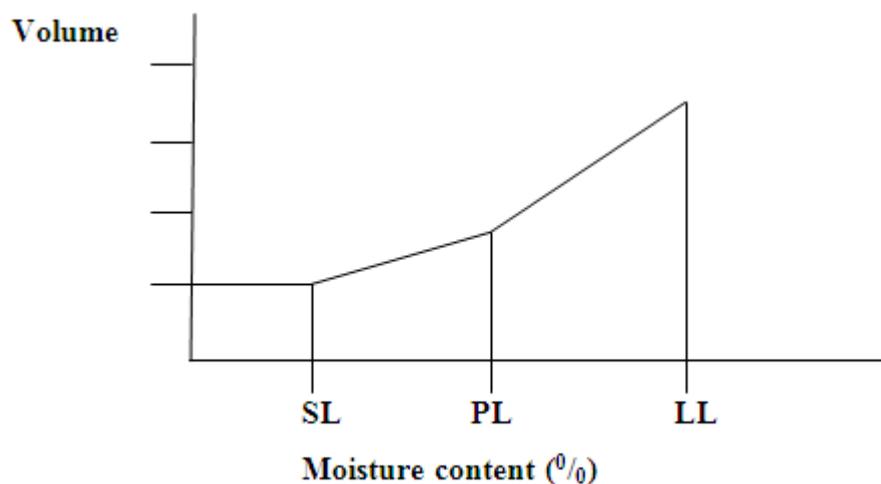
### 2.7.2 Atterberg Limit Test

A Swedish soil scientist called Atterberg in 1911 developed this test. It was evolved because the grain size distribution discloses very little about the engineering properties of very fine grained soil in which the clay minerals predominate. A better indication of their properties is obtained, measuring the water contents at which certain changes in the physical behaviour can be observed (Derucher, 1988). In order to determine the consistency and plastic behaviour of the fine soil, he established definite by arbitrary limits for each of the limits: liquid, plastic and shrinkage limits as shown in Figure 2.3 (Sowers, 1979, Gurcharm and Jagdish, 1983). Figure 2.4 shows the relationship between the change in moisture content and volume change.

#### Atterberg Limits

Stage	Description	Boundary
Liquid state	Deforms easily; consistency of pea, soup to soft butter	Liquid limit
Plastic state	Deforms without cracking; consistency of soft butter to stiff puffy.	
Semi-solid state	Deforms permanently but cracks; consistency of cheese.	Plastic limit
Solid state	Breaks before it will deform; consistency of hard candy.	Shrinkage limit

**Figure 2.3: Consistency of un-grained soil at different moisture contents (Sower, 1979).**



**Figure 2.4:** *Moisture content versus volume relation.*

### 2.7.3 Compaction Test

The aim of the laboratory compaction test is to determine the maximum dry density (**MDD**) and corresponding optimum moisture content (**OMC**) that a soil sample must achieve for maximum performance under load. The detail procedure of the test is in BS1377 (1975).

### 2.7.4 California Bearing Ration test (CBR)

The California state highways department for the evaluation of sub-grade strengths developed the CBR test, which is an adhoc penetration test. It is a measure of the shearing resistance of a soil to penetration under controlled density and moisture conditions. The strength of a soil is found by causing a plunger (or piston) of standard size to penetrate a specimen of soil prepared to the density and moisture conditions of the soil to be tested in standard mould. The resistance to penetration is measured and the expressed as a percentage of the known resistance to penetration of the plunger in a crushed aggregate (Khanna, 1977). The detail procedures of the test are in BS1377.

## 2.8 Engineering Soil Classification System

Soil classification system is an arrangement of different soil into groups having similar properties. For engineering purposes, soils can be classified according to the following systems.

### 2.8.1 Grain Size Classification System

This system uses the result of sieve analysis test only. It classifies soils as gravel, sand, silt and clay. The limits of grain size for each component are given in Table 2.3.

Source: Gurcharm and Jagdish (1983).

**Table 2.3: Grain Size Distribution of Soils.**

	Type of Soil	Diameter
1.	Gravel	2.0mm and above
2	Sand	2.0mm to 0.6mm coarse sand 0.6mm to 0.2mm medium sand 0.2mm to 0.06mm fine sand
3	Silt	0.06mm to 0.02 coarse silt 0.02mm to 0.006 medium silt 0.006mm to 0.002mm fine silt
4.	Clay	0.002mm to 0.0006mm coarse clay 0.0006mm to 0.0002mm medium clay 0.0002mm and below fine/colloidal clay

### 2.8.2 Textural Classification System

The classification is based on grain size distribution but it employs triangular chart for better classification when compared with grain size classification system. It was developed by the Agricultural engineers. The equilateral triangular chart has boundary of sand, silt and sand.

### 2.8.3 United Soil Classification System

This system is an outgrowth of the Airfield classification system developed by Atterberg Casagrande in 1948 as a rapid method for identifying and grouping soils for military construction. The Unified system uses letters instead of numbers to designate the various groups. It uses grain size distribution, liquid and plastic limits as classification tools, which is advancement over the grain size distribution and textural classification systems.

For coarse grained soils (more than 50% retained on No 200sieves)

- (i) G symbolizes gravelly soils.
- (ii) S symbolizes sandy soils.

For subdivision of sand or gravels are:

- (a) W symbolizes well-graded, fairly clean material
- (b) C symbolizes coarse materials with clay binder
- (c) P symbolizes poorly graded.
- (d) M symbolizes coarse materials containing silts or rock flour.

For fine-grained soils (more than 50% passing No. 200 sieve)

- i) M symbolizes inorganic, silt and very fine sand
- ii) C symbolizes inorganic, clays

iii) O symbolizes organic, clays and silts.

#### 2.8.4 AASHTO Classification System

It is one of the oldest systems of grouping soils for engineering purpose. It was first introduced in 1920 and since then, it has undergone many revisions and modifications and it widely used for evaluating soils for highway sub-grade and embankment construction. It uses grain size distribution, Atterberg limits and group index which make it more superior to the earlier systems. This system divides all soils into three categories: granular, silt, clay and organic soil. The symbols A-1 to A-8 are given to the classes which loosely indicate a decreasing quality for highway construction with increasing number. Some of the classes are subdivided to indicate differences in plasticity as shown in Table 2.4 overleaf (Sowers, 1979). An important feature of the AASHTO classification is the group index, which is based on the service performance of soils. The group index is defined by the empirical equation

$$GI = 0.2a + 0.005ac + 0.01bd$$

Where:

a = the portion of percentage passing No. 200 sieve greater than 35% but less than 75%

b = the portion of percentage passing No.200 sieve greater than 15% but less than 55%

c = the portion of the numerical liquid limit greater than 40% but less than 60

d = the portion of the numerical plasticity index greater than 10% but less than 30

(Oglesby and Hicks, 1982 and Sowers, 1979).

T ABLE 2.4 AASHTO DESIGNATIONS

Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5 A-7-6
Sieve analysis: %passing											
No: 10 (2.00mm)	50max	50max	51min								
No: 40 (0.425m)	30max	25max	10max	35max	35 max	35max	35 max	36min	36 min	36 min	36min
No: 200 (0.075mm)	15max										
Characteristics of fraction											
passing No: 40											
Liquid limit											
Plastic index	6max	6 max									
Group index	0		NP	40max	41min	40max	41min	40max	41min	40max	41min
Usual types of	stone fragment		0	10max	10min	11min	11 min	10max	10 max	11min	11max
significant	gravel and sand		fine sand	0				8max	12max	16max	20max
Constituent materials								silty soil			
General rating as sub-grade			Excellent to good								

## 2.9 Soil Stabilization / Modification

Stabilization of soil can be defined as any method used to improve soil engineering and physical properties such as increasing its strength and reducing or eliminating its expansion.

According to Johnson et al (1988) stabilization infers improvement in both strength and durability. Osinubi and Katte (1991) referred to soil stabilization as the alteration or control of any soil property.

In soil modification, strength may or may not increase and the procedure is carried out to make the soil in question workable, through the modification of soil properties. Modifiers that are often used include cement, lime and bitumen, of which lime is the most commonly used. When lime is used as a modifier, it makes the soil workable for subsequent stabilization with cement or other agents.

### 2.9.1 Classification and Type of Soil Stabilization

Classifications are based on the method of treatment being used, which can be categorized into two major groups; mechanical and chemical stabilization.

- ❖ Mechanical stabilization: this sometimes term granular stabilization, soil aggregate mixture is the only factor. In this process, aggregate and soil are properly proportioned and the resulting mixture is adequately compacted to get a stable layer.
- ❖ Chemical stabilization: this method involves the use of chemical for the process of stabilization. The chemical to be used depend on the property of soil to be improved. The use of lime, cement and bitumen with other additives are good example of chemical stabilization.

The use of lime as a modifier /stabilizer of soil with low strength, Ingles and Metcalf (1972) as well as Kedzi (1979) recommended 2 - 3 % of hydrated lime for soil modification. Whereas Yoder and Witczak (1975) recommended ½ - 4 % lime for the same process.

The optimum amount of lime for maximum strength gain in stabilizing soil with lime according to Eades and Grim (1960) is 4 - 6 % for Kaolinite, about 8 % for illite and montmorillonite.

### 2.9.2 Mechanism of Lime Stabilization

According to Chen (1975) and Brown (1996), addition of lime to clay soil provides abundance of calcium ions ( $\text{Ca}^{2+}$ ) and magnesium ions ( $\text{Mg}^{2+}$ ). These ions tend to displace

other common cations such as sodium ( $\text{Na}^+$ ) or potassium ( $\text{K}^+$ ), in a process known as cation exchange. Replacement of sodium or potassium ions with calcium significantly reduces the plasticity index of the clay. A reduction of plasticity is usually accompanied by reduced potential for swelling. The addition of the lime increases the soil pH, which also increases the cation exchange capacity.

## CHAPTER THREE

### 3.0 METHODOLOGY

#### 3.1 Collection of Samples

A reconnaissance survey of the studied and selected areas were conducted in the month of March and April, 2010 of which soil samples were collected in air tight bags, describe visually and labelled for laboratory testing. The sites were finally chosen after sighted traces of clay on them, they are namely;

- i. Odo-ona in Ibadan South-West LGA., Oluyole.
- ii. Ibadan grammar, closed to Molete in Ibadan South-East LGA, Mapo.
- iii. Ogunpa market in Ibadan North-West LGA, Onireke.

Trial pits of depth ranging from 1m – 2m were dug at the various locations to get fairly distributed clay samples mean while the collection of samples and their handling were done in accordance with the Civil Engineering code of practice CP 2001 ‘Site Investigation’. Thereafter the samples were later conveyed to the laboratory and subjected to the following tests listed below.

- i. Sieve / Hydrometer Analysis
- ii. Atterberg limit test with 0% - 12% lime
- iii. Compaction test ,, ,, ,,
- iv. California bearing ratio test. ,, ,,

Some samples and locations are shown in figure 3.1



**Fig. 3.1: Samples collected in Ibadan.**

## **3.2 Laboratory Test Procedures**

### **3.2.1 Sieve / Hydrometer Analysis**

#### **3.2.1.1 Sieve Analysis**

The sieve analysis test was executed on oven dried clay sample specimen of weight between 300-600gm. The whole specimen was placed in a set of sieves and sieved by manual shaking of the set by a short period. The set of sieve size was between 3/8 to 0.075 $\mu$ m (No. 200). The weight of clay material retained on each sieve was found and the relationship between the 'percentage passing' and the corresponding sieve sizes were plotted and analysed for each of the samples tested. The sieve analysis plotted in the Appendix A1-A3.

### 3.2.1.2 Hydrometer Analysis

Since the determination of grain size distribution by sieve analysis is limited to those materials larger than the No. 200 (0.045-mm) sieve, meanwhile silt and clay happen to have grain size smaller to sieve No 200. However, hydrometer is used to classify fine-grain soil. A soil is considered frost susceptible if it contains 3<sup>0</sup>/<sub>0</sub> (percent) or more by weight of particles smaller than 0.020 mm in diameter. Frost susceptibility should always be considered in areas subject to substantially freezing temperatures, since repeated freezing, and subsequent thawing, of water in the soil can seriously affect the ability of the soil to support a structure. Hydrometer analysis is the test used to determine the grain size distribution of the soils passing the No. 200 sieve. Hydrometer analysis is based on Stokes' law, which relates the terminal velocity of a free-falling sphere in a liquid to its diameter. The relation is expressed by the following equation.

$$V = \frac{(G_s - G_w) D_2^2}{18n}$$

where

V= Terminal Velocity

G<sub>s</sub>= specific gravity of solid

G<sub>w</sub>= specific gravity of liquid which the sphere is falling

D<sub>2</sub>= diameter of the sphere

n= viscosity of the liquid.

It is assumed that Stokes' law can be applied to a mass of dispersed soil particles of various shapes and sizes. Larger particles settle more rapidly than the smaller ones. The hydrometer analysis is an application of Stokes' law that permits the calculation of the grain size distribution in silts and clays, where the soil particles are given the sizes of equivalent spherical particles. The density of a soil-water suspension depends upon the concentration and specific gravity of the soil particles. If the suspension is allowed to stand, the particles will gradually settle out of the suspension, and the density will be decreased. The hydrometer is the instrument used to measure the density of the suspension at a known depth below the surface. The density measurement, together with knowledge of specific gravity of the soil particles, determines the percentage of dispersed soil particles in suspension at the time and depth of measurement. Stokes' law is used to calculate the maximum equivalent particle diameter for the material in suspension at this depth and for the elapsed time of settlement. A series of density measurements at known depth of suspension

and at known times of settlement gives the percentages of particles finer than the diameters given by Stokes' law.

### 3.2.2 Atterberg Limit Determination

This is the determination of the degree of plasticity of the clay sample. It involves determining the liquid limit, plastic limit and linear shrinkage before and after the application of lime at a specified proportion.

#### ❖ Liquid limit

This is the moisture content in percentage, required to close a distance of 12.7mm (0.5") along the bottom of a groove after 25 blows or drop in a liquid limit device. It was determined by finding the moisture content corresponding 25 blows from the relationship between the moisture content and the number of blows for four specimens of each sample when plotted.

#### ❖ Plastic limit

This was determined by finding the moisture content, in percentage at which the clay material (paste form) crumbles when rolled into threads of 3.2mm diameter. The average of the moisture content determine for the specimen of each clay samples gave the plastic limit for each sample.

#### ❖ Plasticity Index

This is the different between the liquid limit and the plastic limit. ( $PI = LL - PL$ ).

#### ❖ Linear shrinkage

It is a measure of the water content, in percentage at which there is no further change in the volume of the soil mass. It is obtained by measuring the length of the soil in the shrinkage mould after oven drying. The average of the values obtained for the specimens of each sample gave the linear shrinkage value.

### 3.2.3 Compaction Test

Soil compaction test is the determination of the mass of dry soil per cubic metre, when the soil is compacted in a specified manner, here, the West African Standard Compaction over a range of moisture content. The test was done using the compaction mould and a 4.5 kg metal rammer falling at approximately 450mm height. The test is performed on a 5kg sample of air-dried/sun dried soil passing 20mm B.S. sieve, mixed with water of about 4% initially. The moist was compacted into the mould in five layers of approximately equal mass; each layer

was given blows, dropping from the rammer height. The mould and the soil were then weighted. Thereafter, the compacted soil specimen was removed from the mould and placed on a large metal tray. A representative sample was then taken and the moisture content of the specimen was determined. Suitable increments of water at 6%, 8%, 10%, and was added successively and mixed into the sample, and the above procedure was repeated for each increment of water added. The weight of moisture content was calculated by subtracting the weight of moisture content container and dry sample from the weight of moisture content container plus wet sample see Appendix C.

The moisture content in percent was calculated thus:

$$\text{Moisture content } \% = \frac{\text{weight of moisture} \times 100}{\text{weight of dry sample}}$$

#### 3.2.4 California Bearing Ratio (CBR) Determination

The aim of the test is to measure the shearing resistance of the soil to penetration under a controlled density and moisture conditions.

The operation used includes all that was in the compaction test in 3.2.3 above and a machine (CBR machine) for applying the test force through the plunger. It consists of a force measuring device and means of applying the force at a controlled rate.

The base plate was replaced by a perforated one and the collar fitted to the other end of the mould and the thread was packed with petroleum jelly.

For the soaked test, the specimen was placed in a bath of water and the water level was kept first above the top of the collar. After 24 hours, the soaked sample was then taken to the testing machine after it had dried for 15 minutes.

The sample was then taken to the CBR machine and reading of the force was taken at intervals of penetration of 0.065mm. The results of the CBR tests are plotted in Appendix C.

## CHAPTER FOUR

## 4.0 RESULTS AND DISCUSSION

## 4.1 Sieve Analysis

The geotechnical index properties of clay samples before the addition of lime are summarized in particle distribution forms in Appendix A1-A3, and the grain size distributions of untreated clay are all in the appendix afore mentioned. The index properties of the clay samples show that 'A' sample (Odo – ona clay) falls below the standards recommended for most geotechnical construction works as Appendix A shows high volume fine grains and would therefore require modification before stabilization while 'B' sample (Ibadan grammar) and 'C' sample (Ogunpa) can be stabilize to meet the engineering standard.

## 4.2 Atterberg Limits

The results obtained when turn test was carried out showed that sample 'A' has the highest liquid limit of all the three samples. Table 4.1 and Appendix B1-B3 show all the atterberg test results. Using the Cassagrande plastic chart, the sample 'A' can be said to be inorganic clay of high plasticity HI. Sample 'A', 'B' and 'C' were treated with lime and the results are shown on table 4.2. Liquid limit and plastic index decrease with increase in lime contents.

**Table 4.1: Variation of atterberg limit indices without lime content on samples A, B and C.**

Samples	Liquid Limit (LL) %	Plastic Limit(PL) %	Plastic Index (PI) %	Plasticity
A	66	20	46	High
B	47.5	18.2	29.3	Low
C	36	12.8	23.2	Low

**Table 4.2: Variation of atterberg limit indices with lime content on samples A, B and C.**

Lime Content %	SAMPLES								
	Odo-ona (A)			Ibadan grammar schl (B)			Ogunpa (C)		
	LL %	PL %	PI %	LL %	PL %	PI %	LL %	PL %	PI %
0	66	20	46	47.5	29.3	18.2	36	12.8	23.2
2	62	28	34	46.3	30.6	15.7	35.6	14.8	20.8
4	60	29	31	45.6	30.8	14.8	34.1	17.0	17.1
6	58	34	24	43.7	31.2	12.5	33.0	18.4	14.6
8	52	35	17	40.1	30.1	10	30.2	17.8	12.4
10	48	30	18	37.8	28.3	9.5	27.5	16.65	10.85
12	47	30	17	32	21.5	10.5	25	19	6

### 4.3 Compaction and California Bearing Ratio Tests

The results obtained from the test on sample 'A' showed that the maximum dry density (MDD) was 1.51 Mg/m<sup>3</sup>, optimum moisture content(OMC) was 15.2 % and the CBR was 10.5 % which is far below 40 % CBR prescribed for use of material for sub base of lightly trafficked roads. Compaction and Atterberg results on sample 'A', 'B' and 'C' are stated on table 4.3. Likewise, the effects of lime applications on CBR on sample 'A', 'B' and 'C' can be seen on table 4.4 and Appendix E.

Despite the fact that samples were treated with lime, the peak CBR values were 38 %, 50 %, and 48 % for samples 'A', 'B' and 'C' respectively. This shows sample 'A' failed to meet the 40 %

CBR recommended while samples B and C can be used as sub-base material. (as used by Osinubi, 1999). Nigerian General Specification for road and bridges, 1997 recommends LL <45 and PI <20 for fill and embankment materials, LL <45 and PI <15 for sub-base materials, and LL <30, PI <9 for base materials. The decrease in density according to Ola (1977) and Lees et al (1982) is as a result of the flocculated and agglomerated clay particles occupying larger spaces leading to a corresponding decrease in dry density. The increasing OMC with increasing lime content is as a result of the extra water required for the pozzolanic reactions.

**Table 4.3: Compaction and CBR results without lime treatment.**

Sample	Odo-ona (A)	Ibadan grammar schl. (B)	Ogunpa (C)
MDD (mg/m <sup>3</sup> )	1.51	2.05	3.5
OMC %	15.2	13	10
CBR %	10.5	18.5	25.1

**Table 4.4: Variation of California Bearing Ratio (CBR) with lime contents on sample A, B & C.**

Lime Content %	Samples								
	Odo- ona (A)			Ibadan grammar schl. (B)			Ogunpa (C)		
	MDD Mg/m <sup>3</sup>	OMC %	CBR %	MDD Mg/m <sup>3</sup>	OMC %	CBR %	MDD Mg/m <sup>3</sup>	OMC %	CBR %
0	1.51	15.2	10.5	2.05	13	18.5	3.5	10	25.1
2	1.49	19	12	2.08	12.5	23.1	3.53	9.5	33
4	1.47	20.5	19.8	2.10	11.75	32.3	3.54	9.2	42
6	1.43	22	27.5	2.12	11.5	40.5	3.57	8.7	48
8	1.40	23.1	38	2.15	10.8	50	3.60	8.0	34
10	1.39	24	29	2.19	11.0	41	3.62	8.5	23
12	1.38	25	20	2.18	11.1	34	3.59	9.1	18

## CHAPTER FIVE

### 5.0 CONCLUSION AND RECOMMENDATIONS

#### 5.1 Conclusion

- ❖ Result indicated that sample 'A' falls under the A-7-6 subgroup of the AASHTO classification system and using Plasticity classification chart, sample A, B, and C are CH, MI and CI respectively. California bearing ratio started falling immediately after 8 % lime content was added, this means lime cannot be a stabilizer to sample 'A' but rather a modifier. Cement, bitumen and other additive may be used as stabilizing agents for A.
- ❖ The results from the tests carried out confirmed that the addition of lime to clay samples affect some geotechnical properties; the liquid limit, plastic limit, plastic index and California bearing ratio (CBR). Hence samples B and C are good for sub-base and base respectively.

#### 5.2 Recommendations

- ❖ Test may be carried out on clay, silt and other challenging materials in a wider range across Nigeria to aid design and construction of projects.
- ❖ A linear equation can be developed relating the soil property indices, such as liquid limit, plastic limit and plastic index and California Bearing Ratio which will be handy tool for highway engineers in the field.

### DEDICATION

I dedicate this work to God Almighty who has enabled me to start and finish this work with sound health and mind. May His name be exalted for ever.

### ACKNOWLEDGEMENT

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**REFERENCES**

1. Agbede, I.O. (1998) Geotechnical Properties of Igumale Shale in Nigeria. Ph.D Thesis.
2. Agbede, I.O. and Joel Mannaseh (1998). Effect of Lime on Geotechnical Properties of Igumale Shale in Nigeria. pp 2-3. [www.edge.com](http://www.edge.com).
3. Aitsebaomo, F.O (1998). Geotechnical characteristics of sub grade soils in South Eastern Nigerian. *Journal of Construction and Material Technology*, 1(1): 52-72.
4. Akawwi, E. and Al-Kharabsheh, A. (2002) "Lime stabilization Effects on Geotechnical properties of Expansive soils in Amman, Jordan". *Electronic Journal of Geotechnical Engineering*.[www.ejge.com/2000 ppr 0020](http://www.ejge.com/2000_ppr_0020).
5. Alao, D.A. (1982). Geology and Engineering properties of laterites from Ilorin, Nigeria.*Engineering Geology*, 19: 111-118.
6. Arokodare, L.O, Adigun, S.A and Etteh, E (1971). Current approach to the development of major rural highways in south-western Nigeria. A seminar presented at Nigeria society of Engineers meeting, Lagos branch, 1-51.
7. Arumala, J.O and Akpokodje, E.G (1987). Soil properties and pavement performance in the Nigeria Delta. *Quarterly Journal of Engineering Geology*, London, 20: 287-297.
8. BRE (1980) Building Research Establishment: Low – rise building on shrinkage clay soils: part 1. Digest 240. Garston, Watford, HMSO.
9. British Standards institutions (1975). Methods of test for soils for civil engineering properties, British standard Institution, London, 143.
10. Brown, R.W. (1996) "Practical Foundation engineering handbook, Mc – Graw – Hill, New York.
11. Chen, F.H. (1975) Foundations on expensive Soils. Elsevier Science Amsterdam.
12. Craig, R.F (1992). Soil Mechanics, Chapman and Hall, London. Fifth edition, 428.
13. De-Graft – Johnson, J.W.S, Bhatia, H.S. and Yebaoa, S.L. (1973) Geotechnical properties of Accra Shales. *Proc. 8th Int. Conf. Soil Mech. Found. Eng., Moscow*, 2: 97-104.
14. Derucher, K.N. (1999). Impact of water provision on Health Unpublished undergraduate project, LAUTECH, Ogbomoso, pp 4-5. Donald P. Coduto, (1998). *Geotechnic Engineering Principles and Practices*.
15. Eades, J.L., and Grim, R.E. (1960) "The reaction of hydrated lime with pure clay minerals in Soil Stabilization" *U.S. Highway Research Board Bulletin*, 262: 51 –63.
16. Federal Ministry of Works and Housing (1997). General specifications (Roads and Bridges). Federal Highway Department, II: 145-284.

17. Gurcharm, S. and Jagdish, S. (1983). (Highway engineering for Degree, Diploma, A.M.I.E and field engineers). Khanna publishers, New Delhi, 651.
18. Ingles, O.G and Metcalf, J.B. (1972) Soil stabilization – Principles and Practice Buther Worths, Sydney, 374.
19. Jegede, G (2000) Pavement Failures caused by soil physical properties at a section of the F209 Highway at Ikole-Ekiti, Southwestern Nigeria. Journal of Biological and Physical science, 1: 48-54.
20. Johnson, W. A., Herrin, M., Davidson, T. D. and Handy, L.R. (1988) Soil stabilization in Highway Design. Reference Guide, edited by Woods, K. B and Ross, S. S. MC Graw – Hill Book Company. New York.
21. Khanna, P.N (1977). Indian Practical Civil Engineers’ Hand book Engineers’ publishers, New Delhi, fourth edition, 721.
22. Madu, R.M (1976). Sand- Laterite mixtures for road construction. Nigerian Journal of Technology, 1(1): 28-37.
23. O’ Flaherty C.A. (1974) “Highway Engineering” Vol. 2. Edward Arnold London, 95.
24. Oglesby, C.H and Hicks, R.G (1982). Highway Engineering, John Wiley and Sons, New York, Fourth edition, 847.
25. Ola, O.S and Raheem, A.A (2002). Soil tests for road construction. Journal of science Engineering and Technology, 9(2): 3971-3981.
26. Osinubi, K. J. and Katte, V. Y (1991) “Effect of Elapsed Time After mixing on Grain.
27. Scott, C.R. (1974). Soil mechanics and foundation. Applied science publishers limited, England, second edition, 361p.

APPENDIX A1

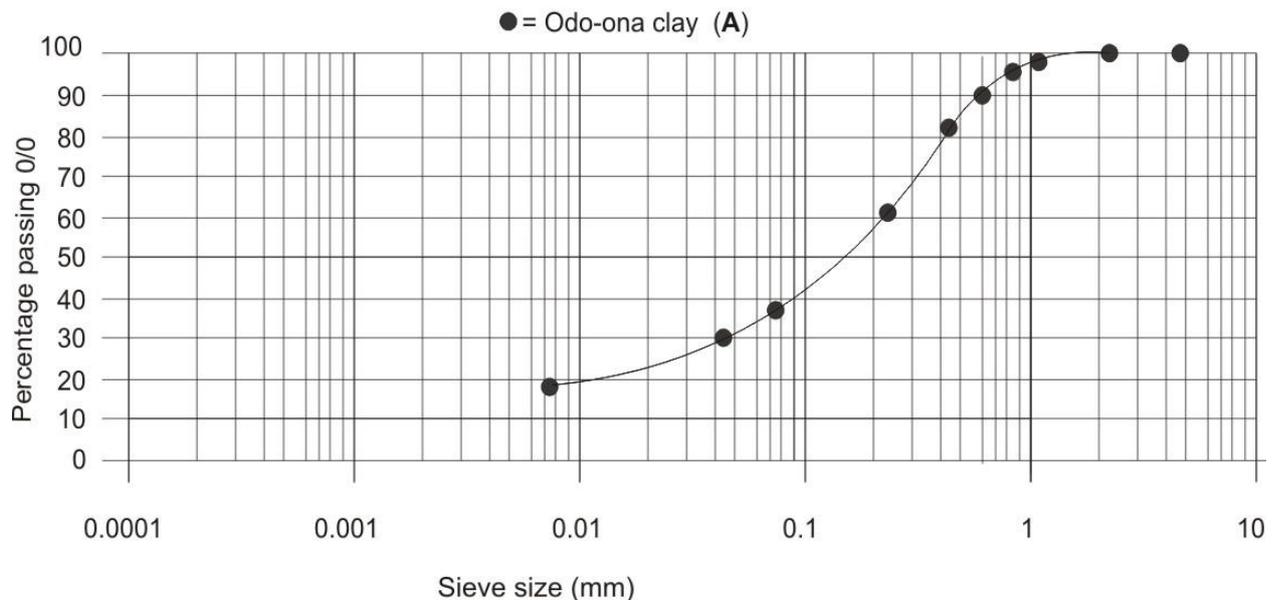
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DATE -----  
 OPERATOR -----  
 DEPTH -----  
 CONSISTENCY -----  
 JOB REF. -----  
 LOCATION -----  
 SAMPLE REF. -----

Sieve size Inch	Sieve size mm	Wt Retd.	0/0 Retd.	0/0 Passing	SPECIFICATION
2 !!	50.80				
1 ½!!	38.10				
1 !!	25.40				
¾ !!	19.10				
½ !!	12.70				
⅜ !!	9.50				
3/16 !!	4.75	–	–	100.00	
NO. 7	2.35	–	–	100.00	
14	1.18	50.80	2.00	98.00	
25	0.85	50.80	03.90	96.10	
36	0.60	50.80	11.32	88.68	
52	0.425	50.80	17.70	82.30	
72	0.212	50.80	38.50	61.50	
100	0.075	397.5	61.53	38.47	
200	0.045	446.9	69.18	30.82	
Through	0.0075	527.8	81.70	18.30	

TOTAL WEIGHT OF SAMPLE 7  
 WEIGHT RETAINED ON NO. 7 SIEVE  
 WEIGHT PASSING NO. 7,8 SIEVE  
 WEIGHT used for fine analysis  
 WEIGHT RETAINED ON NO. 200 BS SIEVE  
 RETAINED ON SIEVE ABOVE NO. 7.  
 RETAINED ON SIEVES BELOW NO. 7  
 EFFECTIVE SIZE (D)<sub>10</sub>  
 COEFFICIENT OF UNIFORMITY(D)<sub>60</sub>  
 (D)<sub>10</sub>

= gm A  
 = gm B  
 = gm C  
 = gm D  
 = WEIGHT RETAINED X 100 0/0  
 = WEIGHT RETAINED X C/A X 100 0/0  
 = mm



APPENDIX A2

PARTICLE SIZE DISTRIBUTION FORM SF.3

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 OPERATOR -----  
 DEPTH -----

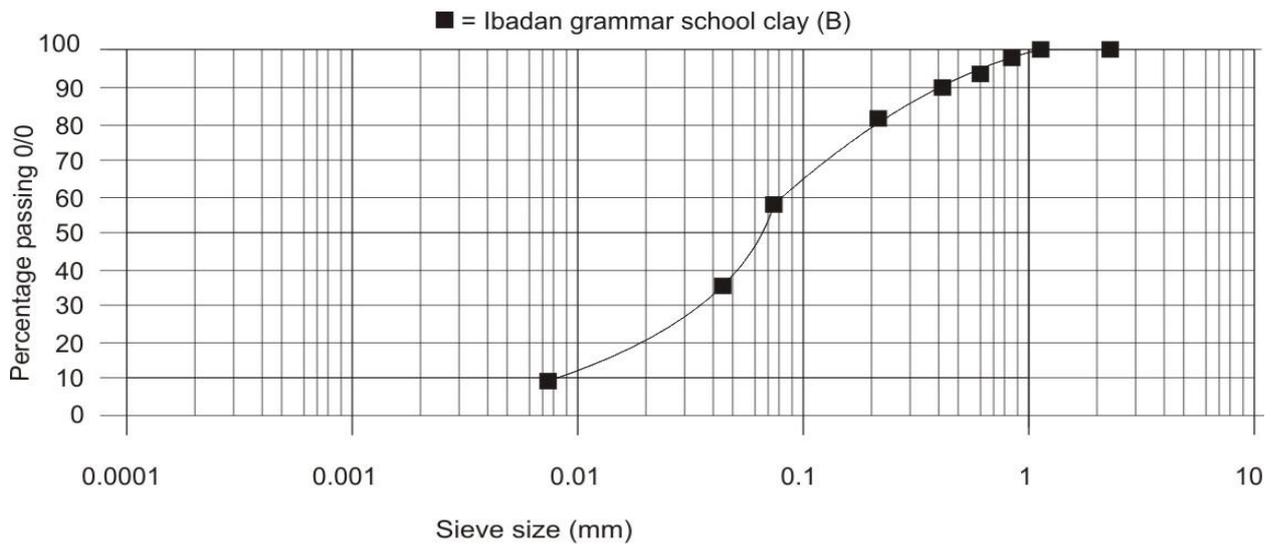
CONSISTENCY

JOB REF. -----  
 LOCATION -----  
 SAMPLE REF. -----

Sieve size Inch	Sieve size mm	Wt Retd.	0/0 Retd.	0/0 Passing	SPECIFICATION
2 !!	50.80				
1 ½!!	38.10				
1 !!	25.40				
¾ !!	19.10				
½ !!	12.70				
⅜ !!	9.50				
3/16 !!	4.75	–	–	–	
NO. 7	2.35	–	–	100.00	
14	1.18	–	–	100.00	
25	0.85	6.51	1.3	98.7	
36	0.60	17.54	3.5	91.5	
52	0.425	50.10	10.0	90.0	
72	0.212	89.20	17.8	82.2	
100	0.075	206.90	41.3	58.7	
200	0.045	375.20	74.9	35.1	
Through	0.0075	451.90	90.2	9.80	

TOTAL WEIGHT OF SAMPLE 7  
 WEIGHT RETAINED ON NO. 7 SIEVE  
 WEIGHT PASSING NO. 7,8 SIEVE  
 WEIGHT used for fine analysis  
 WEIGHT RETAINED ON NO. 200 BS SIEVE  
 RETAINED ON SIEVE ABOVE NO. 7.  
 RETAINED ON SIEVES BELOW NO. 7  
 EFFECTIVE SIZE (D)<sub>10</sub>  
 COEFFICIENT OF UNIFORMITY(D)<sub>60</sub>  
 (D)<sub>10</sub>

= gm A  
 = gm B  
 = gm C  
 = gm D  
 = WEIGHT RETAINED X 100 0/0  
 = WEIGHT RETAINED X C/A X 100 0/0  
 = mm



APPENDIX A3

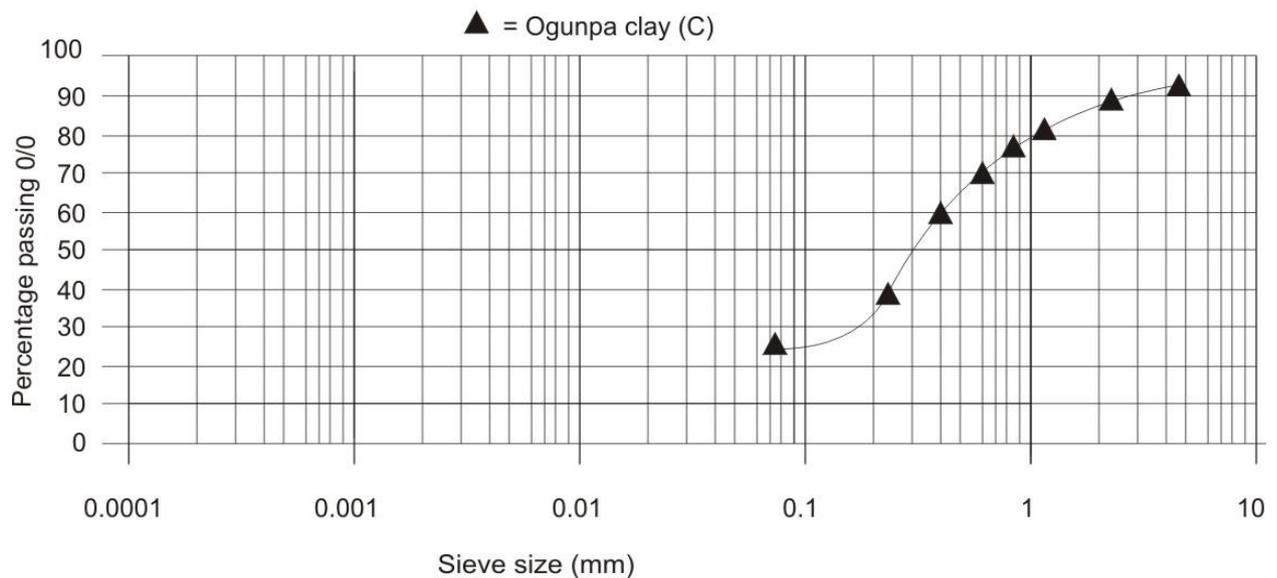
PARTICLE SIZE DISTRIBUTION FORM SF.3

DATE -----  
 OPERATOR -----  
 DEPTH -----  
 CONSISTENCY -----  
 JOB REF. -----  
 LOCATION -----  
 SAMPLE REF. -----

Sieve size Inch	Sieve size	Wt Retd.	0/0 Retd.	0/0 Passing	SPECIFICATION
2 !!	50.80				
1 ½!!	38.10				
1 !!	25.40				
¾ !!	19.10				
½ !!	12.70				
⅜ !!	9.50				
3/16 !!	4.75	44.6	9.9	90.1	
NO. 7	2.35	61.2	13.6	86.4	
14	1.18	78.3	17.4	82.6	
25	0.85	101.3	22.5	77.5	
36	0.60	135.9	30.2	69.8	
52	0.425	179.6	39.9	60.1	
72	0.212	278.1	61.8	38.2	
100	0.075	380.3	84.5	25.5	
200	0.045	450.0	100.0	—	
Through	0.0075	450.0	100.0	—	

TOTAL WEIGHT OF SAMPLE 7  
 WEIGHT RETAINED ON NO. 7 SIEVE  
 WEIGHT PASSING NO. 7,8 SIEVE  
 WEIGHT used for fine analysis  
 WEIGHT RETAINED ON NO. 200 BS SIEVE  
 RETAINED ON SIEVE ABOVE NO. 7.  
 RETAINED ON SIEVES BELOW NO. 7  
 EFFECTIVE SIZE (D)<sub>10</sub>  
 COEFFICIENT OF UNIFORMITY(D)<sub>60</sub>  
 (D)<sub>10</sub>

= gm A  
 = gm B  
 = gm C  
 = gm D  
 = WEIGHT RETAINED X 100 0/0  
 = WEIGHT RETAINED X C/A X 100 0/0  
 = mm

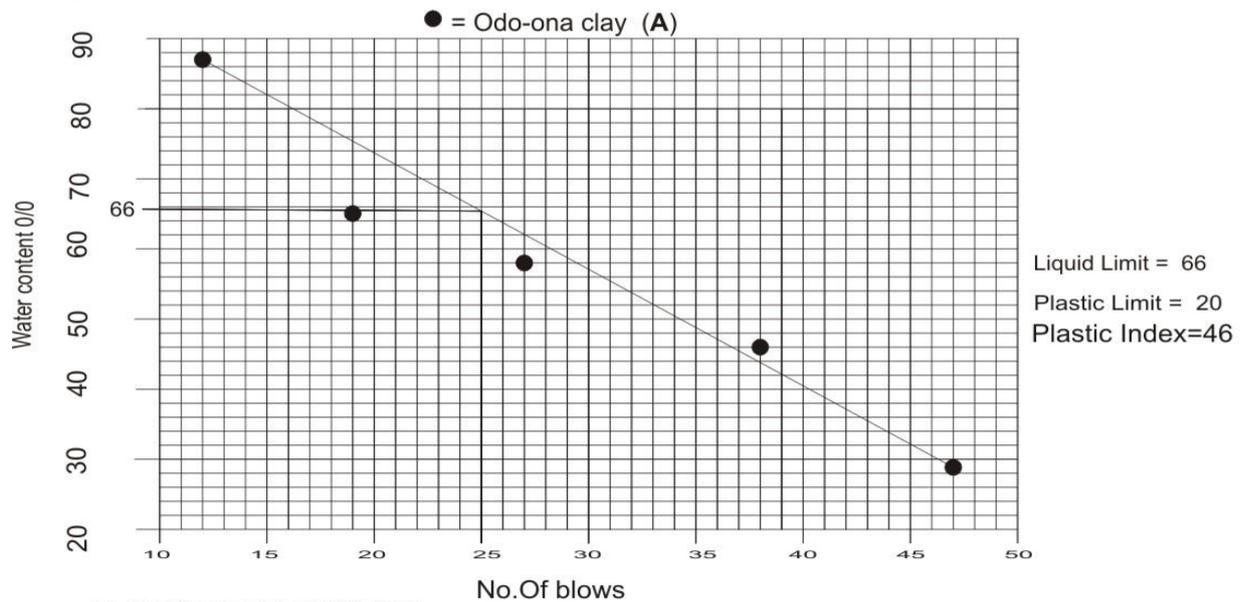


Appendix B1

Project: ----- **ATTERBERG LIMIT DETERMINATION** Source: -----  
 Contract: ----- Sample No.: -----  
 Location: ----- Date: -----

Liquid Limit Determination

Soil Sample	A				
Cont. No					
Wt. Of wet soil + cont.	3115.2	3342.2	3343.0	2969.2	4003.0
Wt. Of dry soil + cont.	2869.0	3036.0	3086.0	2817.0	3911.0
Wt. Of cont.	2586.0	2565.0	2643.0	2481.0	2588.0
Wt. Of dry soil	283.0	471.0	443.0	336.0	323.0
Wt. Of water	246.2	306.2	256.9	152.2	92.7
Water content 0/0	87	65	58	45.3	28.7
No. Of blows	12	19	27	38	47



PLASTIC DETERMINATION

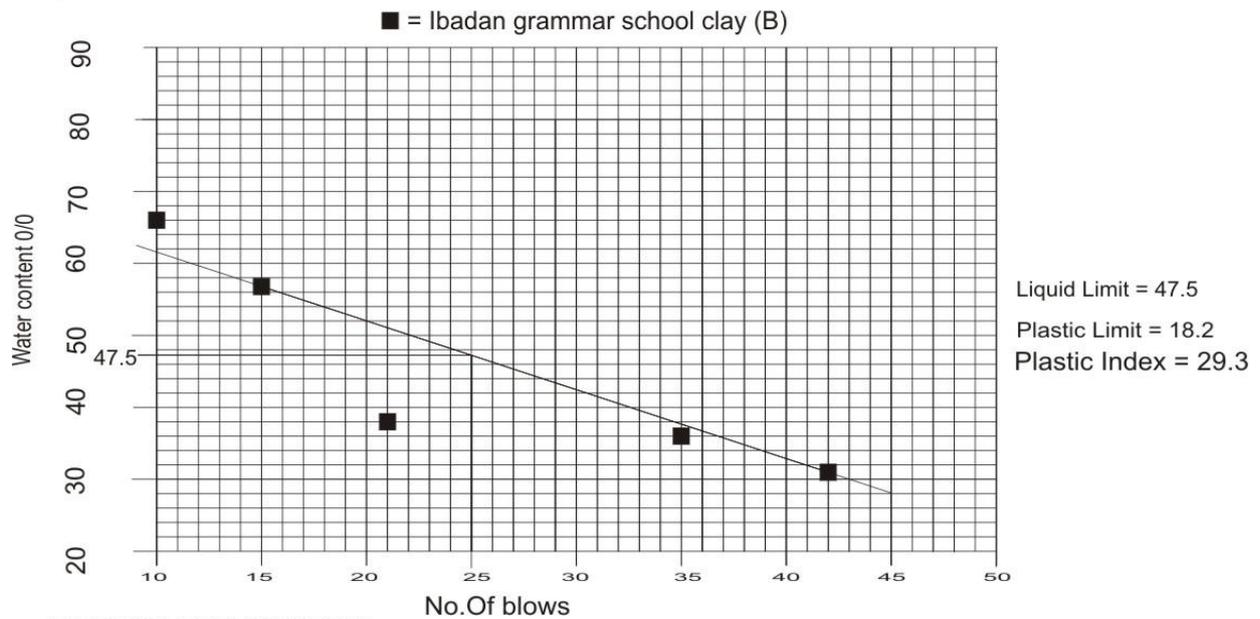
Soil Sample	A				
Cont. No					
Wt. Of wet soil + cont.	2792.0	2820.8			
Wt. Of dry soil + cont.	2744.0	2775.0			
Wt. Of cont.	2510.0	2540.0			
Wt. Of dry soil	243.0	235.0			
Wt. Of water	47.97	45.83			
Water content 0/0	20.5	19.5			
	20				

Appendix B2

Project: ----- ATTERBERG LIMIT DETERMINATION Source: -----  
 Contract: ----- Sample No.: -----  
 Location: ----- Date: -----

Liquid Limit Determination

Soil Sample	B				
Cont. No					
Wt. Of wet soil + cont.	2360.4	2809.5	2386.6	2314.0	1568.2
Wt. Of dry soil + cont.	2202.0	2541.0	2265.0	2192.0	1487.0
Wt. Of cont.	1962.0	2070.0	1945.0	1853.0	1225.0
Wt. Of dry soil	240.0	471.0	320.0	339.0	262.0
Wt. Of water	158.4	268.5	121.6	122.0	87.2
Water content O/O	66	57	38	36	31
No. Of blows	10	15	21	35	42



PLASTIC DETERMINATION

Soil Sample	B								
Cont. No									
Wt. Of wet soil + cont.	2264.0	2127.3							
Wt. Of dry soil + cont.	2217.0	2085.0							
Wt. Of cont.	1965.0	1847.0							
Wt. Of dry soil	252.0	238.0							
Wt. Of water	47.0	42.25							
Water content O/O	18.65	17.75							
	18.2								

Appendix B3

ATTERBERG LIMIT DETERMINATION

Project: -----

Contract: -----

Location: -----

Liquid Limit Determination

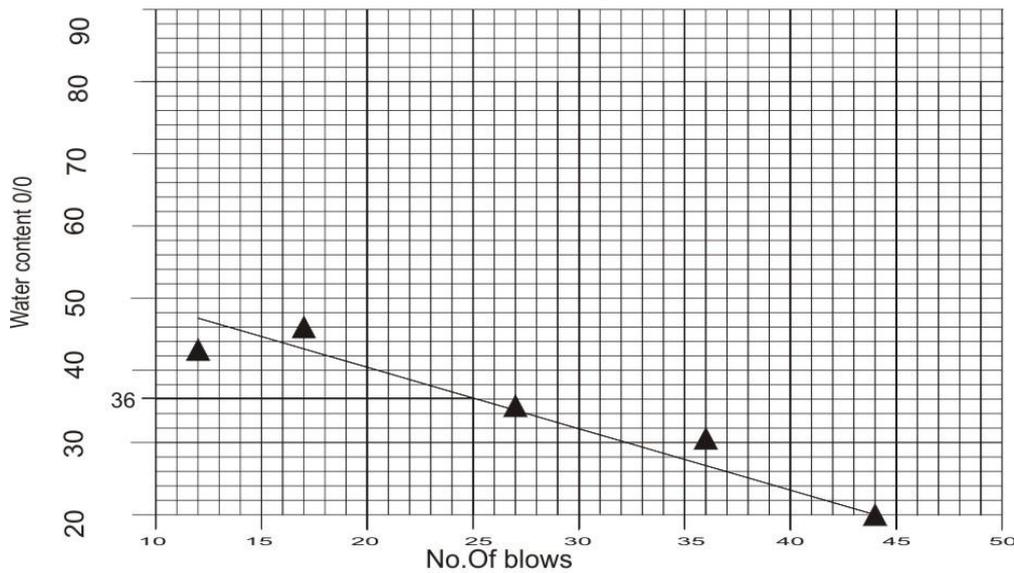
Source: -----

Sample No.: -----

Date: -----

Soil Sample	C				
Cont. No					
Wt. Of wet soil + cont.	2227.5	2296.8	2496.4	2376.8	2898.0
Wt. Of dry soil + cont.	2120.0	2191.0	2412.0	2297.0	2834.0
Wt. Of cont.	1870.0	1961.0	2171.0	2031.0	2514.0
Wt. Of dry soil	250.0	230.0	241	266.0	320.0
Wt. Of water	107.5	105.8	84.4	79.8	64.0
Water content O/O	43	46	35	30	20
No. Of blows	12	17	27	36	44

▲ = Ogunpa clay (C)



PLASTIC DETERMINATION

Soil Sample	C								
Cont. No									
Wt. Of wet soil + cont.	2731.9	2818.8							
Wt. Of dry soil + cont.	2698.0	2788.0							
Wt. Of cont.	2447.0	2540.0							
Wt. Of dry soil	251.0	248.0							
Wt. Of water	33.9	30.0							
Water content O/O	13.5	12.1							
	12.8								

APPENDIX C1

COMPACTION/CBR TEST

Project :-----		No. Layers :-----
Contract :-----		Wt. Of Hammer:-----
Location :-----		Drop Distance :-----
Source :-----	Weight (kg/lbs)	No. Of Blow :-----
Type of Test :-----	Volume (m <sup>3</sup> /ft)	Mold Size :-----

TEST DATA									
Mold No.									
Wt. Of water									
Wt. Of wet sample + mold									
Wt. Of mold									
Wt. Of wet sample									
Volume of sample									
Wet density		1.58		1.62		1.46		1.42	
Cont. No									
Wt. Of wet soil + cont.	119.32	111.33	100.64	119.5	97.57	89.64	122.6	128.35	
Wt. Of dry soil + cont.	109.43	102.6	89.2	104.8	85.25	77.86	103.2	107.46	
Wt. Of water	9.89	8.73	11.4	14.7	12.32	11.78	19.38	20.89	
Wt. Of cont.	16.93	16.2	16.8	16.1	16.45	16.16	16.72	16.65	
Wt. Of dry soil	92.5	86.4	72.4	88.7	68.8	61.7	86.5	90.81	
Moisture Content 0/0	10.7	10.1	15.8	16.6		19.1	22.4	23	
Average Moisture Content 0/0		10.4		15.2		18.5		22.7	
Dry Density		1.42		1.51		1.38		0.95	

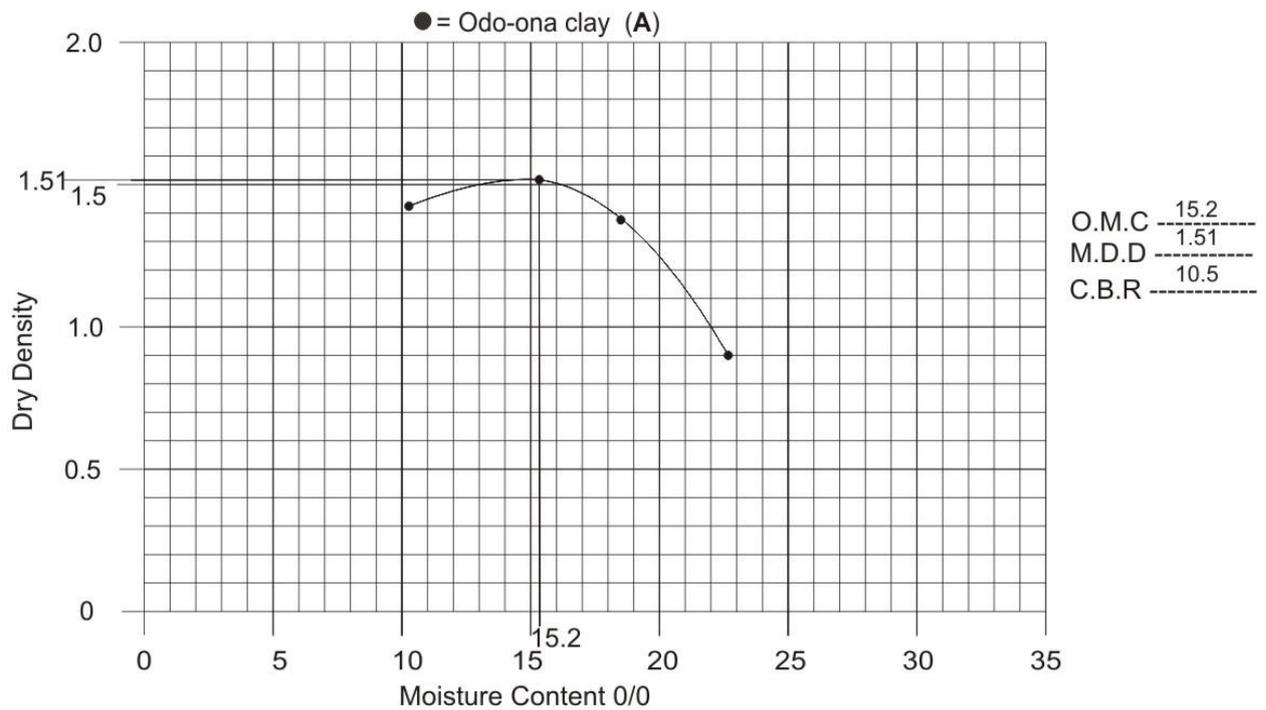


Figure C. Dry Density vs Moisture contents

APPENDIX C2

COMPACTION/CBR TEST

Project :-----		No. Layers :-----
Contract :-----		Wt. Of Hammer:-----
Location :-----	Weight (kg/lbs)	Drop Distance :-----
Source :-----	Volume (m <sup>3</sup> /ft)	No. Of Blow :-----
Type of Test :-----		Mold Size :-----

TEST DATA									
Mold No.									
Wt. Of water									
Wt. Of wet sample + mold									
Wt. Of mold									
Wt. Of wet sample									
Volume of sample									
Wet density		1.62		2.25		1.97		1.40	
Cont. No									
Wt. Of wet soil + cont.	126.12	125.57	125.3	125.5	118.0	116.1	129.87	125.11	
Wt. Of dry soil + cont.	118.6	117.2	113.3	112.95	103.7	103.0	110.7	107.6	
Wt. Of water	7.52	8.37	12.1	12.56	14.3	13.1	19.17	17.51	
Wt. Of cont.	18.4	18.7	18.2	18.15	17.5	18.0	17.2	17.8	
Wt. Of dry soil	100.2	98.5	95.0	94.8	86.2	85.0	93.5	89.8	
Moisture Content 0/0	7.5	8.5	12.75	13.25	16.6	15.4	20.5	19.5	
Average Moisture Content 0/0		8		13		16		20	
Dry Density		1.5		2.05		1.75		1.15	

■ = Ibadan grammar school clay (B)

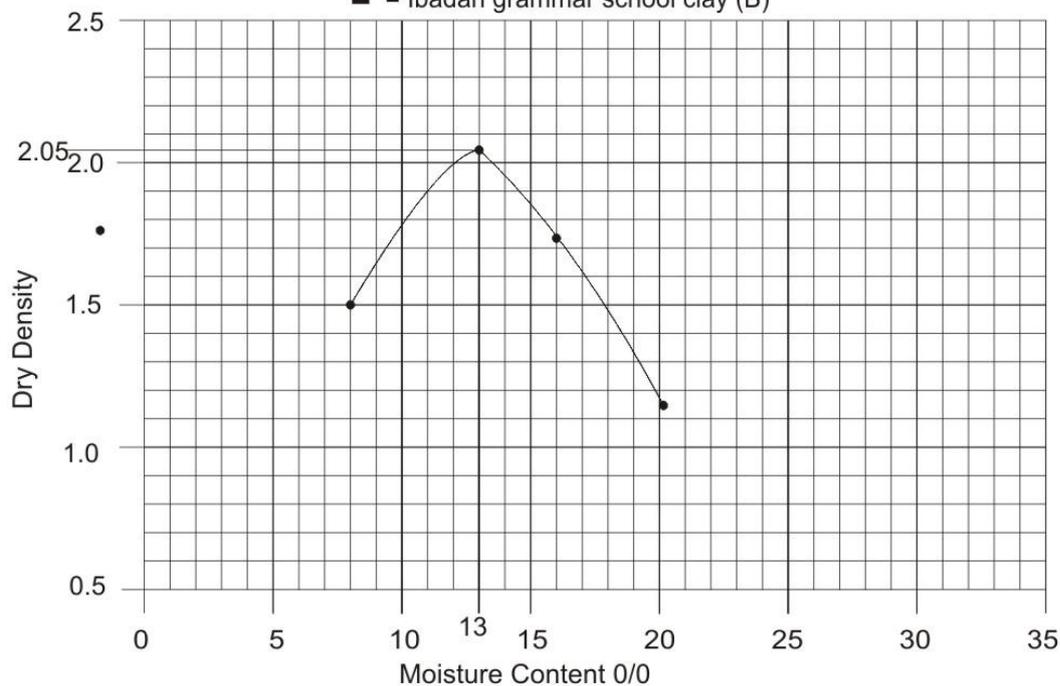


Figure C. Dry Density vs Moisture contents

APPENDIX C3

COMPACTION/CBR TEST

Project :-----		No. Layers :-----
Contract :-----		Wt. Of Hammer:-----
Location :-----		Drop Distance :-----
Source :-----	Weight (kg/lbs)	No. Of Blow :-----
Type of Test :-----	Volume (m <sup>3</sup> /ft)	Mold Size :-----

		TEST DATA							
Mold No.									
Wt. Of water									
Wt. Of wet sample + mold									
Wt. Of mold									
Wt. Of wet sample									
Volume of sample									
Wet density		2.35	2.4	2.43	2.51				
Cont. No									
Wt. Of wet soil + cont.		114.9	119.7	115.1	117.2	120.6	126	109.3	130.7
Wt. Of dry soil + cont.		109.6	113.7	106.6	107.7	106	113	96.1	114.1
Wt. Of water		5.35	5.96	8.5	9.52	14.58	13.03	13.19	16.56
Wt. Of cont.		17.3	17.6	17.1	17.0	17.1	17.2	17.6	17.8
Wt. Of dry soil		92.3	96.1	89.5	90.7	88.9	95.8	78.5	96.3
Moisture Content 0/0		5.8	6.2	9.5	10.5	16.4	13.6	16.8	17.2
Average Moisture Content 0/0		6	10	15	17				
Dry Density		2.2	3.5	2.7	1.8				

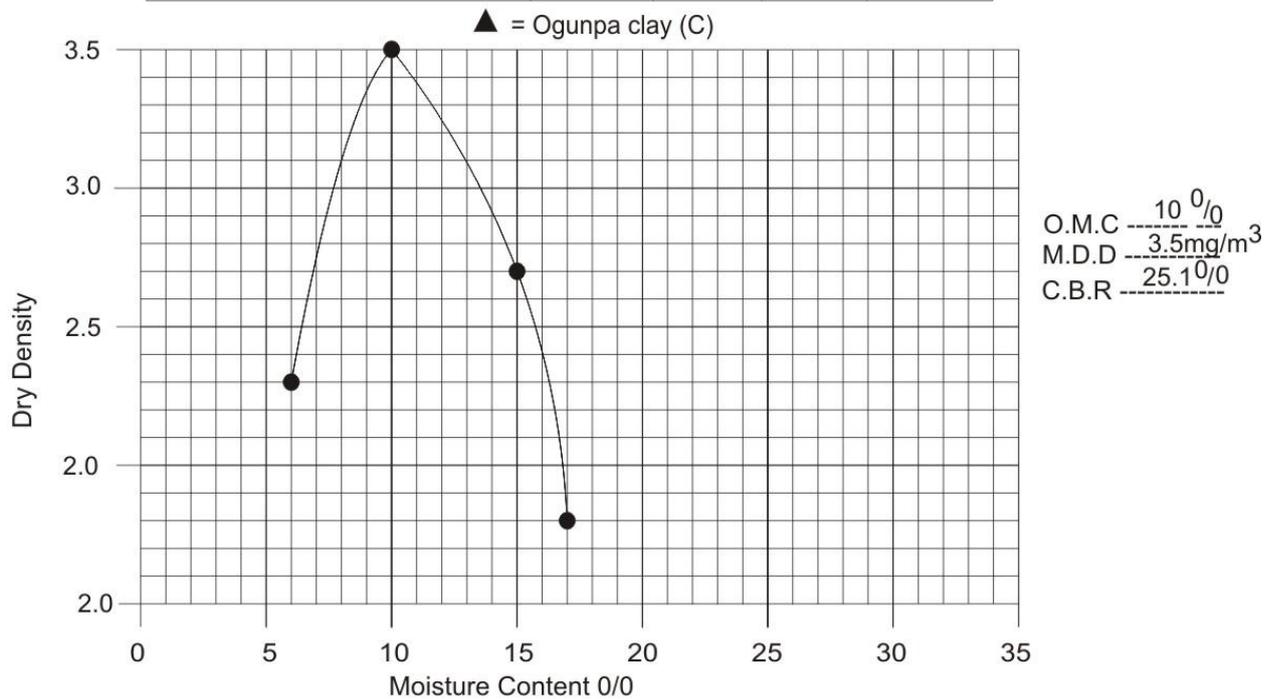


Figure C. Dry Density vs Moisture contents

APPENDIX D1 SAMPLE 'A' (ODO-ONA)

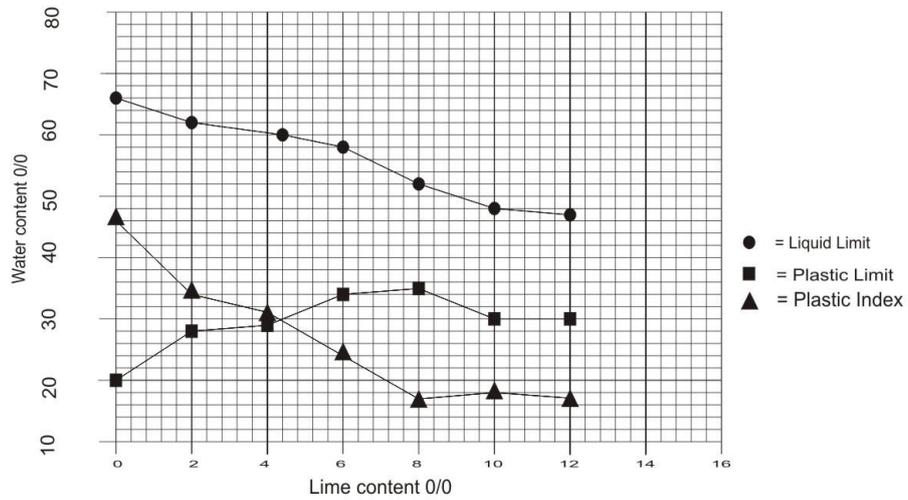


Fig.D11 Variation of atterberg limit indices with lime content

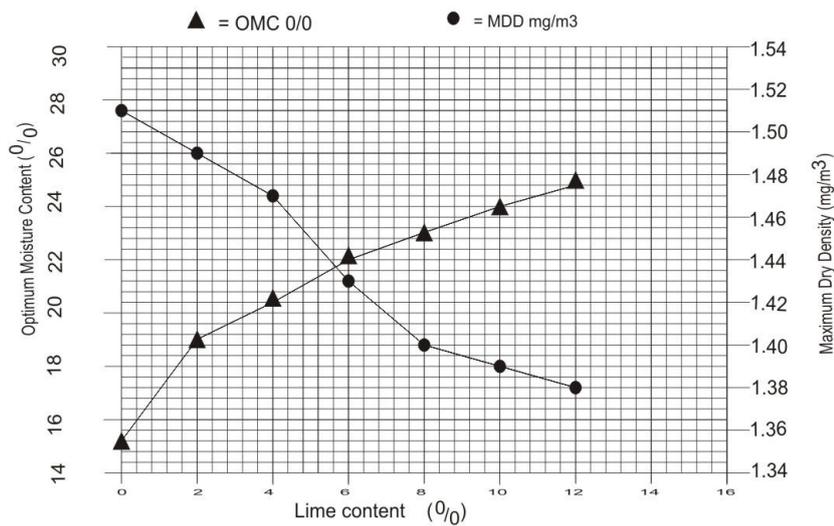


Fig. D12 Variation of Max Dry Density and Optimum Moisture Content with lime content

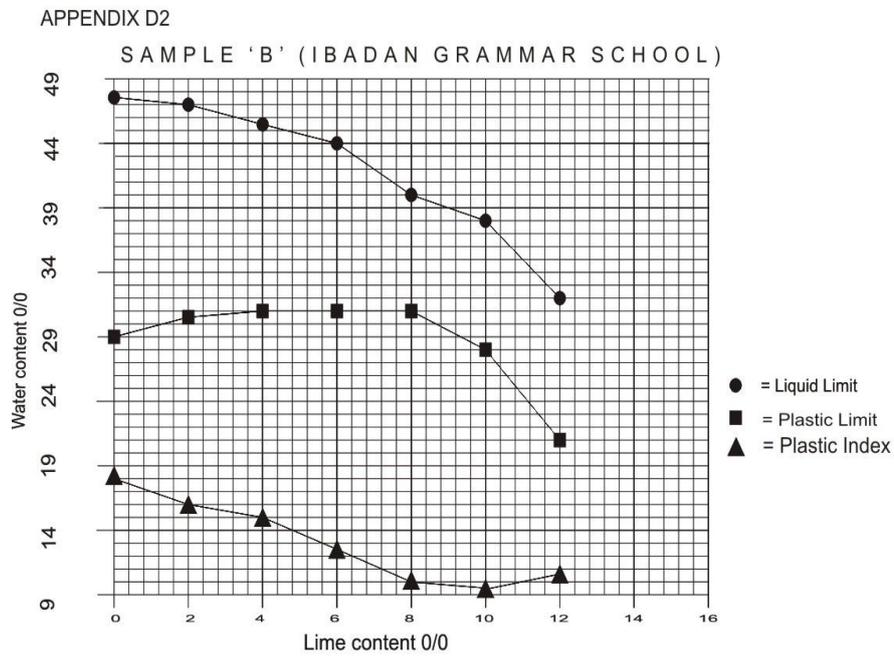


Fig.D21 Variation of atterberg limit indices with lime

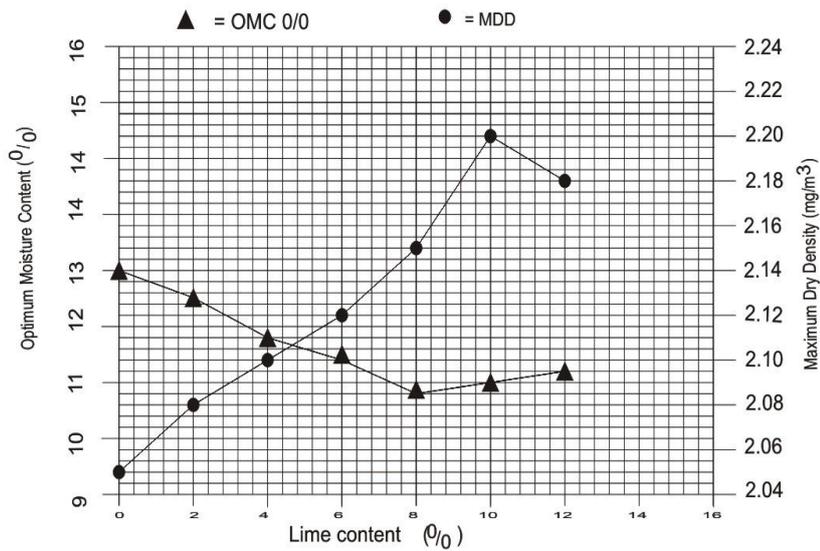


Fig. D22 Variation of Max Dry Density and Optimum Moisture Content with lime content

APPENDIX D3 SAMPLE 'C' (OGUNPA)

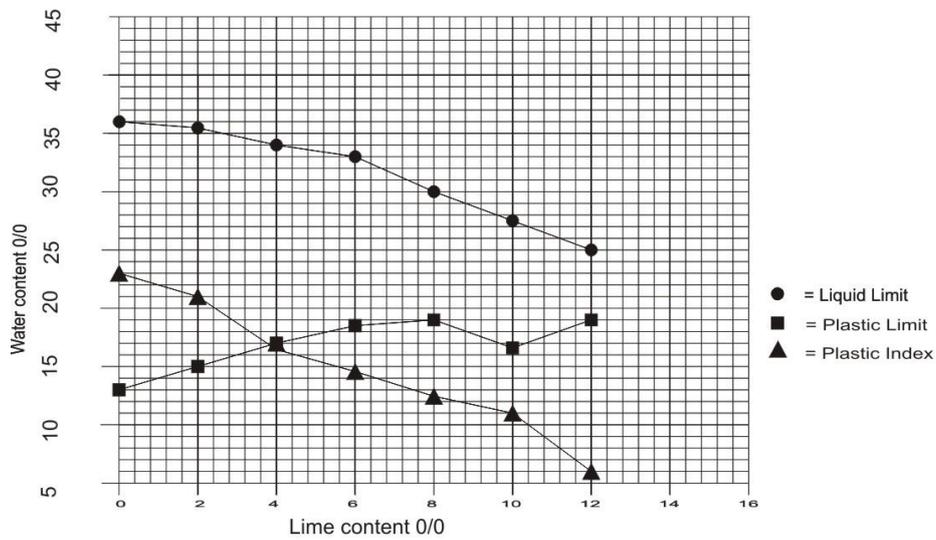


Fig.D31 Variation of atterberg limit indices with lime

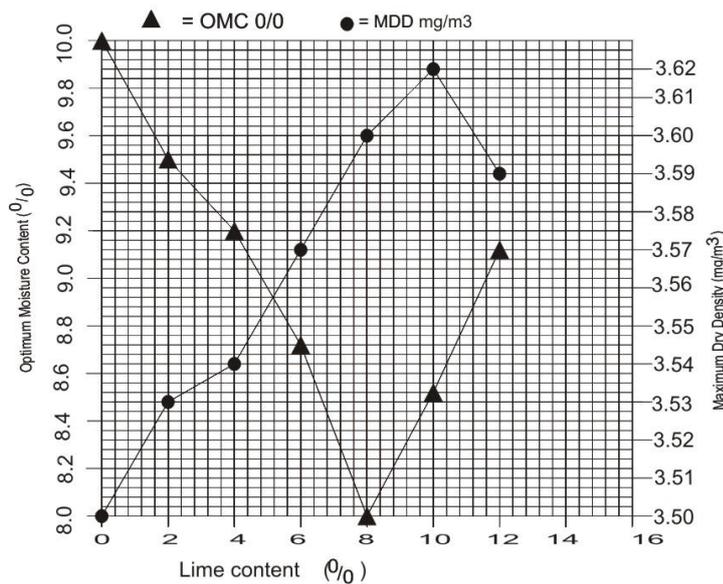


Fig. D32 Variation of Max Dry Density and Optimum Moisture Content with lime content

## APPENDIX E

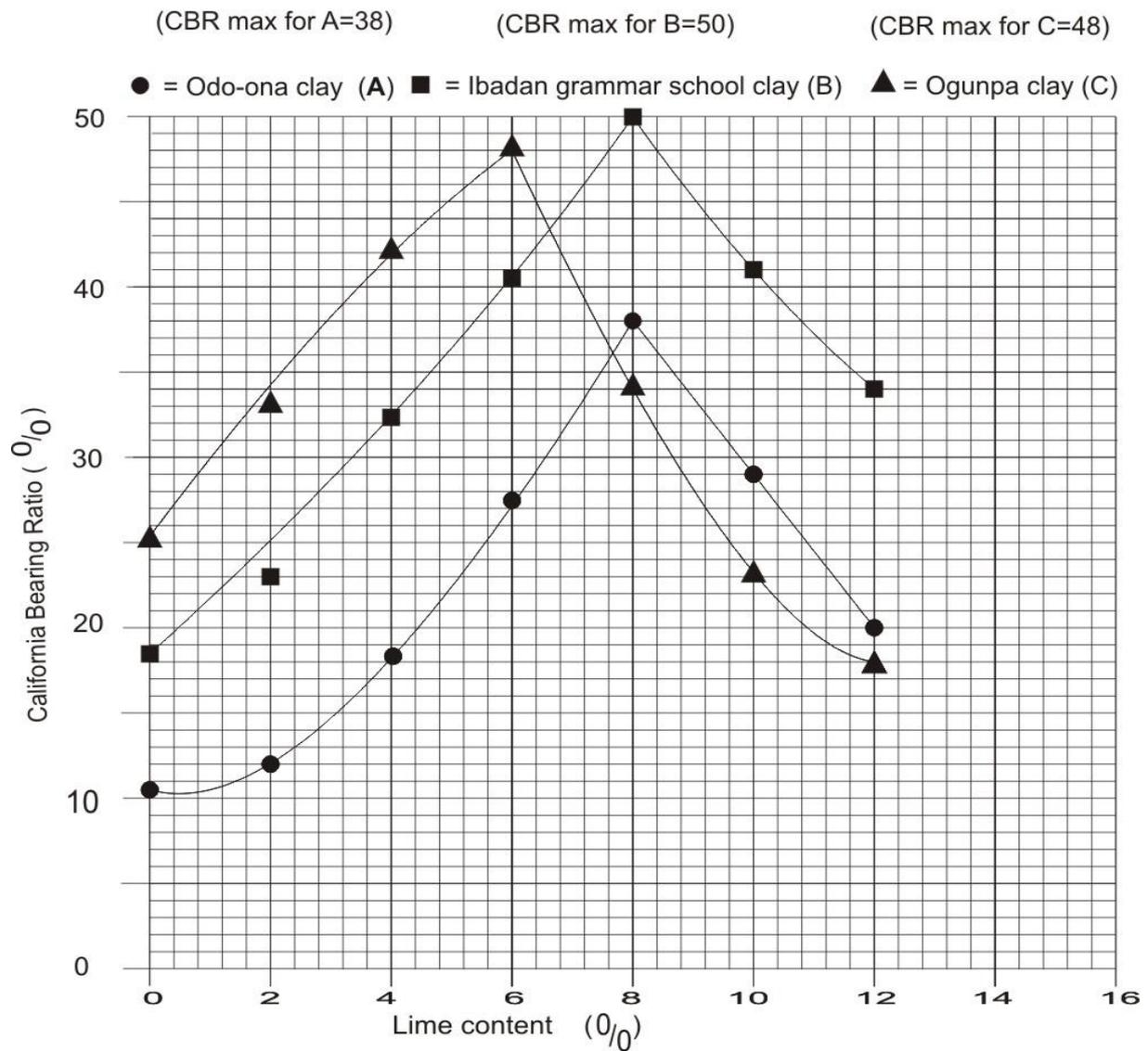


Fig. E. Variation of california bearing ratio (CBR) with lime content on A, B, and C.