GEOTECHNICAL CHARACTERIZATION OF SOILS IN IKOLE-EKITI AREA OF EKITI STATE

By

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ABSTRACT

Not only do soils serves as "underfoot" for roads, airfields, train tracks, and houses that sustain our complex society; Engineers, typically geotechnical engineers. classify soils according to their engineering properties as they relate to its use as foundation support or building material. The study of geotechnical characterization of soils in Ikole-Ekiti, Ekiti State was carried out to determine the index properties and strength properties of the soil. The laboratory test carried out included, Moisture content, Particle size analysis, Compaction test, Atterberg Limits tests, California Bearing Ratio, Direct Shear test and Permeability test. All analyses were carried out in accordance with the British Standard Institution As determined, recorded value ranging from 8.1 – 26.9% (moisture content). The results of particle grain size distribution test show that two of soil samples are lateritic in nature and two or clayey in nature. This means that they will tend to increase in compressibility and decrease in shear strength. Also, as a result of their poorly graded nature, they will have negative effects such as low effective porosity, small mean pore size, medium density and medium permeability. The compaction results show that the maximum dry density (MDD) and the optimum moisture content (OMC) of the locations range for 1.69 – 2.05kg/m3 and 14.3 – 20.70% respectively. Atterberg Limits of 31.80 – 58.50% (liquid limit). 15.10 -28.30% (plastic limit) and 13.70-33.80% (plasticity index). These results indicate that the soil is poorly graded, well drained, it is of intermediate plasticity, medium swelling potential. The lateritic soils were classified as A3, A24 and A26 and are adjudged suitable for subgrade, good fill and subbase and base materials. This geotechnical information obtained will serve as baseline information for future road foundation design and construction in the study area. The soils were classified as clayey A7 for two locations, A28 and A27 for the other locations and are adjudged suitable for subgrade, good fill and subbase and base materials. This geotechnical information obtained will serve as baseline information for future road foundation design and construction in the study area.

AKNOWLEDGEMENTS

I am grateful to God for His faithfulness in all my endeavours: to Him be the glory and adoration. He saved me through the dangers of the days and nights and kept me till now, forever I will praise Him.

May God bless my parents for their unending supports

May God bless my good supervisor and H.O.D., Department of Civil Engineering, Professor J.B. Adeyeri from whom I have learnt so much and as such owe so much. I sincerely appreciate your vast knowledge which you have shared with me. I pray you will God will keep you to reap the fruit of your labour.

I acknowledge the support received from every member of staff of the Department of Civil Engineering, all my able lecturers and technologists

DEDICATION

"...for His mercy endureth forever!"

This work is dedicated to the Almighty God who is my all; both great and small.

CERTIFICATION

This is to certify that this proposal was prepared by OTITOLAYE Olayinka Temidayo (CVE/11/0376) under my supervision, in partial fulfilment of the requirements for the award of a Bachelor of Engineering (B.Eng) degree in Civil Engineering, Federal University Oye-Ekiti, Ekiti State Nigeria.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
CBR	California Bearing Ratio
USCS	Unified Soil Classification System
IS	International Standard
S/N	Serial Number
LL	Liquid Limit
PL	Plastic Limit
MDD	Maximum Dry Density
OMC	Optimum Moisture Content

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CHAPTER ONE

INTRODUCTION

1.1 General Background

Successful engineering projects often involve the use of engineering principles in the appropriate manner which in turn answers concerns such as safety and economy. Such concerns include and is not limited to a proper understanding of site conditions on which projects are to be built.

Not only do soils serve as "underfoot" for roads, airfields, train tracks, and houses that sustain our complex society; they also provide the materials – wood, brick, sand, and gravel – used to build these facilities. "A cloak of loose, soft material, held to the earth's hard surface by gravity, is all that lies between life and lifelessness" (Fuller, 1975). To a civil engineer, a soil is an un-cemented earth material that can be moved with just a backhoe (i.e. without blasting, which would make it "rock" to an engineer). But scientifically, soil is a natural body consisting of layers (soil horizons) of mineral constituents of variable thickness, which differ from the parent materials in their morphological, physical, chemical, and mineralogical characteristics. It is composed of particles of broken rock that have been altered by chemical and environmental processes that include weathering and erosion. Soil differs from its parent rock due to interactions between the lithosphere, hydrosphere, atmosphere, and biosphere. In other words, it is a mixture of mineral and organic constituents that are in solid, gaseous and aqueous states.

In terms of soil texture, soil type usually refers to the different sizes of mineral particles in a particular sample. Soil is made up in part of finely ground rock particles, grouped according to size as sand and silt in addition to clay, organic material such as decomposed plant matter. Each component, and their size, plays an important role. For example, sand, determine aeration and drainage characteristics, while the tiniest, sub-microscopic clay particles are chemically active, binding with water and plant nutrients. The ratio of these sizes determines soil type: clay, loam, clay-loam, silt-loam, and so on. In addition to the mineral composition of soil, humus (organic material) also plays an important role in soil characteristics and fertility for plant life. Soil may be mixed with larger aggregate, such as pebbles or gravel.

Soil classification deals with the systematic categorization of soils based on distinguishing characteristics as well as criteria that dictate choices in use. Soil classification is a dynamic subject, from the structure of the system itself, to the definitions of classes, and finally in the application in the field. Soil classification can be approached from the perspective of soil as a material and soil as a resource. Engineers, typically geotechnical engineers, classify soils according to their engineering properties as they relate to use for foundation support or building material. Modern engineering classification systems are designed to allow an easy transition from field observations to basic predictions of soil engineering properties and behaviours. There are two soil classification systems in common use for engineering purposes. The American Society for Testing Materials, ASTM is used for virtually all geotechnical engineering work and sometimes also for highway and road construction. American Association of State Highway and Transport Officials, AASHTO soil classification is another system that is frequently used. Both systems use the results of grain size analysis and determinations of Atterberg limits to determine soil's classification. Soil components may be described as gravel, sand, silt, or clay. A soil comprising one or more of these components is given a descriptive name and a designation consisting of letters or letters and numbers which depend on the relative proportions of the components and the plasticity characteristics of the soil.

1.2 Aim and Objectives

The aim of this research is to determine the geotechnical characteristics of soil in Ikole area of Ekiti state.

This research is carried out to obtain the following objectives;

i. To carry out some geotechnical tests such as, specific gravity, gradation, permeability, direct shear, consistency, compaction and California Bearing Ratio tests.

ii. To classify the soils, using the American Association of State Highway and Transport Officials (AASHTO) and Unified Soil Classification System (USCS) methods.
iii. To establish the economic value of the soils.

1.3 Justification of Study

The study is considered to be very important as it will investigate the properties of soil in Ikole, and signify the classification of the soil, which in turns guides the use of the soil.

1.4 Study Area

The soil that will be used for this research will be taken from four (4) locations in Ikole area of Ekiti state, Nigeria. The map of Ekiti state and the map of Ikole-Ekiti are shown in Plate 1.1 and Plate 1.2 respectively.

Ikole-Ekiti is the Headquarters of the old Ikole District Council, the defunct Ekiti North Division and the Headquarters of defunct Ekiti North Local Government and now Headquarter of Ikole Local Government. Ikole is about 65 kilometres from Ado, the capital of Ekiti State of Nigeria. The town is situated on a very plain and well-drained land on the northern part of the State – about 40 kilometres from the boundary of Kwara State. The population of the town according to the 1963 census is about 52,000. The town is gifted with good fertile farmlands which ensure future expansion of agriculture and allied industries as well as a high swell in its population growth.

Ikole is situated in the deciduous forest area of the State. Rainfall is about 70 inches per annum. Rain starts in March and peters out in November. The good drainage of the land makes it very suitable for agricultural pursuits. It is a common feature that trees shed their leaves every year during the dry season which begins in November and ends in February. The two seasons – Dry Season (November – February) and Rainy Season (early March – mid November) are quite distinct and they are very important to the agricultural pursuits of the people.

The people of Ikole are predominantly farmers. About 80% of the male adult population engages in farming. The male adults have large plantations of food crops such as yams, cocoyam, cassava, maize, beans, rice and plantains. Some male adults have and maintain plantations mainly through hired labour. The farmers also plant cash crops such as cocoa which used to be the mainstay of the economy of this area. kolanuts, palm produce, coffee, cotton and tobacco are planted in smaller scales.

In addition, there are people who are Tailors, Traders, Carpenters, Mansions, Bricklayers, Goldsmiths, Blacksmiths, Shoe-makers, etc. by profession. The women-folk engage in various trades – selling of cloths, food stuffs etc.

The Government establishments in Ikole include:

a. Specialist Hospital,
b. Ikole Water Scheme,
c. Staff Housing Scheme,
d. Agricultural Development Project,
e. Brigade Headquarters,
f. Sports Stadium Complex,
g. Local Government Secretariat Complex,
h. the Clinic,
i. Technical Secondary School started by the Community which has been taken over by the State Government,
j. Electricity Project,
k. Federal Government College,
l. Local Government Maternity Centres and Dispensary etc.
m. Federal Government Housing Scheme (In progress)
n. State judiciary's Chief Magistrate's Court,
o. High court and Grade I Customary Courts
p. Various Government Departments such as Education, Agriculture and Rural Development, Trade Industry and Co-operatives, Home Affairs Sports and Information etc have their offices well-established at Ikole.

q. The Sports Council is maintaining a standard Swimming Pool and a Lawn Tennis Court near the Offices of the Ministry of Works and Transport near the Ansar-Ud-Deen High School along Ifaki road.

r. The Nigeria (Divisional) Police Headquarters is along Ijesa-Isu road near Emiola Memorial Standard Hotel.

s. Federal University Oye-Ekiti. (Ikole Campus)

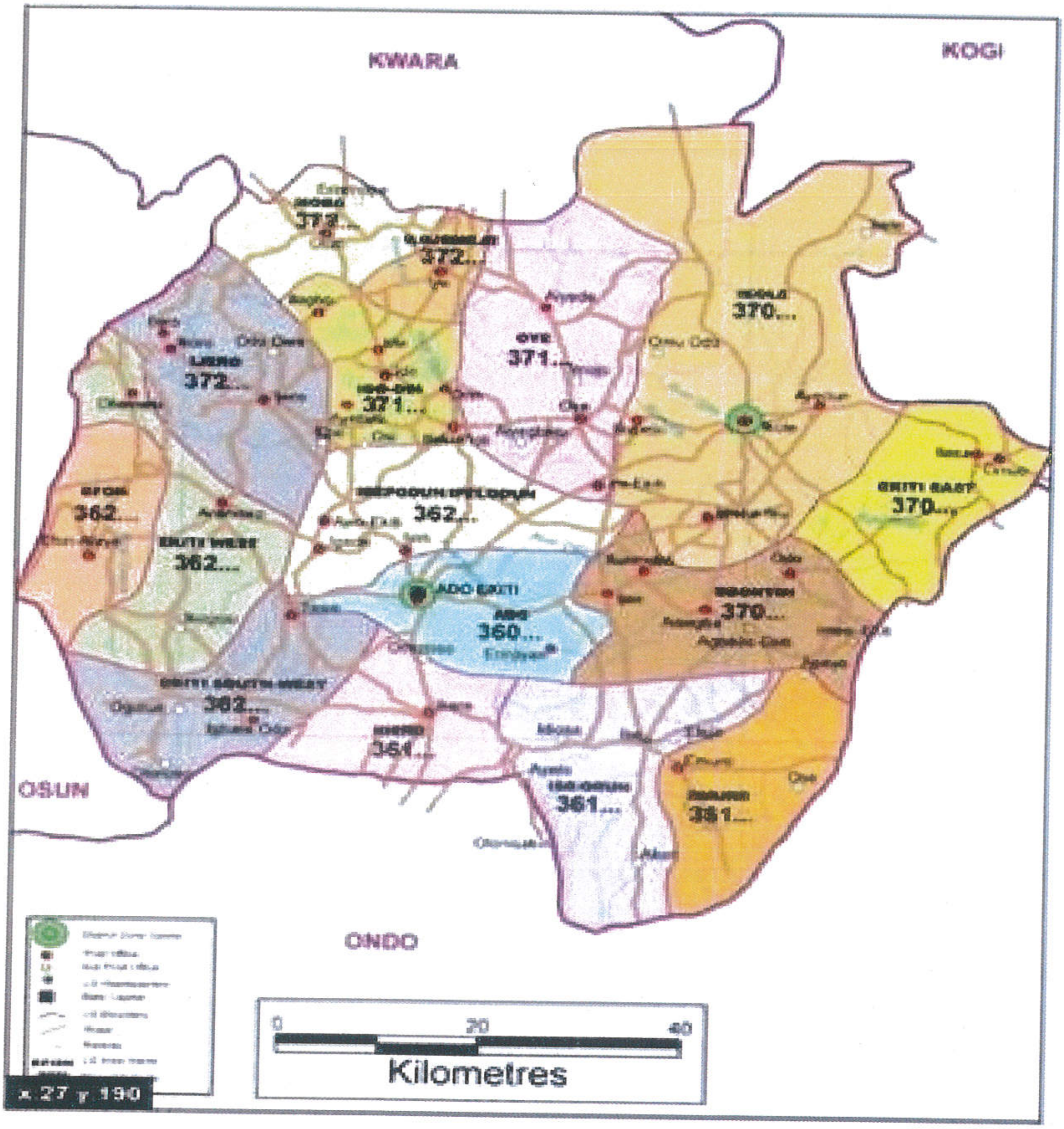


Plate 1.1: Map of Ekiti State

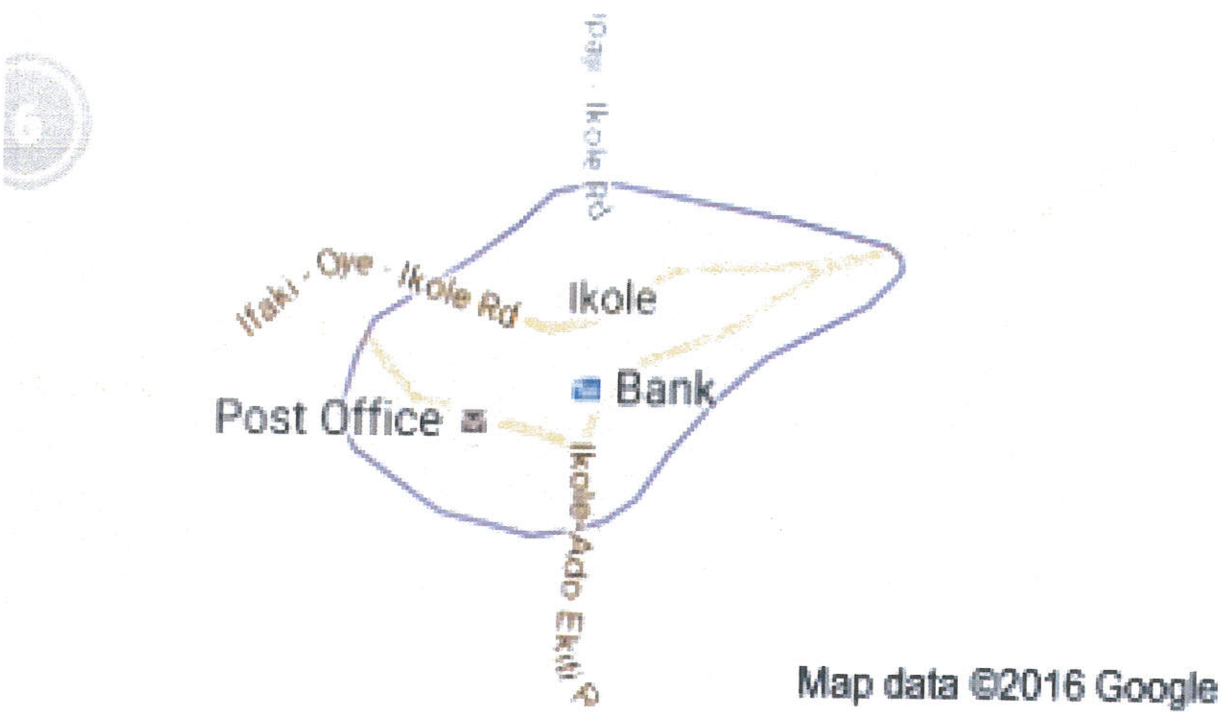


Plate 1.2: Map of Ikole-Ekiti, Ekiti State

1.5 Scope and Limitations of Study

The samples of disturbed soils will be collected from four (4) locations in Ikole-Ekiti and will be subjected to the following tests;

1. Permeability test
2. Index properties tests;
   i) Specific gravity
   ii) Sieve analysis
   iii) Consistency test
3. Strength tests;
   i) Compaction test
   ii) California Bearing Ratio (CBR) test
   iii) Direct Shear Test

The study is an investigation of the soil in Ikole area of Ekiti state. The research is limited to only four (4) locations for the whole area of Ikole land.

CHAPTER TWO

LITERATURE REVIEW

2.1 Geotechnical Engineering

Geotechnical engineering is the branch of civil engineering concerned with the engineering behavior of earth materials. Geotechnical engineering is important in civil engineering, but also has applications in military, mining, petroleum and other engineering disciplines that are concerned with construction occurring on the surface or within the ground. Geotechnical engineering uses principles of soil mechanics and rock mechanics to investigate subsurface conditions and materials; determine the relevant physical/mechanical and chemical properties of these materials; evaluate stability of natural slopes and man-made soil deposits; assess risks posed by site conditions; design earthworks and structure foundations; and monitor site conditions, earthwork and foundation construction. (Holtz et. al, 1981)

A typical geotechnical engineering project begins with a review of project needs to define the required material properties. Then follows a site investigation of soil, rock, fault distribution and bedrock properties on and below an area of interest to determine their engineering properties including how they will interact with, on or in a proposed construction. Site investigations are needed to gain an understanding of the area in or on which the engineering project will be built. Investigations can include the assessment of the risk to human being, property and the environment from natural hazards such as earthquakes, landslides, sinkholes, soil liquefaction, debris flows and rock falls.(Jon W. et al., 1989)

Foundations built for above-ground structures include shallow and deep foundations. Retaining structures include earth-filled dams and retaining walls. Earthworks include embankments, tunnels, dikes and levees, channels, reservoirs, deposition of hazardous waste and sanitary landfills.

Geotechnical engineering is also related to coastal and ocean engineering. Coastal engineering can involve the design and construction of wharves, marinas, and jetties.

Ocean engineering can involve foundation and anchor systems for offshore structures such as oil platforms.

The fields of geotechnical engineering and engineering geology are closely related, and have large areas of overlap. However, the field of geotechnical engineering is a specialty of soil engineering, where the field of engineering geology is a specialty of geology (Bowles, 1988).

2.2 Soil

Soil is the mixture of minerals, organic matter, gases, liquids, and the countless organisms that together support life on earth. Soil is a natural body known as the pedosphere which performs four important functions: it is a medium for plant growth; it is a means of water storage, supply and purification; it is a modifier of the atmosphere of Earth; it is a habitat for organisms; all of which, in turn, modify the soil.

Soil is considered to be the "skin of the earth" and interfaces with its lithosphere, hydrosphere, atmosphere, and biosphere(Chesworth, 2008). Soil consists of a solid phase (minerals and organic matter) as well as a porous phase that holds gases and water. Accordingly, soils are often treated as a three-state system. (McCarthy, 1982)

Soil is the end product of the influence of the climate, relief (elevation, orientation, and slope of terrain), organisms, and parent materials (original minerals) interacting over time (Giluly et.al, 1975). Soil continually undergoes development by way of numerous physical, chemical and biological processes, which include weathering with associated erosion.

2.2.1 Soil Description and Classification

It is essential that a standard language should exist for the description of soils. A comprehensive description includes the characteristics of both the soil material and the in-situ soil mass. Material characteristics can be determined from disturbed samples of the soil, i.e. samples having the same particle size distribution as the in-situ soil but in which the in-situ structure has not been preserved. The principal material characteristics are particle size distribution (or grading) and plasticity, from which the soil name can be deduced. Particle size distribution and plasticity properties can be determined either by standard laboratory tests or by simple visual and manual procedures. Secondary material characteristics are the colour of the soil and the shape, texture and composition of the particles. Mass characteristics should ideally be determined in the field but in many cases they can be detected in undisturbed samples, i.e. samples in which the in-situ soil structure has been essentially preserved. (Cernica, 1995)

A description of mass characteristics should include an assessment of in-situ compactive state (coarse soils) or stiffness (fine soils) and details of any bedding, discontinuities and weathering. The arrangement of minor geological details, referred to as the soil macro-fabric, should be carefully described, as this can influence the engineering behaviour of the in-situ soil to a considerable extent. Examples of macro-fabric features are thin layers of fine sand and silt in clay, silt-filled fissures in clay, small lenses of clay in sand, organic inclusions and root holes. The name of the geological formation, if definitely known, should be included in the description; in addition, the type of deposit may be stated (e.g. till, alluvium, river terrace), as this can indicate, in a general way, the likely behaviour of the soil. It is important to distinguish between soil description and soil classification. (Cernica,1995)

Soil description includes details of both material and mass characteristics, and therefore it is unlikely that any two soils will have identical descriptions. In soil classification, on the other hand, a soil is allocated to one of a limited number of groups on the basis of material characteristics only. Soil classification is thus independent of the in-situ condition of the soil mass. If the soil is to be employed in its undisturbed condition, for example to support a foundation, a full soil description will be adequate and the addition of the soil classification is discretionary. However, classification is particularly useful if the soil in question is to be used as a construction material, for example in an embankment. Engineers can also draw on past experience of the behaviour of soils of similar classification. (Townsend, 1973)

2.2.2 Details of Soil Description

A detailed guide to soil description is given in BS 5930. According to this standard the basic soil types are boulders, cobbles, gravel, sand, silt and clay, defined in terms of the particle size ranges, added to these are organic clay, silt or sand, and peat. These names are always written in capital letters in a soil description. Mixtures of the basic soil types are referred to as composite types.

A soil is of basic type sand or gravel (these being termed coarse soils) if, after the removal of any cobbles or boulders, over 65% of the material is of sand and gravel sizes. A soil is of basic type silt or clay (termed fine soils) if, after the removal of any cobbles or boulders, over 35% of the material is of silt and clay sizes. However, these percentages should be considered as approximate guidelines, not forming a rigid boundary. Sand and gravel may each be subdivided into coarse, medium and fine fractions. The state of sand and gravel can be described as well graded, poorly graded, uniform or gap graded. In the case of gravels, particle shape (angular, sub-angular, sub-rounded, rounded, flat, elongated) and surface texture (rough, smooth, polished) can be described if necessary. Particle composition can also be stated. Gravel particles are usually rock fragments (e.g. sandstone, schist). Sand particles usually consist of individual mineral grains (e.g. quartz, feldspar). Fine soils should be described as either silt or clay: terms such as silty clay should not be used. Fine soils containing 35–65% coarse material are described as sandy and/or gravelly SILT (or CLAY). Deposits containing over 50% of boulders and cobbles are referred to as very coarse and normally can be described only in excavations and exposures. Mixes of very coarse material with finer soils can be described by combining the descriptions of the two components, e.g. COBBLES with some FINER MATERIAL (sand); gravelly SAND with occasional BOULDERS.

2.3 Lateritic Soils

The fundamental characteristic of these soils is the nature and constitution of the mineral soil mass. Generally they are surface formations in tropical and subtropical areas which are enriched in sesquioxides of Iron (Fe_2O_3) and or Aluminum (Al_2O_3) and develop by intensive and long lasting weathering of the underlying parent rock. This enrichment/concentration may be residual accumulation or by solution movement or chemical precipitation (Maignien, 1966; Gidigasu, 1976; Charman, 1988; Fookes, 1997). In all cases, it is the result of secondary physico-chemical process and not the normal primary process of sedimentation, metamorphism, volcanism or plutonism. The accumulated hydrated oxides are sufficiently concentrated to affect the character of the deposit in which they occur. They may be present alone in an unhardened soil, as hardened layer or as a constituent such as concretionary nodules in a soil matrix or a cemented matric enclosing other materials (Charman, 1988).

2.3.1 Formation of lateritic soils

Lateritic soils are formed in hot, wet tropical regions with an annual rainfall between 750mm and 2000mm (usually in areas with a significant dry season) on a variety of different types of rocks with high iron content. The location on the earth, that characterize these conditions fall between latitude 35^0S and 35^0N (Maignien, 1966; Newill and Dowling, 1970; Gidigasu, 1975; Fookes, 1997).

Localization involves physico-chemical alteration of primary rock forming minerals into materials rich in 1:1 latice clay minerals (kaolinite) and laterite constituents (Fe, Al, Ti, Mn). In the first place, Ca, Mg, Na and K are released, leaving behind a siliceous framework consisting of silica tetrahedral and alumina octahedral. Silica which is soluble at all pH values, will be leached slowly while alumina and ferri sesquioxides (Al_2O_3, Fe_2O_3 and TiO_2) remain together with kaoline as the end product of clay weathering. The end result is a "reddish matrix" made from kaolinite, goethite and fragments of pisolitic iron crust (Maignien, 1966; Gidigasu, 1976, Charman, 1988; Fookes, 1997)

Two aspects of the parent rock affect the formation of laterite. One is the availability of iron and aluminium minerals. These are more readily available in basic rocks. The other is quartz content of the parent rock. Where quartz is a substantial component of the original rock, it may remain as quartz grains. Laterite profiles occur on flat slopes in the terrain where runoff is limited. On the level ground, where drainage is poor, expansive clay dominate at the expense of the laterite. From the above, three major stages have been identified in the process as follows (Maignien, 1966; Gidigasu, 1976).

1. **Decomposition:** Physico-chemical breakdown of primary minerals and the release of constituent elements (SiO_2, Al_2O_3, Fe_3O_3, CaO, MgO, K_2O, Na_2O, etc) which appear in the simple ionic forms.
2. **Laterization:** Leaching under appropriate conditions of combined silica and bases and te relative accumulation or enrichment of oxides and hydroxides of sesquioxides (Fe_3O_3, Al_2O_3 and TiO_2). The soil conditions under which the various elements are rendered soluble and removed through leaching or combination with other substances depend mainly on the pH and Eh of the groundwater and the drainage conditions. The level to which the second stage is carried depends on the nature and the extent of the chemical weathering of the primary minerals. Under conditions of low chemical and soil forming, the physic-chemical weathering does not continue beyond the clay forming stage and tends to produce end-products consisting of clay minerals predominantly represented by kaolinite and occasionally by hydrate or hydrous oxides of iron and Aluminum.
3. **Desiccation or Dehydration:** The partial or complete dehydration (sometimes involves hardening) or the sesquioxide rich material and secondary minerals. The dehydration of colloidal hydrated iron oxides involves loss of water and the concentration and crystallization of the amorphous iron colloids into dense crystals in the sequence; limonite, goethite, with haematite to hematite. Dehydration may be causes by climatic changes, upheaval of the land, or may also be by human activities for example by clearing of forests.

2.3.2 Global distribution of laterite soils

Laterites and lateritic materials are widely distributed throughout the world but occur more frequently in the tropics and subtropics of Africa, Australia, India, South-east Asia and South America (Maignen, 1966). The global distribution of laterite and associated materials is broadly governed by world climatic zones. Lateritic formation requires conditions of temperature and rainfall similar to those of the humid tropical and subtropical zones. There are other factors peculiar regions which govern the type of laterite which may be found, whether an indurated hardpan laterite, a nodular laterite or some other form. These factors include local variations in climate and the geology as well as the geomorphology associated with the weathering and soil development stages of landscape formation (Gidigasu, 1975).

2.3.3 Chemical and mineralogical composition

Clay mineralogical constitution of this soil is principally kaolinite often mixed with quartz. The higher proportion of sesquioxides of iron (Fe_2O_3) and aluminum (Al_2O_3) relative to other chemical components is s feature characteristic of all grades of lateritic soils. Those groups in which the iron oxide predominates are called ferruginous laterite soils and those in which alumina predominates – aluminous laterite soils. Iron is present usually as oxide minerals notably haematie (Fe_2O_3) and also as hydrated oxide – goethite (FeOOH) or as limonite (an amorphous mixture of hydrated oxide which retain various amounts of water). Aluminium occurs as its hydrated oxides gibbsite (Al_2O_3, $3H_2O$) and/or boehmite (Al_2O_3H2O). It is also contained with the lattice structure of kaolinite as an Aluminum silicate. Lateritic soils may contain significant amounts of manganese often identifiable as black nodules or concretions with titanium occur in limited quantities as titanium oxides. Zinc, chromium, nickel, cobolt, molebdenium, vandanium and other trace elements have been identified. Free silica is present as quartz inherited from the parent material. the predominant clay mineral is well-crystallized kaolinite.

2.3.4 Colour

Lateritic soils have characteristic reddish shades, which appear to be due to the various degrees of iron oxides – goethite and hematite, titanium and manganese hydration. The shades also reflect the degree of maturity. Generally, lateritic soils derive their colour from two sources.

1. From organic matter: Black, brown, grey
2. From mineral composition
   (a) Iron: red, orange, yellow, brown, blue and green.
   (b) Calcium, Magnesium, Sodium and Potassium: White
   (c) Aluminium: White
   (d) Manganese: Black, Brown.

2.3.5 Geotechnical properties of lateritic soils

Geotechnical characteristics and field performance of laterite soils as well as their reaction to different stabilizing agents may be interpreted in the light of all or some of the following parameters.

1. Genesis and pedological factors (parent materials, climate, topography, vegetation, period of time in which the weathering processes have operated)
2. Degree of weathering (decomposition, sesquioxide enrichment and clay size content, degree of leaching)
3. Position of the topographic site and
4. Depth of site in the profile (Gidigasu, 1976)

2.3.5.1 Particle size distribution

The particle size distribution of the soil may provide the following information:

1. A basis for identification and classification of the soil
2. The compactibility characteristics
3. Permeability
4. Swealllability and
5. A rough idea of deformation characteristics of the soil mass.

Texturally, lateritic soils are very variable and may contain all fraction sizes; boulders, cobbles, gravel, sand, silt and clay as well as concretionary rocks. Quartzitic gravels which are formed from the alteration of quartz rich parent rocks are generally well graded with 20% of silt and clay – size fraction. Concretionary laterites have a higher content of fines ranging between 35 – 40%. Foot slope concretionary laterite gravels are coarse and gap graded (less sand), compared to high level gravels (Gidigasu, 1976).

2.3.5.2 Plasticity characteristics of laterite soils

The interaction of the soil particles at the microscale level is reflected in the consistence limits or Atterberg limits of the soil. Knowledge of the Atterberg limit may provide the following information:

1. A basis for identification and classification of a given soil
2. Texture
3. Strength and compressibility characteristics
4. Swell potential of the soil or the water holding capacity.

Plasticity may be affected by present preparation, degree of moulding and time of mixing drying and rewetting and irreversible changes in plasticity on drying. Drying drives off adsorbed water which is not completely regained on rewetting (this is the case in both oven drying and air drying, Fookes, 1997). Soil which contains hydrated oxides of iron and aluminium may become less plastic on drying. This is partly because dehydration of sesquioxides creates a stronger bond between the particles which resists penetration by water. Studies on the relationship between the natural moisture content and liquid limit and plastic limit have shown that generally, the natural moisture content is less than the plastic limit in normal lateritic soils (Vergas, 1953). However, the lateritic soils from high rainfall areas may have moisture as high as the liquid limit (Ackroyd, 1971).

2.3.5.3 Compaction characteristics of lateritic soils

The composition characteristics are determined by their grading characteristics and plasticity of fines. These in turn can be traced to genetic and pedological factors. A significant characteristic of lateritic soils is the influence of the strength of the concretionary coarse particles on compaction. Most lateritic soils contain a mixture of quartz and concretionary coarse particles which vary from very hard to very soft. The strength of these particles has major implications in terms of field and laboratory compaction results and their subsequent performance in service. Weak coarse fractions break down under load with a resulting increase in fines of the soil (Ackroyd, 1971). The degree to which the materials break down is related to the content of iron oxide and the degree of dehydration. The higher the iron oxide content and the more the degree of dehydration, the harder the concretionary particles become. Placement variables (moisture content, amount of compaction, and type of compaction effort) also influence the compaction characteristics. Varying each of these placement variables has an effect on permeability, compressibility, swellability, strength and stress-strain characteristics (Lamb, 1958)

2.3.5.4 Strength properties of lateritic soils

Lateritic soils are weathered under conditions of high temperatures and humidity with well-defined alternating wet and dry seasons and continually leached by rainwater causing a tendency for deterioration of its strength characteristics. Shear strength characteristics of these soils have been found to depend significantly on the parent materials and the degree of weathering (i.e. degree of decomposition, laterization and dessication) which is a function of the position of the sample in the soil profile and the compositional factors (Lohnes *et al.*, 1971; Wallace, 1973). The higher the degree of laterization, the more favourable the shear strength parameters (Baldovin, 1969). Furthermore, the structural elements in the soil are often a less stable coarse-grained aggregation of variable strength which may break down in performance (Charman, 1988; Maigien, 1966) in addition to their varying silt and clay content which often render them moisture sensitive (Nicholson *et al.*, 1993; Ola, 1978). The aforementioned properties give an indication of their engineering limitations that restricts its uses and of such sites to minor engineering projects.

2.3.5.5 Permeability of laterite soils

Lateritic soils are distinguished from other soils by the presence of high proportion of sesquioides of iron (Fe_2O_3) and aluminium (Al_2O_3) relative to other chemical components. The sesquioxide gel within the fine fraction of the soil tends to coat the surface of individual soil particles. This coating reduces the surface activity of the clay mineral (principally kaolinite often mixed with quartz) and by extension the ability of the clay mineral to absorb water. It also causes a physical cementation of adjacent grains that result in the aggregation of particles to form coarser particles and a reduction in surface activity (15 m^2/g) and ability of the clay particles to absorb water. This factor also contributes significantly in lowering the plasticity of the soil and the inability of the soil to achieve very low hydraulic conductivity. The Base Exchange capacity of the clay mineral is usually very low (-5 meq/100g). Both factors reduce plasticity (Rowe *et al.*, 1995; Charman, 1988; Towsend *et l.*, 1971; Maigien, 1966) and invariably the ability of the soil to achieve low hydraulic conductivity.

2.3.6 Stabilization of lateritic soils

Stabilization may be defined as any processes by which a soil material is improved and made more stable. The goals of stabilization are therefore to improve the soil strength, to improve the bearing capacity and durability under adverse moisture and stress conditions and to improve the volume stability of a soil mass.

The tendency of lateritic gravels to be gap graded with depleted sand fraction and to contain a variable quantity of fines as well as to have coarse particles of variable strength which breakdown, limits their usefulness in some engineering projects such as road pavement (Charman, 1988). To ameliorate the above deficiencies and consequently to improve their performance characteristics, they need to be stabilized.

2.4 Geotechnical Properties of Soil

Geotechnical investigations are performed by geotechnical engineers or engineering geologists to obtain information on the physical properties of soil and rock around a site to design earthworks and foundations for proposed structures and for repair of distress to earthworks and structures caused by subsurface conditions.

2.4.1 Soil Sampling

Soil samples are often categorized as being either "disturbed" or "undisturbed;" however, "undisturbed" samples are not truly undisturbed. A disturbed sample is one in which the structure of the soil has been changed sufficiently that tests of structural properties of the soil will not be representative of in-situ conditions, and only properties of the soil grains (e.g., grain size distribution, Atterberg limits, and possibly the water content) can be accurately determined. An undisturbed sample is one where the condition of the soil in the sample is close enough to the conditions of the soil in-situ to allow tests of structural properties of the soil to be used to approximate the properties of the soil in-situ.

2.4.2 Geotechnical and Physical Properties of Soil

The geotechnical and physical properties of soil are indicated as follow;

2.4.2.1 Strength

The strength of a soil measures its ability to withstand stresses without collapsing or becoming deformed (Brady and Weil, 1996). Soil strength can be considered in terms of the capacity of a soil to withstand normal and/or shear stresses. Shear stress can be resisted only by the skeleton of solid particles, by means of the forces developed at the inter-particle contacts. Normal stress may be resisted by the soil skeleton due to an increase in the inter-particulate forces. If the soil is fully saturated, the water filling the voids can also withstand normal stress by an increase in pressure (Craig, 1992). A soil's ability to withstand normal stresses can be influenced by a number of related soil characteristics, amongst which are:

i) Bearing resistance
ii) Soil compressibility; and
iii) Soil compactability.

These factors in turn are determined by parameters such as soil moisture content, particle size distribution and the mineralogy of the soil particles. In general, coarser textured materials have greater soil strengths than those with small particle size (Brady and Weil, 1996). For example, quartz sand grains are subject to little compressibility, whereas silicate clays are easily compressed.

The bearing capacity of the material can be important both in terms of long-term engineering performance to carry loads and also supporting heavy plant in the short-term.

2.4.2.2 Compaction

Compaction is the process of increasing the density of a soil by packing the particles closer together with a reduction in the volume of air; there is no significant change in the volume of water in the soil. In general, the higher the degree of compaction, the higher the shear strength will be and the lower the compressibility of the soil (Craig, 1992).

The bulk density of a material is defined as the mass of a material (including solid particles, any contained water and any fluid stabiliser) per unit volume including voids. The dry density (ρ_d) is the mass of material after drying to constant mass at 105°C, and after removal of any fluid stabilisers, contained in unit volume of un-dried material (BS 1924: Part 1: 1990). The dry density of a material can be determined for a given compaction at varying moisture contents. This will determine the optimum moisture content at which a specified amount of compaction will produce a maximum dry density.

2.4.2.3 Collapse and Swelling

Certain soil formations are prone to volume change due primarily to variation in moisture content. For example, loess deposits are characterised by high void ratio, low unit weight and are incompressible when dry. However, when wet, or subject to dynamic loading or shock they can be prone to sudden collapse. Inundation collapse is also a common phenomenon associated with loose man-made fills.

Soils can swell due to rebound after a period of compression or as a result of the introduction of water. Montmorillonite clays, for example, characteristically swell when saturated leading to significant changes in volume. Swelling may also occur in soil due to the action of frost or from the exposure to air and moisture as in the case of some shale. Here expansion results from the formation of clay minerals. Swelling test requirements also exist for stabilised soils (MacNeil and Steele, 2001).

2.4.2.4 Particle size

Particle size is defined as the percentages of various grain sizes present in a material as determined by sieving and sedimentation (British Standard BS 1924: Part 1: 1990). BS 1924: Part 1: 1990 identified three classes of stabilised material depending on their particle size. These are shown in Table 2.1. Any material is regarded as belonging to the finest-grained group appropriate under the definitions given. Materials that contain large or irregular shaped particles can be difficult to test in the laboratory, and in the field they are likely to cause damage to the mixing plant. BS 1924: Part 1: 1990 stated that materials containing greater than 10% retained on the 37.5mm test sieve cannot be fully examined by the majority of test procedures given in that standard. This problem can be overcome by pre-screening to remove the large pieces or crushing the larger particles to within acceptable limits. The fine and medium-grained materials can be further classified as shown in Table 2.2. The grading of the material to be stabilised can influence the strength gain properties of the treated material. Well-graded materials have been found to exhibit a linear increase in unconfined compressive strength (UCS) with increased addition of cement binder (and lime binder before all the clay minerals have reacted).

Table 2.1: Classification of materials based on particle size distribution

Class	Definition
Fine-grained materials	Containing less than 10% retained on a 2mm test sieve
Medium-grained materials	Containing more than 10% retained on a 2mm test sieve but not more than 10% retained on a 20mm test sieve
Coarse-grained materials	Containing more than 10% retained on a 20mm test sieve but not more than 10% on a 37.5mm test sieve.

Source: BS 1924: Part 1: 1990

Table 2.2: Soil classifications and properties

Grain size	Coarse sand	Fine sand	Silt	Clay
Maximum (mm)	2	0.2	0.06	0.002
Average number of particles per g	350	350 000	3×10^8	3×10^{11}
Average surface area per g (cm^2)	40	400	4000	60 000
Typical mineralogical make-up	Quartz, feldspars, rock fragments	Quartz, feldspars, ferro-magnesium minerals	Quartz, feldspars, ferro-magnesium minerals, heavy minerals	Quartz, feldspars, secondary clay minerals
General Characteristics	Loose grained, non-sticky, air in pore space of moist sample. Visible to the naked eye.	Loose grained, non-stick, no air in pore space of moist sample, visible to the naked eye.	Smooth and flourlike, non-cohesive, Microscopic	Sticky and plastic, microscopic to sub microscopic, exhibit Brownian movement
Implications for Stabilization/Solidification (s/s)	Likely to be easily mixed. Potential for increased permeability (over well graded/fine grained soil)	Likely to be easily mixed. Potential for Increased permeability (over well graded/fine grained soil). May be moisture sensitive.	Sensitivity to mois ture change needs to be addressed at design.	Uniform mixing may be difficult, but clay is easily stabilised. Clay minerals can react with binders to form cementitious products.

Source: Townsend, 1973

The mean particle size is not reported to affect this phenomenon; therefore a linear increase in strength can be expected for either clays or gravels. However, uniformly graded materials are identified as the exception to this linear behaviour when smaller quantities of binder are added. Sherwood (1993) suggested that this is due to the binder acting as filler in uniformly graded materials. Once the binder has improved the grading of the material Sherwood (1993) reported a linear increase again.

2.4.2.5 Cohesion and Plasticity

The properties of clay minerals give unique engineering properties to clay soils: cohesion and plasticity. Cohesive material can be defined as all material which, by virtue of its clay content, will form a coherent mass. Non-cohesive (granular) material will not form a coherent mass (BS 1924: Part 1: 1990). Where soils that are predominantly coarse-grained contain sufficient fine grains to show apparent cohesion and plasticity, they will be classified as fine soils (BS 5930: 1999). As a consequence, a cohesive soil can comprise less than 10% clay-sized particles.

Knowledge of the cohesiveness of a soil assists in the selection of Stabilisation/Solidification (S/S) treatment methods. Due to the poor mixing characteristics of cohesive material, treatment using ex-situ (e.g. pug mill) S/S techniques may not be possible, without the inclusion of a lime-treatment step. The addition of lime to cohesive soils can result in a decrease in plasticity due to the flocculation of clay particles as well as a longer-term pozzolanic reaction. The initial change in plasticity can significantly improve the workability of the material, enabling existing treatment techniques to be used. The plasticity of a fine-grained soil can be measured by its Atterberg limits. The plastic limit is defined as the moisture content at which soil changes in texture from a dry granular material to a plastic material that can be moulded. With increasing moisture content a cohesive material becomes increasingly sticky, until it behaves as a liquid. The point at which this phenomenon occurs is known as the liquid limit. The range of moisture content between the plastic limit (PL) and the liquid limit (LL) is defined as the plasticity index (PI) i.e. LL – PL = PI. These concepts are illustrated in Figure 2.1.

The transition points are fairly arbitrary, determined by index tests described in BS 1377- 2:1990, but they do serve a valuable function in the classification of cohesive soils. With an increase in moisture content, granular soils pass rapidly from a solid to a fluid condition. In these circumstances the PL and LL cannot be identified and such soils are classified as non-plastic (Sherwood, 1993).

Increasing Moisture Content $\longrightarrow$

Powdered condition	Plastic condition	Liquid condition

P L

L $\longleftarrow$ Plasticity $\longrightarrow$ L

Index (PI)

PI = LL – PL

Figure 2.1: Definitions of soil plasticity (Sherwood, 1993)

Cohesive soils may be classified according to their plasticity properties. Silts have low plasticity indices, which mean that they quickly become difficult to handle once the moisture content exceeds the plastic limit. With increasing clay content in a soil, both the plastic limit and the liquid limit increases. The difference between the two limits may widen due to the activity of the clay minerals present (Sherwood, 1993, Cernica, 1995). The activity of clay minerals can be related to plastic index, fineness of clay particles and behavioural tendency to volume changes (Cernica, 1995).

Cohesive soils characteristically have high plasticity indices. Stavridakis and Hatzigogos (1999), state that in soils containing expansive clay minerals with high liquid limits (40-60%), the liquid limit can be used to gauge the amount of cement required to stabilise the soil. Although soils with liquid limits >60% can be stabilised, the amounts of cement required can be uneconomical and result in unacceptable volume increase.

2.4.2.6 Moisture content

The moisture content of a soil is the ratio of the mass of water to the mass of solids in the soil (Craig, 1992). The moisture content is determined as the mass of free water that can be removed from a material, usually by heating at 105°C, expressed as a percentage of the dry mass (BS 1924: Part 1: 1990). If a soil or waste contains too much water then the porosity and permeability are likely to increase. If the amount of moisture present in a soil is above optimum then the density of the compacted product is reduced and this may have an impact on the strength achieved in an S/S product. It is often necessary to adjust the moisture content in soils prior to S/S and this can be achieved by stockpiling and draining with time, by the addition of lime or by blending the soil with other materials. Alternatively, water can be added to soil that is too dry.

Drying soils with lime is commonly undertaken and it was traditional practice to allow a clay-lime mix to stand for a period of typically 24 h, either in a stockpile or for single layer treatment in situ, in order that complete lime distribution could occur. Current thinking, however, suggests that immediate water content adjustment and compaction is more beneficial in achieving a long-term strength gain (Holt and Freer-Hewish, 1996; Glendinning et al. 1998). Boardman (1999) stated that immediate compaction would undoubtedly be beneficial for contaminated soil treatment, as long as thorough mixing is possible, since the pozzolanic reaction bonds that form at an early stage would assist with contaminant retention and minimise the flow of water through the stabilised material.

2.4.2.7 Permeability

The term permeability expresses the coefficient of permeability or hydraulic conductivity, describing the rate (ms^{-1}) at which water can flow through a permeable medium. Permeability is related to the distribution of particle sizes, particle shape and soil structure. In general, the smaller the particles, the smaller the average size of the pores and the lower is the coefficient of permeability. The transport of water through a soil will be faster if the soil has a higher coefficient of permeability than if it has a lower value (Craig, 1992). However, it should be noted that the rate of transport of contaminants depends upon a number of factors including solubility and the rate at which contaminants are attenuated in a soil. The determination of the coefficient of permeability using the constant head method or in a cell under known effective stress conditions are described in BS 1377: 1990.

2.4.2.8 Frost heave and Frost shattering

Sherwood (1992) described frost heave as an effect that can occur when temperatures are sub-zero for several days. If it is possible for water to move from the water table to the frozen zone easily, it will be drawn up into this zone where it will freeze to form ice lenses. Once this has occurred, further water may be drawn up and be subsequently frozen. It is expansion due to the freezing of transported water that is the primary cause of frost heave, rather than freezing of water originally present. Permeability of the material is identified as the leading factor behind frost heave susceptibility.

Frost shattering is identified as the result of expansion of excess water present in voids of the surface of the material as it freezes (Sherwood, 1992).

2.4.2.9 Temperature

The disruption of structure of a waste form can result from the action of frost on freshly solidified materials. Guidance on using concrete in cold weather (American Concrete Institute, 1994) can be used to help mitigate the effects of cold weather on waste form placement.

Whilst temperature does not affect the improvement of plasticity in a lime modified soil, it can adversely affect early age strength development. Sherwood, 1992 stated for lime, a minimum temperature of 7°C is stipulated whereas for cement, temperature is less important but a minimum of 3°C is given.

2.4.2.10 Specific gravity

Specific gravity of a substance denotes the number of times that substance is heavier than water. In simpler words it can be defined as the ratio between the mass of any substance of a definite volume divided by mass of equal volume of water. In case of soils, specific gravity is the number of times the soil solids are heavier than an equal volume of water.

2.5 Classification of Soils

Different soils with similar properties may be classified into groups and sub-groups according to their engineering behaviour. Classification systems provide a common language to concisely express the general characteristics of soils, which are infinitely varied, without detailed descriptions. Currently two elaborate classification systems are commonly used by soils engineers. Both systems take into consideration the particle-size distribution and Atterberg limits. They are the American Association of State Highway and Transportation Officials (AASHTO) classification system and the Unified Soil Classification System (USCS). AASHTO classification system is used mostly by state and county highway departments, geotechnical engineers generally prefer the Unified system.

2.5.1 The American Association of State Highway and Transportation Officials (AASHTO) System

The AASHTO system of soil classification was developed in 1929 as the Public Road administration classification system. It has undergone several revisions, with the present version proposed by the Committee on Classification of Materials for Subgrades and Granular Type Roads of the Highway Research Board in 1945(ASTM designation D - 3282 AASHTO method M14.5) .

Figure 2.2 shows the relationship between liquid limit and plasticity index for silt-clay groups, the AASHTO classification in present use is given in Table2.3 according to this system soil is classified into seven major groups: A -l through A-7. Soils classified under groups A-1, A-2 and A-3 are granular materials of which 35% or less of the particles pass through the No. 200 sieve. Soils of which more than 35% pass through the No. 200 sieve are classified under groups A-4, A-5, A-6, and A-7. These soils are mostly silt and clay-type materials. The classification system is based on the following criteria:

l. Grain size

a. Gravel: fraction passing the 7-5-mm (3-in.) sieve and retained on the No. l0 (2-mm) sieve

b. Sand: fraction passing the No.10 (2-mm) sieve and retained on the No.200 (0.07mm) sieve

c. Silt and clay: fraction passing the No. 200 sieve

2. Plasticity: The term silty is applied when the fine fractions of the soil have a plasticity index of 10 or less. The term clayey is applied when the fine fractions have a plasticity index of 11 or more.

3. If cobbles and boulders (size larger than 75 mm) are encountered, they are excluded from the portion of the soil sample from which classification is made. However, the percentage of such material is recorded.

Table 2.3: Classification of soils and soil-aggregate mixtures (AASHTO M 145-91).

CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES											
General Classification	Granular Materials (35% or less passing 75µm) [No. 200]							Silt-Clay Materials (More than 35% passing 75µm) [No. 200]			
Group Classification	A-1		A-3*	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis:											
Percent passing:											
2mm (No. 10)	50 max.	---	---	---	---	---	---	---		---	---
425µm (No. 40)	30 max.	50 max.	51 min.	---	---	---	---	---	---	---	---
75µm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 425µm (No. 40):											
Liquid Limit	---		---	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity Index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min**
Usual Types of Significant Constituent Materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General Rating as Subgrade	Excellent to Good							Fair to Poor			

The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate the superiority of A-3 over A-2.

**The plasticity index of A-7-5 is equal to or less than the liquid limit minus 30. The plasticity index of the A-7-6 subgroup is greater than the liquid limit minus 30.

There are three broad types under which the AASHTO groups and subgroups are divided. These are "granular" (A-1, A-3, and A-2), "silt-clay" (A-4 through A-7), and highly organic (A-8) materials. The transitional group, A-2, includes soils which exhibit the characteristics of both granular and silt-clay soils, making subdivision of the group necessary for adequate identification of material properties.

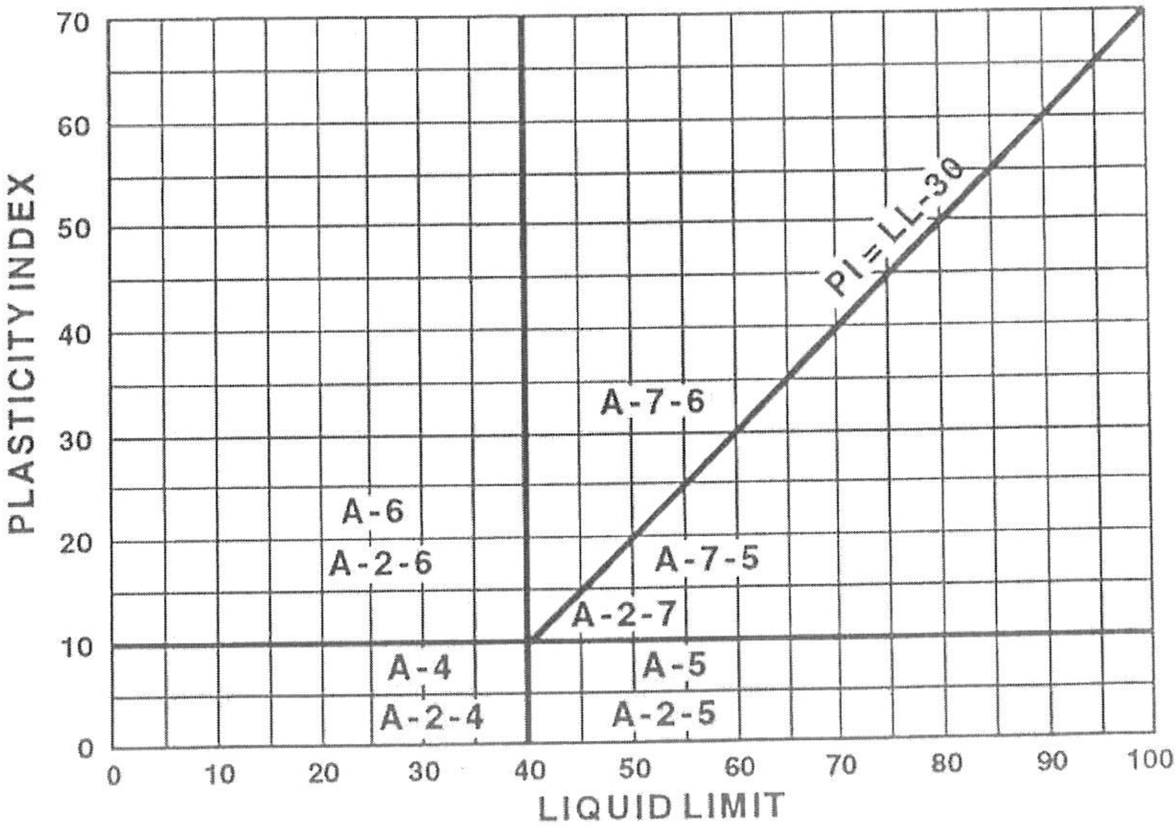


Figure 2.2: Relationship between liquid limit and plasticity index for silt-clay groups (AASHTO M 145-91).

2.5.2 Unified Soil Classification System

Another classification system used widely throughout the engineering community is the Unified Soil Classification System (USCS). The present system, modified by the U.S. Army Corps of Engineers and the Bureau of Reclamation, was introduced during World War II by Casagrande of Harvard University to assist engineers in the design and construction of airfields. As with the AASHTO system, the USCS utilizes grain-size distribution and plasticity characteristics to classify soils. The USCS, however, categorizes soils into one of 15 major soil groups that additionally account for the shape of the grain-size distribution curve.

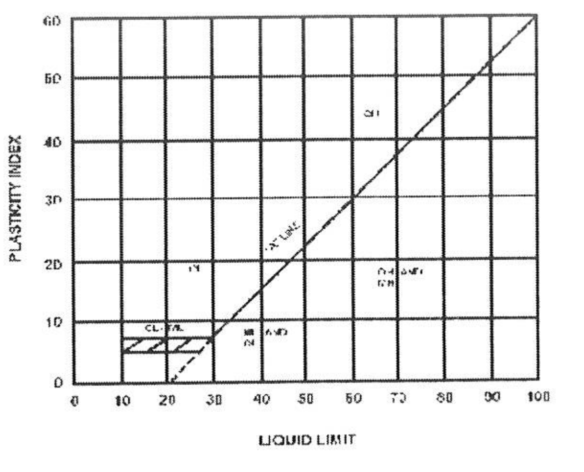
Table 2.4 shows the USCS classification system along with the criteria utilized for associating the group symbol, such as "CL," with the soil. In this chart, D_{60} refers to the diameter of the soil particles that 60 percent of the sample would pass on a sieve, as indicated on the gradation curve. Similarly, D_{10} relates to the maximum diameter of the smallest 10 percent, by weight.

Table 2.4: Unified Soil Classification System chart (after U.S. Army Corps of Engineers, Waterways Experiment Station, TM 3-357, 1953).

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN HALF OF COARSE FRACTION IS GREATER THAN NO. 4 SIEVE SIZE)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	DETERMINE PERCENTAGES OF SAND AND GRAVEL FROM GRAIN-SIZE CURVE, DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN NO. 200 SIEVE), COARSE-GRAINED SOILS ARE CLASSIFIED AS FOLLOWS: LESS THAN 5 PERCENT - GW, GP, SW, SP; MORE THAN 12 PERCENT - GM, GC, SM, SC; 5 TO 12 PERCENT - BORDERLINE CASES REQUIRING DUAL SYMBOLS (b)	$Cu = \frac{D60}{D10}$ GREATER THAN 4; $Cc = \frac{(D30)^2}{D10 * D60}$ BETWEEN 1 AND 3	
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR GW	
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM (a) d / u	SILTY GRAVELS, GRAVEL-SAND MIXTURES		ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	ABOVE "A" LINE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW "A" LINE WITH P.I. GREATER THAN 7	
	SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE)	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		$Cu = \frac{D60}{D10}$ GREATER THAN 6; $Cc = \frac{(D20)^2}{D10 * D60}$ BETWEEN 1 AND 3	
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES		NOT MEETING ALL GRADATION REQUIREMENTS FOR SW	
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM (a) d / u	SILTY SAND, SAND-SILT MIXTURES		ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4	LIMITS PLOTTING IN HATCHED ZONE WITH P.I. BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			SC	CLAYEY SAND, SAND-CLAY MIXTURES		ATTERBERG LIMITS BELOW "A" LINE P.I. GREATER THAN 7	
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE)	SILTS AND CLAYS (LIQUID LIMIT LESS THAN 50)		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY	PLASTICITY CHART		
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS			
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
	SILTS AND CLAYS (LIQUID LIMIT GREATER THAN 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS			
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS			
	HIGHLY ORGANIC SOILS		Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS			

PLASTICITY CHART



a) Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.
b) Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

The plasticity chart shown in the lower right-hand portion of Table 2.4.2 is a graphical representation of the USCS based solely on the plastic and liquid limits (Section 4-2.06.02) of the material passing the 0.425mm (No. 40) sieve. Clays will plot above the "A-line" and silts below. The chart further divides the clays and silts into low (less than 50) and high liquid limits.

2.5.3 Correlation of the Classification Systems

The AASHTO and USCS classification systems are attempts to associate pertinent engineering properties with identifiable soil groupings. However, each system defines soil groups in a slightly different manner. For example, AASHTO classification systems distinguish gravel from sand at the 2.0 millimetres (No. 10) sieve, whereas the USCS uses a break at the 4.76 millimeters (No. 4) sieve. The same coarse-grained soil could, therefore, have different percentages of gravel and sand in the USCS classification systems.

CHAPTER THREE

METHODOLOGY

3.1 Preamble

The practice of testing soil samples in the geotechnical laboratory plays important role in soil mechanics and civil engineering practices. This is because the performance and durability of soil for any use is basically hinged on the strength characteristics of such soil. Therefore, evaluation of materials by various geotechnical tests to determine their suitability is highly essential. This will ensure a satisfactorily performance when put into service for use

3.2 Sampling of Materials

In order to carry out the geotechnical examination work, a borehole was sunk at the locations chosen for collection of soil sample, some of the processes of sample taking are shown in Plate 3.1 and Plate 3.2

Disturbed soil samples were collected below the formation level of about 1.0 metre depth below the existing ground level and the overlying soil material as well as the top soil was discarded. The soil samples were contained in covered and labelled sacks and taken to the laboratory for tests. The coordinates of the location are given below;

Table 3.1: Coordinates of Locations

S/N	TRIAL PIT	COORDINATE IN DEGREE		COORDINATE IN METRIC(m)	
		NORTHING	EASTING	NORTHING	EASTING
1	Ijesa Isu-Ikole Road	7.764809°	5.509817°	862887.69	612294.94
2	Ikole Centre town	7.793713°	5.492262°	866099.74	610344.09
3	Omuo-Ikole Road	7.804725°	5.641358°	867323.48	626912.83
4	Oye-Ikole Road	7.796448°	5.479679°	866403.67	608945.77

Plate 3.1: Samples being taken at the given location

Plate 3.2: Measuring the trial pit with measuring tape

3.3 Methods

3.3.1 Particle Size Distribution

This test is done to determine the particle size distribution of a soil sample

3.3.1.1 Tools

i) A set of fine IS Sieves of sizes – 2mm, 600µm, 425µm, 212µm and 75µm

ii) A set of coarse IS Sieves of sizes – 20mm, 10mm and 4.75mm

iii) Weighing balance, with an accuracy of 0.1% of the weight of sample

iv) Oven

v) Mechanical shaker

vi) Mortar with rubber pestle

vii) Brushes

viii) Trays

3.3.1.2 Preparation Of Sample

i) Soil sample, as received from the field, was dried in air or in the sun. In wet weather, the drying apparatus was used in which case the temperature of the sample do not exceed 60°C. Tree roots and pieces of bark were removed from the sample.

ii) The big clods were broken with the help of wooden mallet. Care was taken not to break the individual soil particles.

iii) A representative soil sample of required quantity as given in Table 3.2 was taken and dried in the oven at 105 to 110°C.

Table 3.2: How to determine wight to be taken for test

Maximum size of material present in substantial quantities (mm)	Weight to be taken for test (kg)
75	60
40	25
25	13
19	6.5
12.5	3.5
10	1.5
6.5	0.75
4.75	0.4

Plate 3.3: Mechanical Sieve Shaker

3.3.1.3 Procedure to determine Particle Size Distribution of Soil

i) The dried sample was taken in a tray, soaked in water and mixed with either 2g of sodium hexametaphosphate and 1g of sodium carbonate per litre of water, which was added as a dispersive agent. The soaking of soil was continued for 10 to 12hrs

ii) The sample was washed through 4.75mm IS Sieve with water till substantially clean water comes out. Retained sample on 4.75mm IS Sieve was oven-dried for 24hrs. This dried sample was sieved through 20mm and 10mm IS Sieves.

iii) The portion passing through 4.75mm IS Sieve was oven-dried for 24hrs. This oven-dried material was riffled and about 200g taken.

iv)The sample of about 200g was washed through 75µm IS Sieve with half litre distilled water, till substantially clear water comes out.

v) The material retained on 75µm IS Sieve was collected and dried in oven at a temperature of 105 to 120°C for 24hrs. The dried soil sample was sieved through 2mm, 600µm,425µm and 212µm IS Sieves. Soil retained on each sieve was weighed.

vi) If the soil passing 75µm is 10% or more, hydrometer method was used to analyse soil particle size.

3.3.1.4 Reporting of Results

The results were plotted on a semi-log graph with particle size as abscissa (log scale) and the percentage smaller than the specified diameter as ordinate

3.3.2 Specific Gravity

This test is done to determine the specific_gravity of fine-grained soil by density bottle. Specific gravity is the ratio of the weight in air of a given volume of a material at a standard temperature to the weight in air of an equal volume of distilled water at the same stated temperature.

3.3.2.1 Tools

i) Two density bottles of approximately 50ml capacity along with stoppers

ii) Constant temperature water bath (27.0 + 0.2°C)

iii) Vacuum desiccator

iv) Oven, capable of maintaining a temperature of 105 to 110°C

v) Weighing balance, with an accuracy of 0.001g

vi) Spatula

3.3.2.2 Preparation of Sample

Soil sample (50g) was grounded to pass through a 2mm IS Sieve. A 5 to 10g sub-sample should be obtained by riffling and oven-dried at a temperature of 105 to 110°C.

3.3.2.3 Procedure to Determine the Specific Gravity of Fine-Grained Soil

i) The density bottle along with the stopper was dried at a temperature of 105 to 110°C, cooled in the desiccator and weighed to the nearest 0.001g (W_1).

ii) The sub-sample, which had been over-dried was transferred to the density bottle directly from the desiccator in which it was cooled. The bottles and contents together with the stopper were weighed to the nearest 0.001g (W_2).

iii) The soil was covered with air-free distilled water from the glass wash bottle and left for a period of 2 to 3hrs. for soaking. Water was added to fill the bottle to about half.

iv) Entrapped air was removed by heating the density bottle on a water bath or a sand bath.

v) The bottle was kept without the stopper in a vacuum desiccator for about 1 to 2hrs. until there was no further loss of air.

vi) The soil was gently stirred in the density bottle with a clean glass rod, the adhering particles was carefully washed off from the rod with some drops of distilled water ensuring that no more soil particles were lost.

vii) The process was repeated till no more air bubbles are observed in the soil-water mixture.

viii) The constant temperature in the bottle was observed and recorded.

ix) The stopper was inserted in the density bottle, wiped and weighed (W_3).

x) The bottle was then emptied, thoroughly cleaned and filled with distilled water at the same temperature. The stopper was inserted in the bottle, wiped dry from the outside and weighed (W_4).

xi) Two of such observations were taken for the same soil.

3.3.2.4 Reporting of Results

The specific gravity G of the soil = $(W_2 - W_1) / [(W_{4-1})-(W_3-W_2)]$. The specific gravity was calculated at a temperature of 27°C and reported to the nearest 0.01. If the room temperature is different from 27°C, the following corrections were done:-

G' = Kg where,

G' = Corrected specific gravity at 27°C

k = [Relative density of water at room temperature]/ Relative density of water at 27°C.

3.3.3 Compaction Test

This test is done to determine the maximum dry density and the optimum moisture content of soil. There are three (3) methods used for compaction, they include

1. Standard Proctor test
2. Modified AASHTO method
3. West Africa method

i) Cylindrical metal mould – it should be either of 100mm dia. and 1000cc volume or 150mm dia. and 2250cc volume, shown in Plate 3.4.

ii) Balances – one of 10kg capacity, sensitive to 1g and the other of 200g capacity, sensitive to 0.01g

iii) Oven – thermostatically controlled with an interior of non-corroding material to maintain temperature between 105 and 110°C

iv) Steel straightedge – 30cm long

v) IS Sieves of sizes – 4.75mm, 19mm and 37.5mm

3.3.3.1 Preparation of Sample

A representative portion of air-dried soil material, large enough to provide about 6kg of material passing through a 19mm IS Sieve (for soils not susceptible to crushing during compaction) or about 15kg of material passing through a 19mm IS Sieve (for soils susceptible to crushing during compaction), was taken. This portion was sieved through a 19mm IS Sieve and the coarse fraction rejected after its proportion of the total sample has been recorded. Aggregations of particles was broken down so that if the sample was sieved through a 4.75mm IS Sieve, only separated individual particles would be retained.

3.3.3.2 Procedure to Determine the Maximum Dry Density and the Optimum Moisture Content of Soil

A) Soil not susceptible to crushing during compaction –

i) A 5kg sample of air-dried soil passed through the 19mm IS Sieve was taken. The sample was mixed thoroughly with a suitable amount of water depending on the soil type (for sandy and gravelly soil – 3 to 5% and for cohesive soil – 12 to 16% below the plastic limit). The soil sample was stored in a sealed container for a minimum period of 16hrs.

ii) The mould of 1000cc capacity with base plate attached, was weighed to the nearest 1g (W_1). The mould was placed on a solid base, such as a concrete floor or plinth and the moist soil was compacted into the mould, with the extension attached, in five layers of approximately equal mass, each layer being given 25 blows from the 4.9kg rammer dropped from a height of 450mm above the soil. The blows should be distributed uniformly over the surface of each layer. The amount of soil used was sufficient to fill the mould,leaving not more than about 6mm to be struck off when the extension is removed. The extension was removed and the compacted soil was leveled off carefully to the top of the mould by means of the straight edge. The mould and soil was then weighed to the nearest gram (W_2).

iii) The compacted soil specimen was removed from the mould and placed onto the mixing tray. The water content (w) of a representative sample of the specimen was determined.

iv)The remaining soil specimen was broken up, rubbed through 19mm IS Sieve and then mixed with the remaining original sample. Suitable increments of water was added successively and mixed into the sample, and the above operations i.e. ii) to iv) was repeated for each increment of water added. The total number of determinations made was at least five and the moisture content was such that the optimum moisture content at which the maximum dry density occurs, lies within that range.

B) Soil susceptible to crushing during compaction–

Five or more 2.5kg samples of air-dried soil passing through the 19mm IS Sieve, was taken. The samples each were mixed thoroughly with different amounts of water and stored in a sealed container as mentioned in Part A)

C) Compaction in large size mould –

For compacting soil containing coarse material up to 37.5mm size, the 2250cc mould was used. A sample weighing about 30kg and passing through the 37.5mm IS Sieve is used for the test. Soil is compacted in five layers, each layer being given 55 blows of the 4.9kg rammer. The rest of the procedure is same as above.

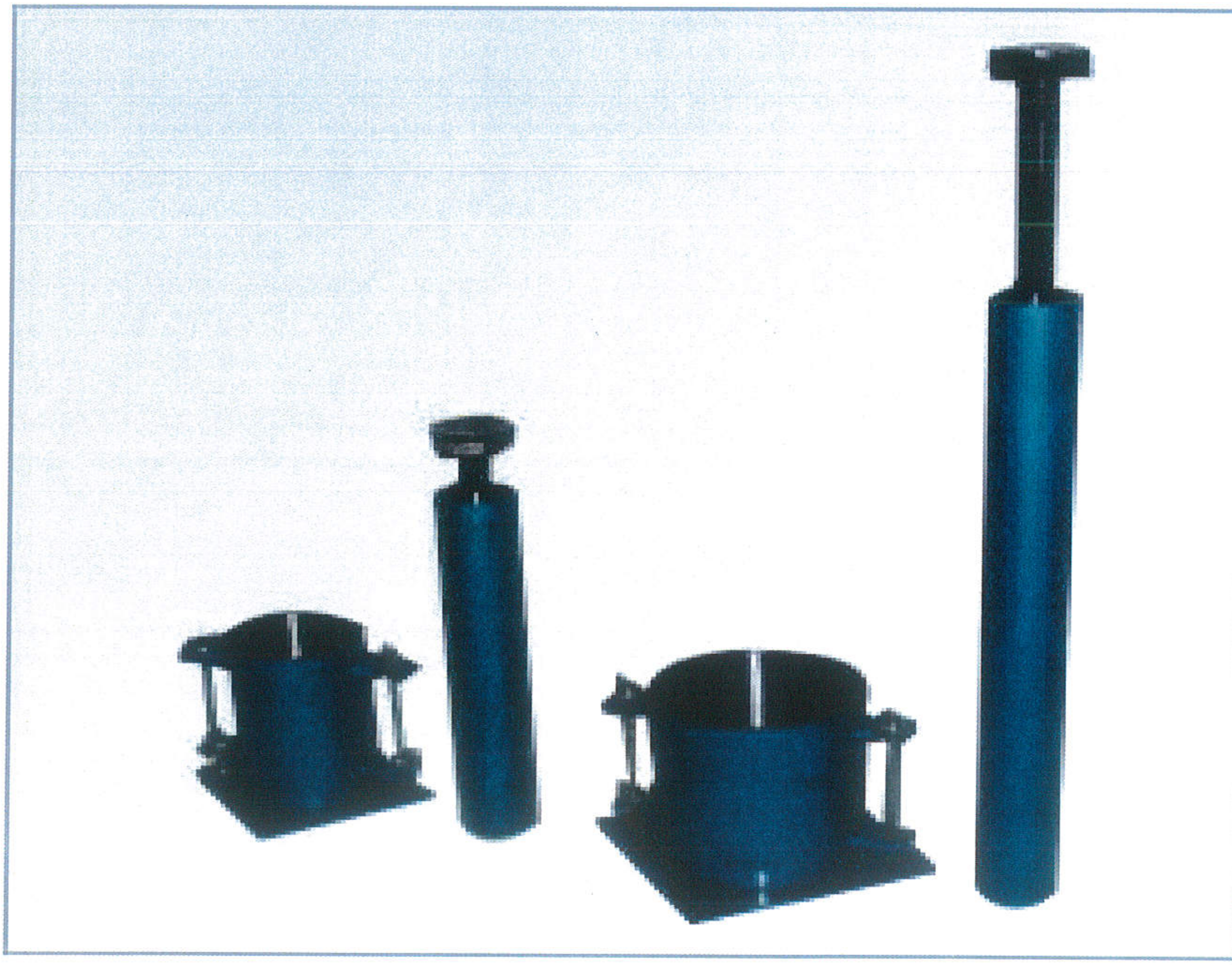


Plate 3.4: Moulds and Rammers

3.3.3.3 Reporting of Results

Bulk density Y(gamma) in g/cc of each compacted specimen was calculated from the equation,

Y(gamma) = $(W_2-W_1)/V$

where, V = volume in cc of the mould.

The dry density Yd in g/cc

Yd = 100Y/(100+w)

The dry densities, Yd obtained in a series of determinations were plotted against the corresponding moisture contents, w. A smooth curve was drawn through the resulting points and the position of the maximum on the curve was determined. The dry density in g/cc corresponding to the maximum point on the moisture content/dry density curve was reported as the maximum dry density to the nearest 0.01. The percentage moisture content corresponding to the maximum dry density on the moisture content/dry density curve was reported as the optimum moisture content and was quoted to the nearest 0.2 for values below 5 percent, to the nearest 0.5 for values from 5 to 10 percent and to the nearest whole number for values exceeding 10 percent.

This test was done to determine the in-situ dry density of soil by sand replacement method as per IS: 2720 (Part XXVIII) – 1974. The apparatus needed were

i) Sand-pouring cylinder conforming to IS: 2720 (Part XXVIII) -1974

ii) Cylindrical calibrating container conforming to IS: 2720 (Part XXVIII) – 1974

iii) Soil cutting and excavating tools such as a scraper tool, bent spoon

iv) Glass plate – 450mm square and 9mm thick or larger

v) Metal containers to collect excavated soil

vi) Metal tray – 300mm square and 40mm deep with a 100mm hole in the centre

vii) Balance, with accuracy of 1g

3.3.3.4 Measurement of soil density

The following method was followed for the measurement of soil density:

i) A flat area, approximately 450sq.mm of the soil to be tested was exposed and trimmed down to a level surface, preferably with the aid of the scraper tool.

ii) The metal tray with a central hole was laid on the prepared surface of the soil with the hole over the portion of the soil to be tested. The hole in the soil was then excavated using the hole in the tray as a pattern, to the depth of the layer to be tested upto a maximum of 150mm. The excavated soil was carefully collected, leaving no loose material in the hole and weighed to the nearest gram(W_w). The metal tray was removed before the pouring cylinder is placed in position over the excavated hole.

iii) The water content (w) of the excavated soil was determined as discussed in earlier posts. Alternatively, the whole of the excavated soil should be dried and weighed (W_d).

iv) The pouring cylinder, filled to the constant weight (W_1) was so placed that the base of the cylinder covers the hole concentrically. The shutter was then opened and sand allowed to runout into the hole. The pouring cylinder and the surrounding area was not vibrated during this period. When no further movement of sand takes place, the shutter

was closed. The cylinder was removed and weighed to the nearest gram (W_4).

3.3.3.5 Reporting of Results

The following values were reported:

i) dry density of soil in kg/m3 to the nearest whole number; also to be calculated and reported in g/cc correct to the second place of decimal

ii) water content of the soil in percent reported to two significant figures.

3.3.4 Plastic Limit Test

This test is done to determine the plastic limit of soil as per IS: 2720 (Part 5) – 1985.The plastic limit of fine-grained soil is the water content of the soil below which it ceases to be plastic. It begins to crumble when rolled into threads of 3mm dia.

3.3.4.1 Tools

i) Porcelain evaporating dish about 120mm dia.

ii) Spatula

iii) Container to determine moisture content

iv) Balance, with an accuracy of 0.01g

v) Oven

vi) Ground glass plate – 20cm x 15cm

vii) Rod – 3mm dia. and about 10cm long

3.3.4.2 Preparation of Sample

Take out 30g of air-dried soil from a thoroughly mixed sample of the soil passing through 425µm IS Sieve. Mix the soil with distilled water in an evaporating dish and leave the soil mass for naturing. This period may be up to 24hrs.

3.3.4.3 Procedure to determine the Plastic Limit of Soil

i) 8g of the soil was taken and rolled with fingers on a glass plate. The rate of rolling was between 80 to 90 strokes per minute to form a 3mm dia.

ii) The threads was reduced to less than 3mm, without any cracks appearing, this means that the water content is more than its plastic limit. The soil was kneaded to reduce the water content and rolled into a thread again.

iii) Repeat the process of alternate rolling and kneading until the thread crumbles.

iv) Pieces of crumbled soil thread was collected and kept in the container used to determine the moisture content.

v) The process was repeated twice more with fresh samples of plastic soil each time.

3.3.4.4 Reporting of Results

The plastic limit was determined for at least three portions of the soil passing through 425µm IS Sieve. The average water content to the nearest whole number was reported.

3.3.5 Liquid Limit Test

This test is done to determine the liquid limit of soil as per IS: 2720 (Part 5) – 1985. The liquid limit of fine-grained soil is the water content at which soil behaves practically like a liquid, but has small shear strength. Its flow closes the groove in just 25 blows in Casagrande's liquid limit device.

3.3.5.1 Tools

i) Casagrande's liquid limit device

ii)Grooving tools of both standard and ASTM types

iii)Oven

iv) Evaporating dish

v) Spatula

vi) IS Sieve of size 425μm

vii) Weighing balance, with 0.01g accuracy

viii) Wash bottle

ix) Air-tight and non-corrodible container for determination of moisture content

3.3.5.2 Preparation of Sample

i) Air-dry the soil sample and break the clods. Remove the organic matter like tree roots, pieces of bark, etc.

ii) About 100g of the specimen passing through 425μm IS Sieve is mixed thoroughly with distilled water in the evaporating dish and left for 24hrs. for soaking.

3.3.5.3 Procedure to Determine the Liquid Limit of soil

i) Place a portion of the paste in the cup of the liquid limit device.

ii) Level the mix so as to have a maximum depth of 1cm.

iii) Draw the grooving tool through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.

iv) For normal fine grained soil: The Casagrande's tool was used to cut a groove 2mm wide at the bottom, 11mm wide at the top and 8mm deep.

v) For sandy soil: The ASTM tool was used to cut a groove 2mm wide at the bottom, 13.6mm wide at the top and 10mm deep.

vi) After the soil pat has been cut by a proper grooving tool, the handle was rotated at the rate of about 2 revolutions per second and the no. of blows counted, till the two parts of the soil sample come into contact for about 10mm length.

vii) Take about 10g of soil near the closed groove and determine its water content

viii) The soil of the cup was transferred to the dish containing the soil paste and mixed thoroughly after adding a little more water. Repeat the test.

ix) By altering the water content of the soil and repeating the foregoing operations, obtain at least 5 readings in the range of 15 to 35 blows. Don't mix dry soil to change its consistency.

x) Liquid limit is determined by plotting a 'flow curve' on a semi-log graph, with no. of blows as abscissa (log scale) and the water content as ordinate and drawing the best straight line through the plotted points.

3.3.5.4 Reporting of Results

Report the water content corresponding to 25 blows, read from the 'flow curve' as the liquid limit.

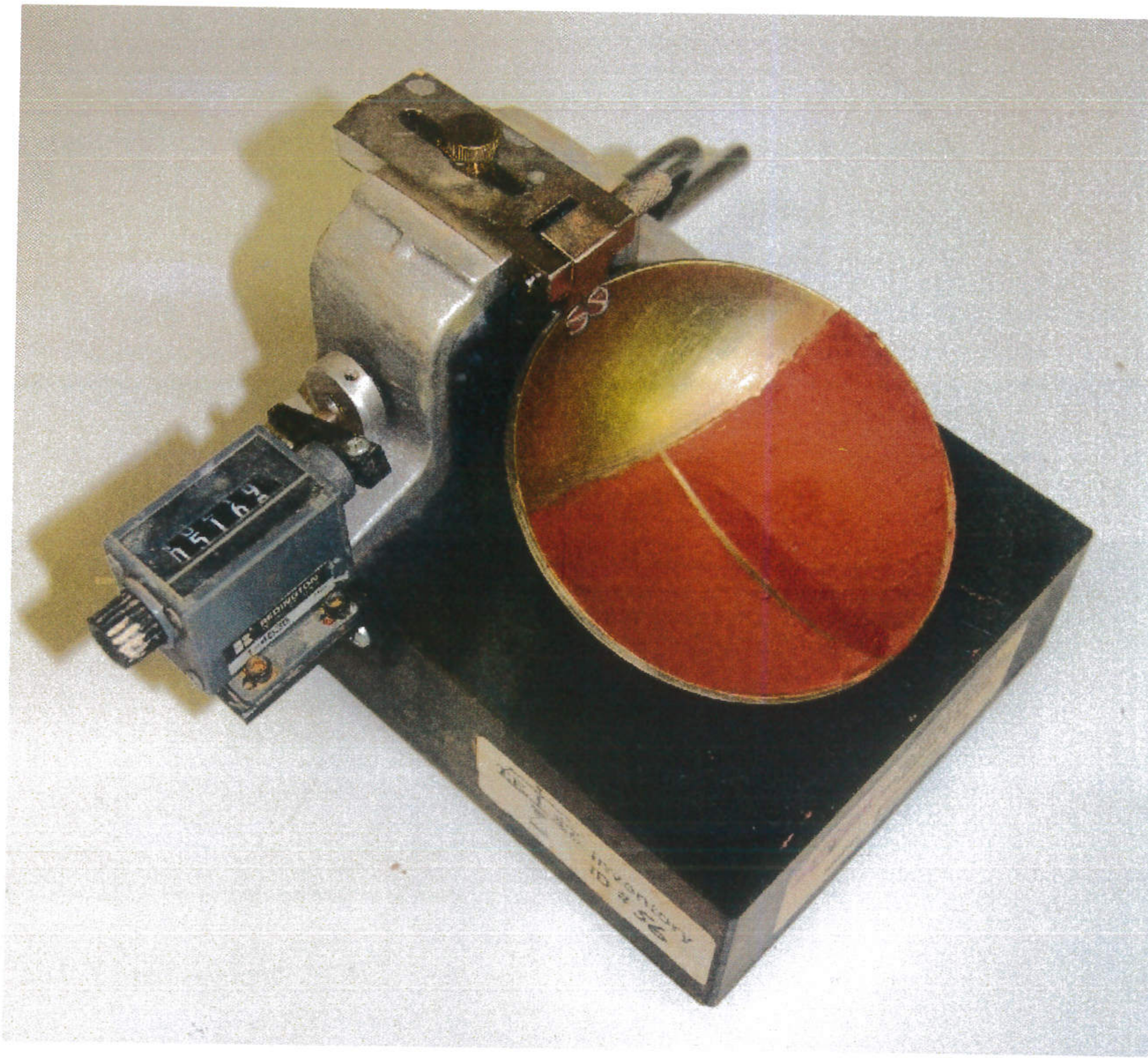


Plate 3.5: Liquid Limit apparatus

3.3.6 California Bearing Ratio Test

It is the ratio of force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25 mm/min. to that required for the corresponding penetration of a standard material. The California Bearing Ratio Test (CBR Test) is a penetration test developed by California State Highway Department (U.S.A.) for evaluating the bearing capacity of subgrade soil for design of flexible pavement.

Tests are carried out on natural or compacted soils in water soaked or un-soaked conditions and the results so obtained are compared with the curves of standard test to have an idea of the soil strength of the subgrade soil.

3.3.6.1 Tools

i) Mould
ii) Steel Cutting collar
iii) Spacer Disc
iv) Surcharge weight
v) Dial gauges
vi) IS Sieves
vii) Penetration Plunger
viii) Loading Machine

3.3.6.2 CBR Test Procedure

Normally 3 specimens each of 7 kg was compacted so that their compacted densities range from 95% to 100% generally with 10, 30 and 65 blows.

i) Empty mould was weighed
ii) Water was added to the first specimen (compacted in five layer by giving 10 blows per layer)
iii) After compaction, the collar was removed and the surface levelled.
iv) Sample was taken for determination of moisture content.
v) The weight of mould + compacted specimen was taken.

vi) Other samples were taken and different blows were applied and the whole process repeated.

vii) After four days, the swell reading was measured and percentage swell was determined.

viii) The mould was removed from the tank and water allowed to drain.

ix) Then specimen was then placed under the penetration piston and surcharge load of 10lb was placed.

x) The load was applied and the penetration load values noted.

3.3.6.3 Reporting of Results

Draw the graphs between the penetration (in) and penetration load (in) and find the value of CBR. Draw the graph between the %age CBR and Dry Density, and find CBR at required degree of compaction.

3.3.7 Direct Shear Test

To determine the shearing strength of the soil using the direct shear apparatus.

3.3.7.1 Tools

i) Direct shear box apparatus
ii) Loading frame (motor attached).
iii) Dial gauge.
iv) Proving ring.
v) Tamper.
vi) Straight edge.
vii) Balance to weigh up to 200 mg.
viii) Aluminum container.
ix) Spatula.

3.3.7.2 Procedure

1. Check the inner dimension of the soil container.

2. Put the parts of the soil container together.

3. Calculate the volume of the container. Weigh the container.

4. Place the soil in smooth layers (approximately 10 mm thick). If a dense sample is desired tamp the soil.

5. Weigh the soil container, the difference of these two is the weight of the soil. Calculate the density of the soil.

6. Make the surface of the soil plane.

7. Put the upper grating on stone and loading block on top of soil.

8. Measure the thickness of soil specimen.

9. Apply the desired normal load.

10.Remove the shear pin.

11. Attach the dial gauge which measures the change of volume.

12. Record the initial reading of the dial gauge and calibration values.

13. Before proceeding to test check all adjustments to see that there is no connection between two parts except sand/soil.

14. Start the motor. Take the reading of the shear force and record the reading.

15.Take volume change readings till failure.

16. Add 5 kg normal stress 0.5 kg/cm2 and continue the experiment till failure

17. Record carefully all the readings. Set the dial gauges zero, before starting the experiment

3.3.8 Permeability Test

The knowledge of this property is much useful in solving problems involving yield of water bearing strata, seepage through earthen dams, stability of earthen dams, and embankments of canal bank affected by seepage, settlement etc. For disturbed soil sample.

3.3.8.1 Preparation of sample

i) .A 2.5 kg sample shall be taken from a thoroughly mixed air dried or oven dried material.

ii) The initial moisture content of the 2.5 kg sample shall be determined. Then the soil shall be placed in the air tight container.

iii) Add required quantity of water to get the desired moisture content.

iv) Mix the soil thoroughly.

v) Weigh the empty permeameter mould.

vi) After greasing the inside slightly, clamp it between the compaction base plate and extension collar.

vii) Place the assembly on a solid base and fill it with sample and compact it.

viii) After completion of a compaction the collar and excess soil are removed.

ix) Find the weight of mould with sample.

x) Place the mould with sample in the permeameter, with drainage base and cap having discs that are properly saturated.

3.3.8.2 Procedure

i) .For the constant head arrangement, the specimen shall be connected through the top inlet to the constant head reservoir.

ii) Open the bottom outlet.

iii) Establish steady flow of water.

iv) The quantity of flow for a convenient time interval may be collected.

v) Repeat three times for the same interval.

3.3.8.3 Presentation of data

The coefficient of permeability is reported in cm/sec at 27o C. The dry density, the void ratio and the degree of reported in cm/sec at 27o C. The dry density, the void ratio and the degree of saturation shall be reported. The test results should be tabulated as below:

CHAPTER FOUR

RESULT AND DISCUSSION

The following is the presentation of the results of the previously described laboratory tests conducted. Appropriate graphs are included as necessary for clarity and further details are provided in the appendices.

Laboratory tests were performed on the sample collected from the four locations used as case study. The assessment characteristics such as Atterberg limits, particle size distribution, specific gravity, compaction test, California bearing ratio and natural moisture content test were determined.

4.1 Natural Moisture Content Test

The result of natural moisture content are shown on Tables 4.1.1 and 4.1.2, some of the results were fairly high consistency, the time of the test, indicating the soil potential for the water retention except for one sample having 12.6% as the lowest moisture content.

Average Moisture Content for the samples include;

S/N	Sample Location	Average Moisture Content
1	Ijesa Isu - Ikole Road	16.8
2	Ikole Center Town	12.6
3	Omuo- Ikole Road	17.8
4	Oye – Ikole Road	13.1

4.2 Particle Size Distribution

Table 4.2 shows the summary of the article size distribution. For the area studied soils, the result show that some of have high percentage finer than 0.0075 fraction, that is, > 35% while soil from Oye-Ikole road has finer < 35%. The materials are constituents of clay, sandy, and silt in which the percentage passing number 200 sieve is higher than 35% in accordance to BS 1377 the soil can be classified as clay. The detail of the particle size analysis is in Appendix A. "I consider it essential that an experienced soils engineer should be able to judge the position of soils, from his territory, on a plasticity chart merely on the basis of his visual and manual examination of the soils" (Casagrande, 1959).

Table 4.2 Results of Particle Size Distribution

	Sieve size	Ijesa Isu-Ikole Road %passing	Ikole Center town %passing	Omuo- Ikole Road %passing	Oye- Ikole Road %passing
1	9.50	98.40	95.30	99.20	96.00
2	4.75	88.70	90.90	88.70	85.40
3	2.36	80.00	83.90	79.20	74.50
4	1.18	75.70	73.80	70.10	64.00
5	0.6	69.20	61.50	60.40	51.90
6	0.30	59.30	48.80	51.70	41.10
7	0.15	53.50	38.20	44.70	32.50
8	0.0075	51.00	32.60	39.90	27.20

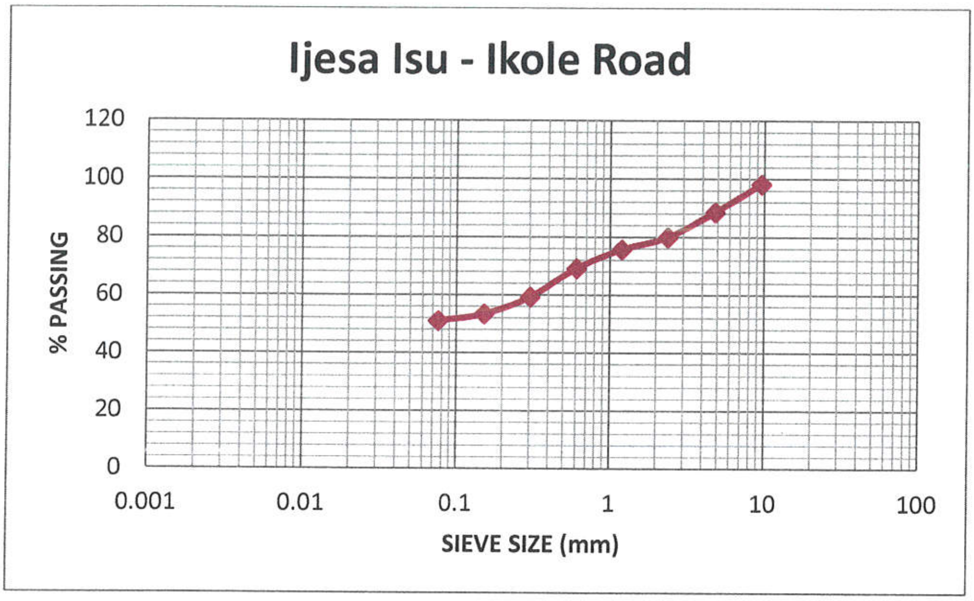


Figure 4.3.1 Graph of sieve size against %passing for Ijesu Isu – Ikole Road

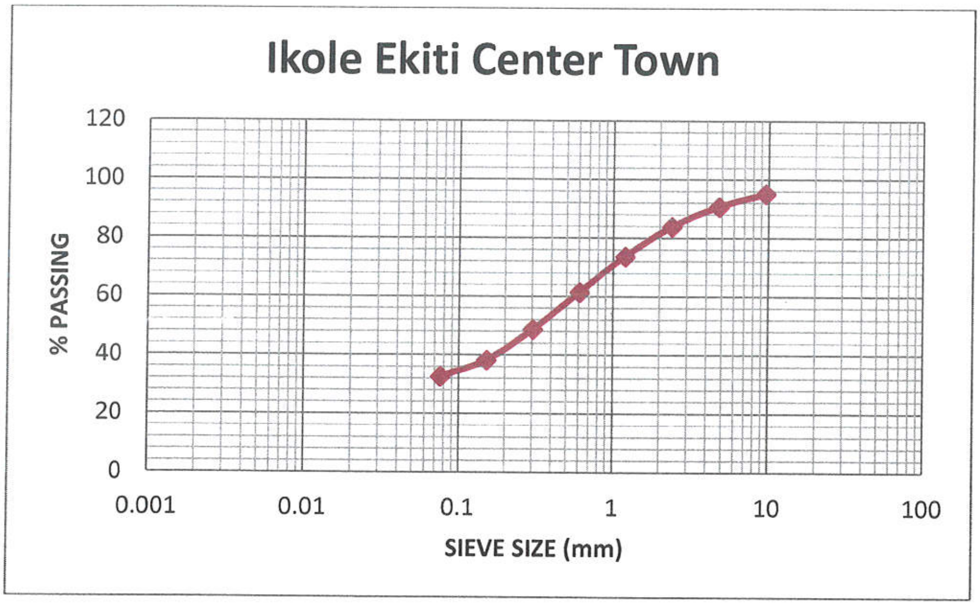


Figure 4.2.2 Graph of sieve size against %passing for Ikole Center Town

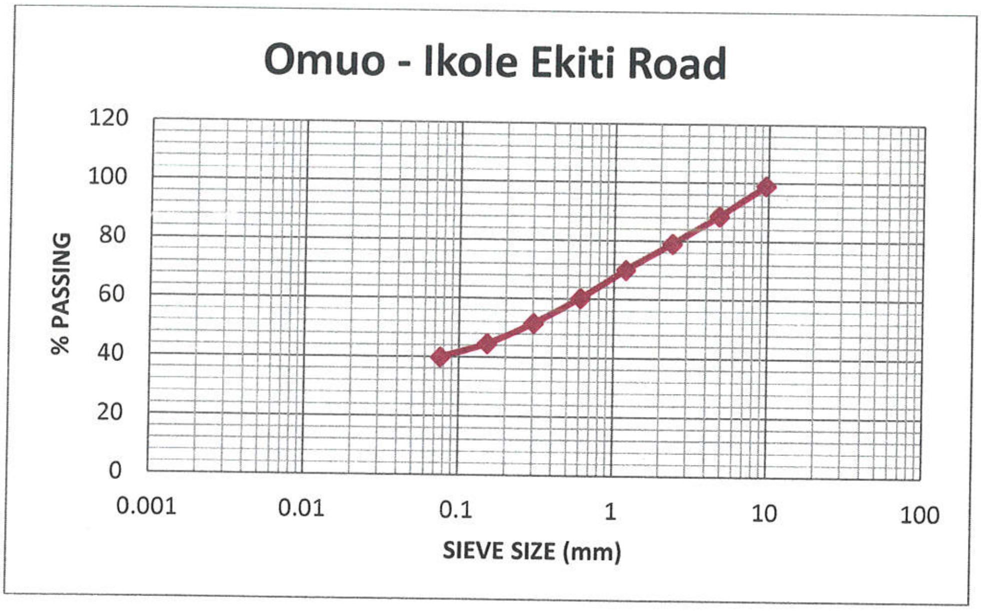


Figure 4.2.3 Graph of sieve size against %passing for 0muo – Ikole Road

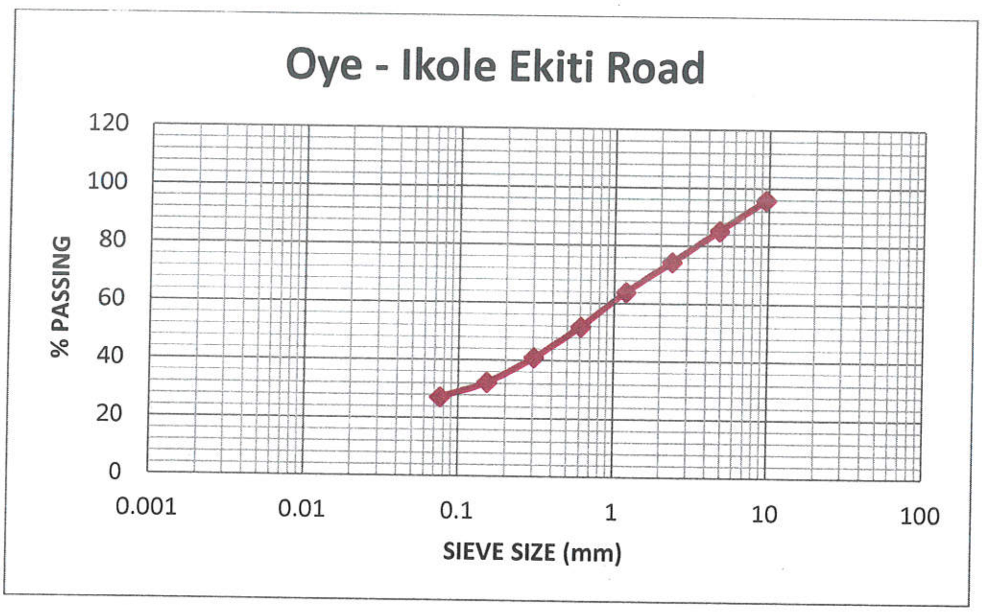


Figure 4.2.4 Graph of sieve size against %passing for Oye – Ikole Road

4.3 The Specific Gravity

The specific gravity of the soil sample was determined according to the procedure stated in chapter three. The result below shows the variation on the result, the calculated average specific gravity was gotten as follows:

Table 4.3: Average Specific Gravity of Samples

S/N	Location of Sample	Average Specific Gravity
1	Ijesa Isu-Ikole Road	2.39
2	Ikole centre town	2.59
3	Omuo-Ikole Road	2.31
4	Oye-Ikole Road	2.31

The result from location varies from – to – the average specific gravity of 2.36 supports the fact that the less clay content

4.4 Compaction Test

From the laboratory, results of the compaction test shows variation on the Optimum Moisture Content and Maximum Dry Density from the different locations, prior to soil California bearing ratio (CBR) determination, compaction was performed on the soil sample using standard Proctor density test (AASHTO T-99) so as to obtain its maximum dry density and the corresponding optimum moisture content. The results of the test are shown on the table below. The optimum moisture content was used for CBR purposes; details of this were provided in the next section. The maximum dry density of the soil is logically a justification of the specific gravity of the soil sample, its organic nature and poor gradation. Specific gravity of organic soils is usually less than 2.00 according to ASTM D 854. Poor graded soils cannot be easily and fully compacted. Figure 4.6.1, 4.6.2, 4.6.3 and 4.6.4 are the graphical presentations of the obtained results.

Table 4.6 shows the results of the compaction tests (maximum density and optimum moisture content) while the graphical illustrations are shown in the Figures 4.4.1, 4.4.2, 4.4.3 and 4.4.4

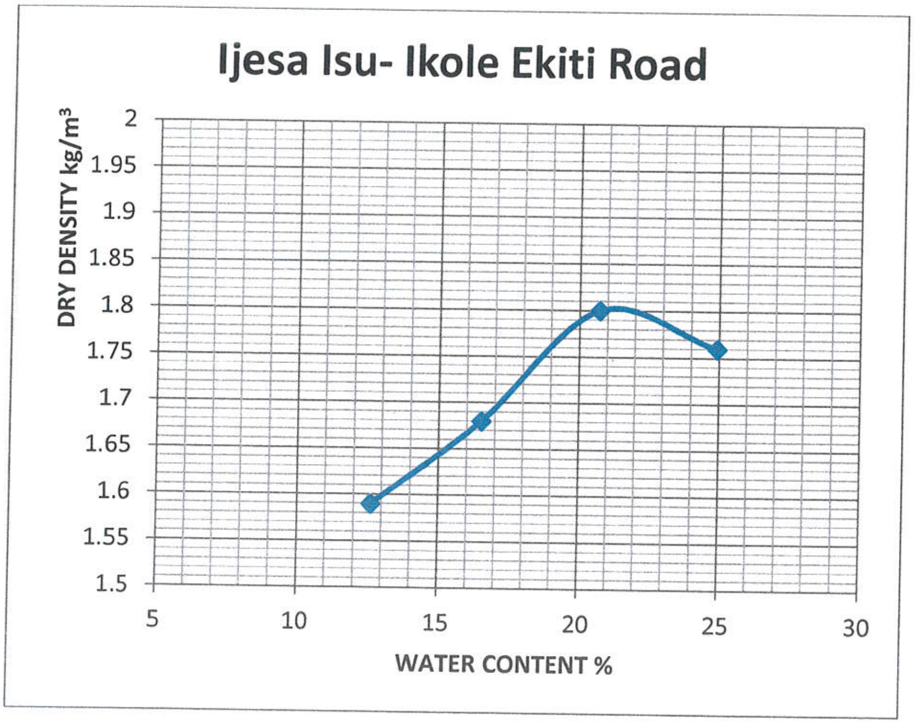


Figure 4.4.1: Graph of Water Content against Dry Density for Ijesa Isu-Ikole Road

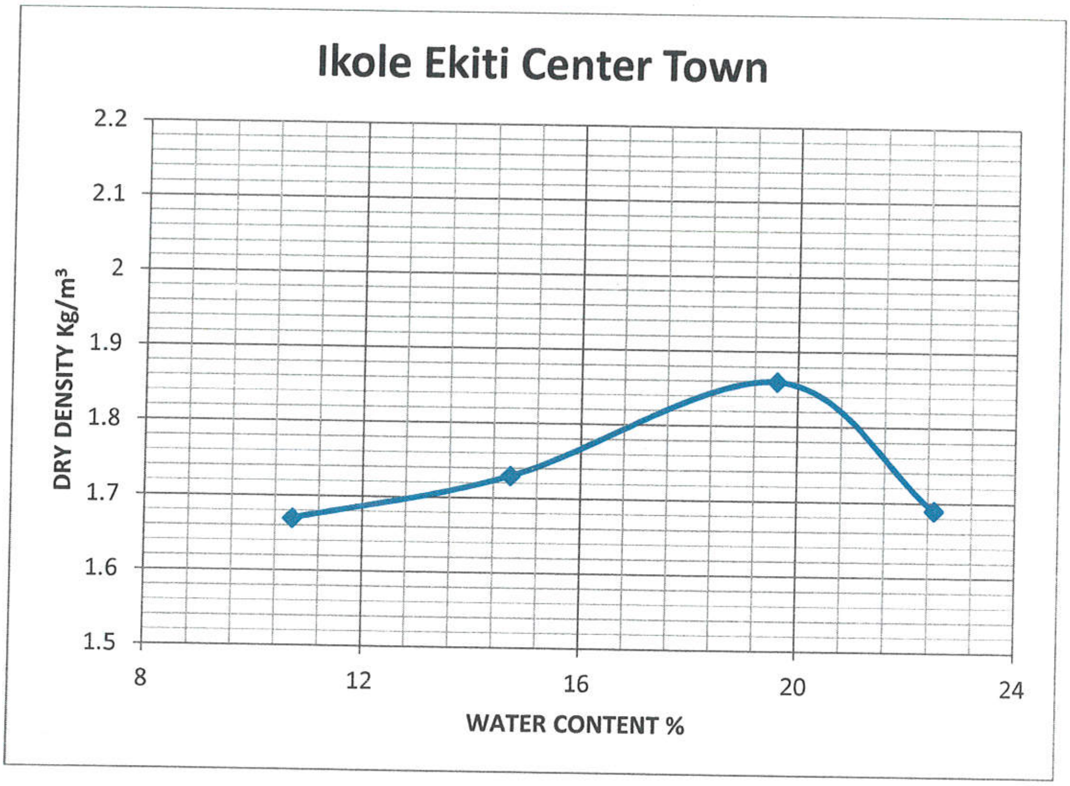


Figure 4.4.2: Graph of Water Content against Dry Density for Ikole Center Town

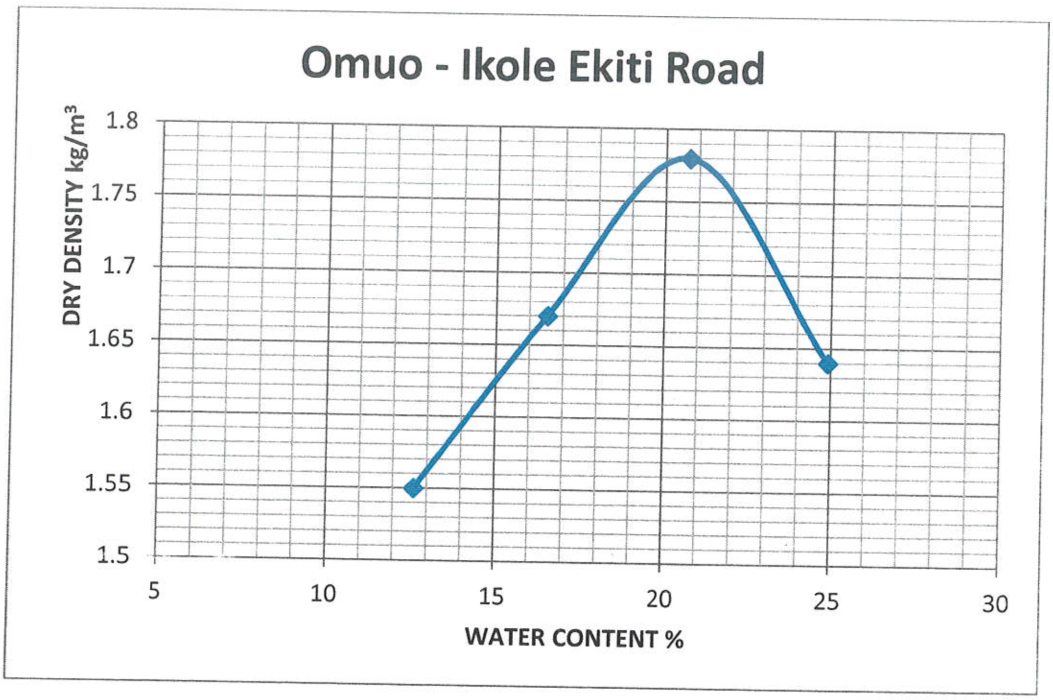


Figure 4.4.3: Graph of Water Content against Dry Density for Omuo-Ikole Road

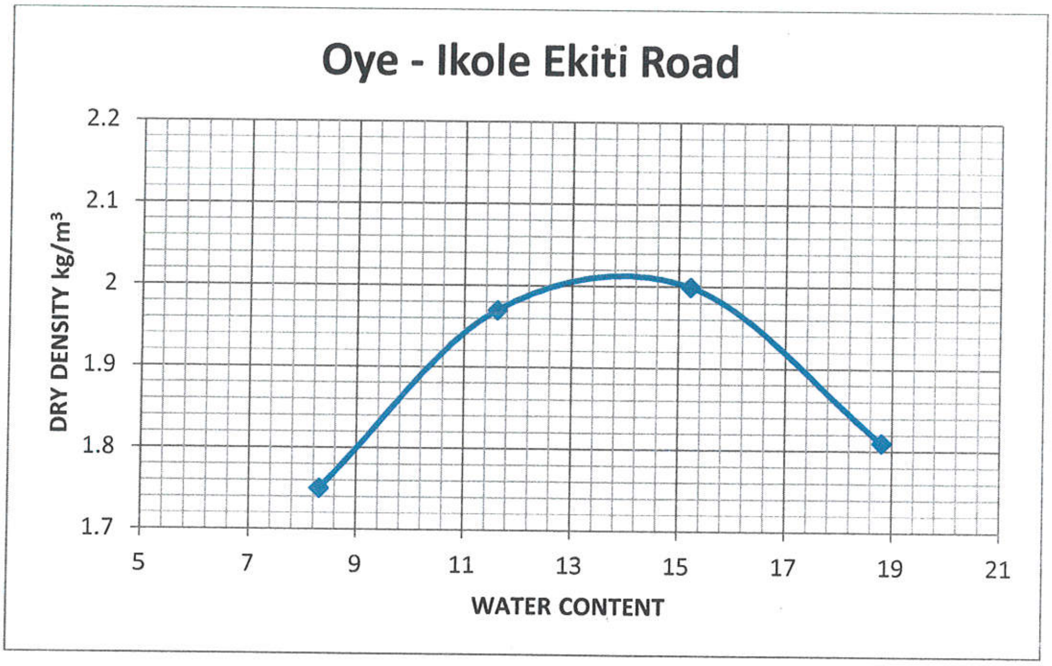


Figure 4.4.4: Graph of Water Content against Dry Density for Oye-Ikole Road

Table 4.4 Results of Compaction Test

S/N	Location	Ijesa Isu- Ikole Road	Ikole Center town	Omuo-Ikole Road	Oye-Ikole Road
1	OMC (%)	19.20	19.60	20.70	14.30
2	MDD(%)	1.76	1.69	1.78	2.05

The result of the test shows that the maximum dry density falls within 2.05kg/m3 and 1.69kg/m3 with the OMC of 20.7% and 15% respectively.

Sample from Oye-Ikole road has the highest MDD and the lowest OMC while soil from Ikole centre town has lower MDD with 19.6% OMC.

4.5 Atterberg Limits

Results of Atterberg limits are shown below in Table 4.4 while the graphs are shown in Figures 4.4.1, 4.4.2, 4.4.3 and 4.4.4, it was observed in all the samples due to the result obtained from the Liquid Limit LL, Plastic Limit PL, and Plasticity index, that the sample can be grouped into A-2-6 AND A-7-5 classification according to AASHTO (1978) and BS 1377(1990). Usually, soil in this class are classified as fair within the range 31.80 and 58.50 for LL while LI were within the range of 0.49 and 6.20 respectively.

The soil sample exhibited a liquid behavior at the liquid limit of 22.2 %, which justifies the classification by sieve analysis above that the soil is predominantly sandy. Clay particles are mostly responsible for plasticity/cohesion, and also silt particles. The graphical presentations of the results of the Atterberg limit tests are shown in Figures 4.5.1, 4.5.2, 4.5.3 and 4.5.4.

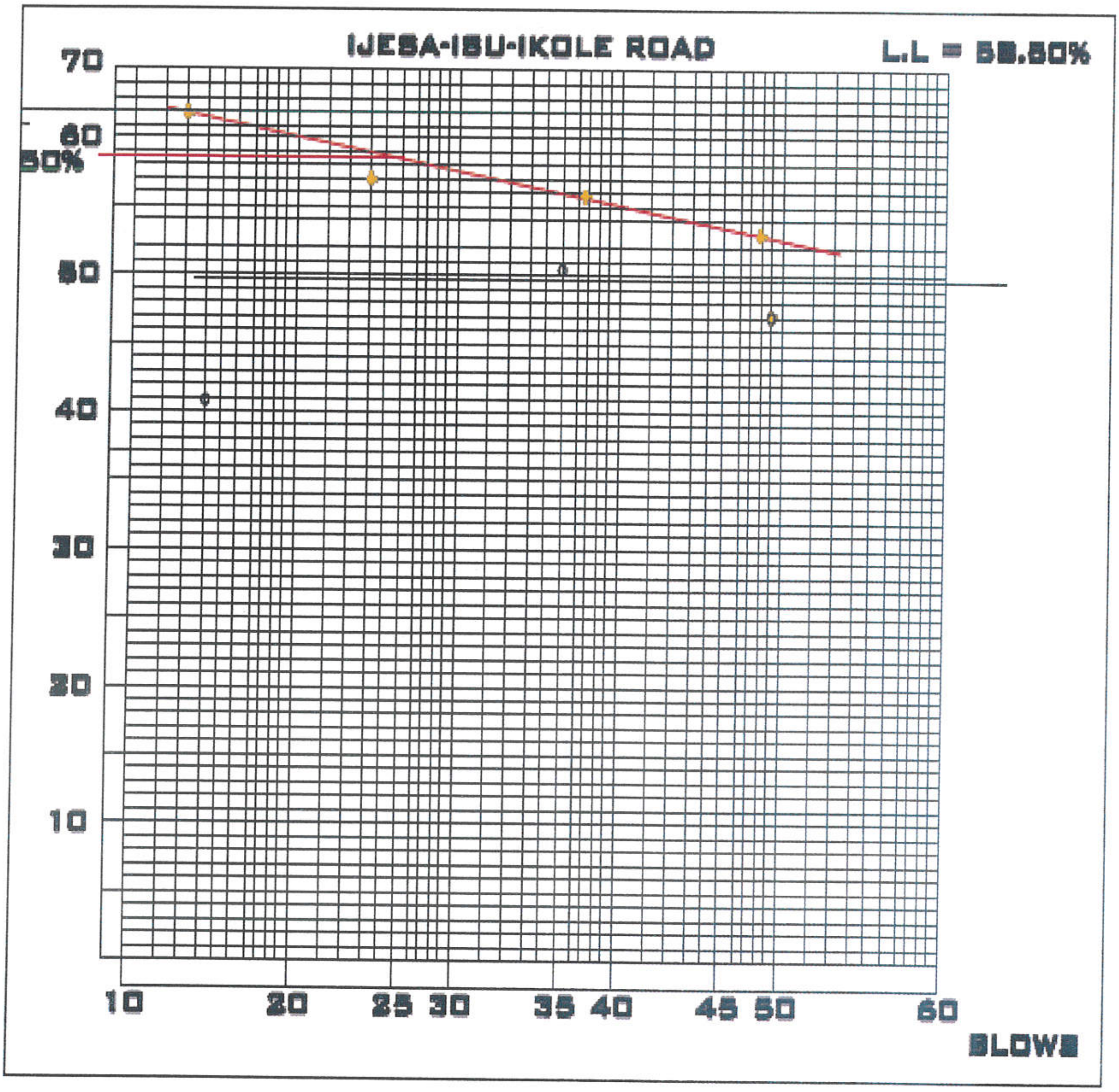


Figure 4.5.1: Graph of Liquid Limit for Ijesa Isu-Ikole Road

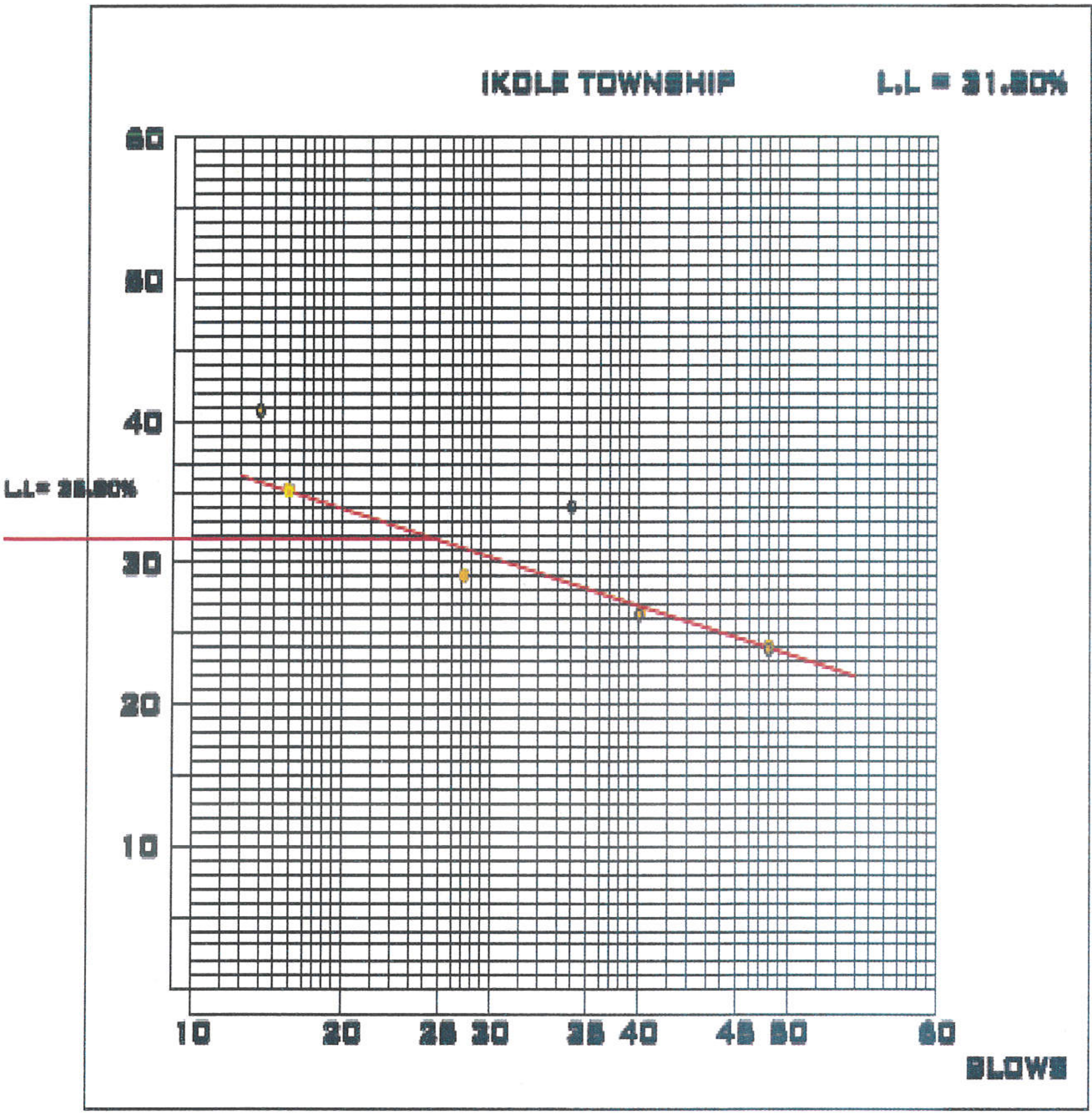


Figure 4.5.2: Graph of Liquid Limit for Ikole Center Town

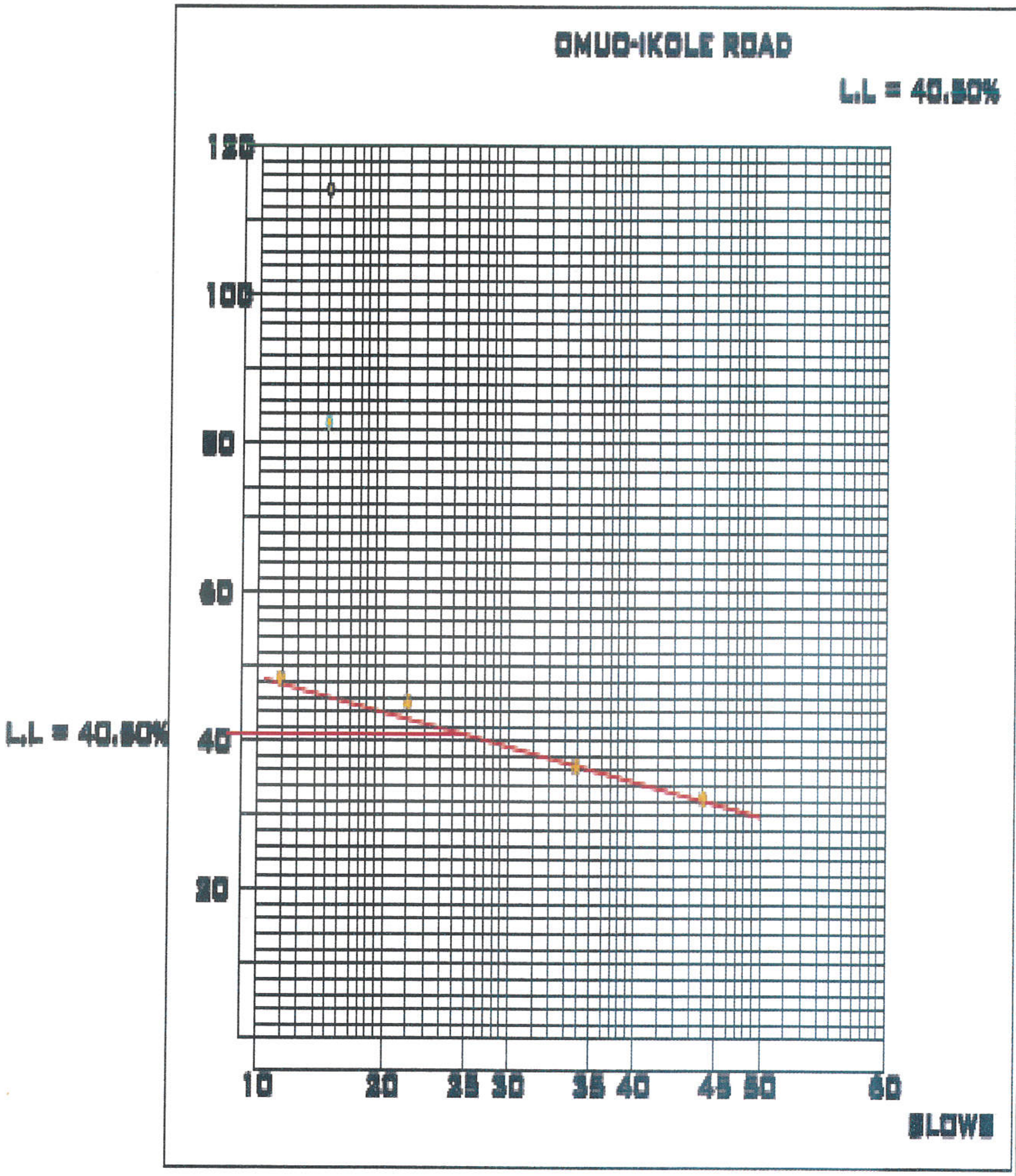


Figure 4.5.4: Graph of Liquid Limit for Omuo-Ikole Road

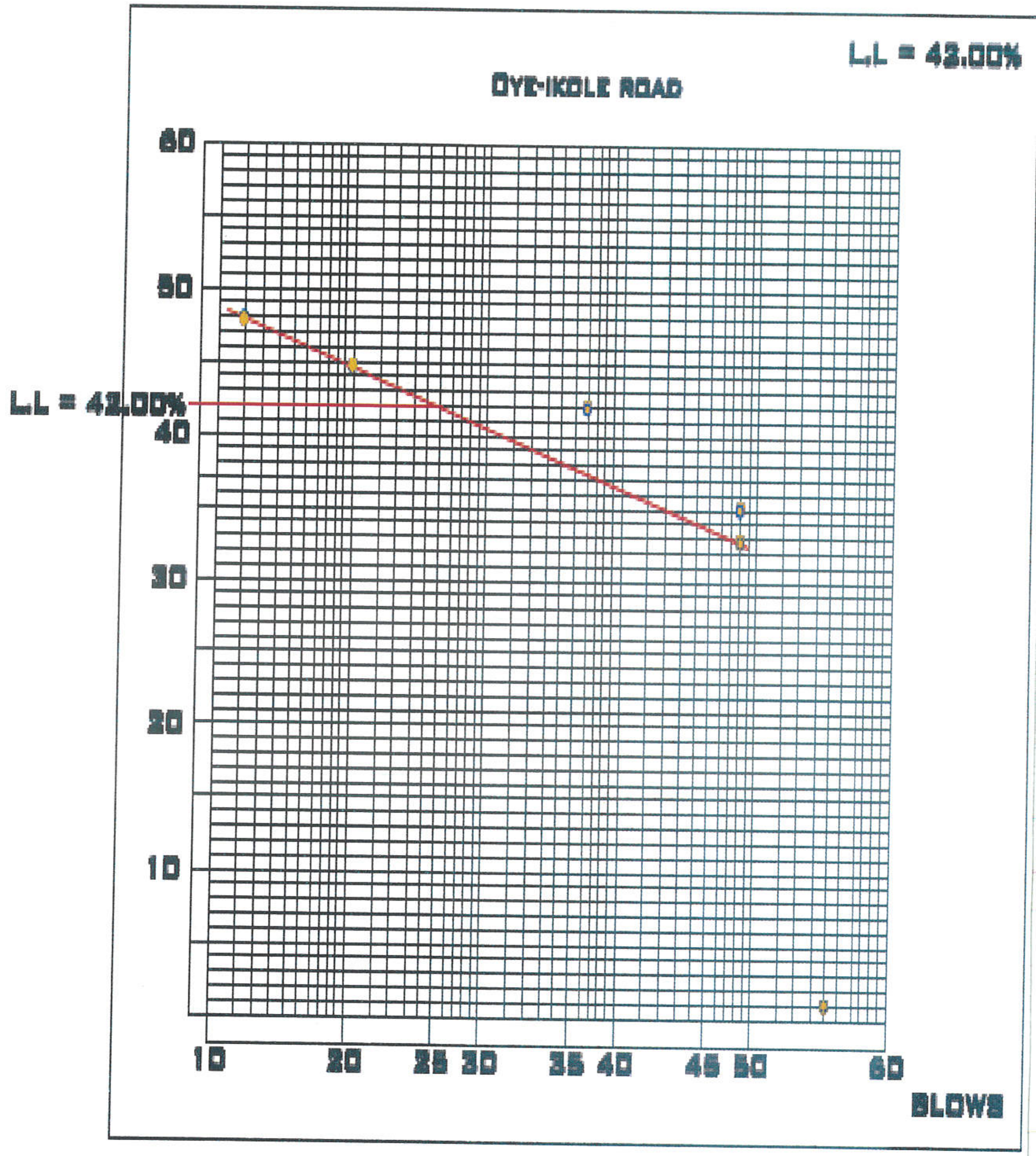


Figure 4.5.5: Graph of Liquid Limit for Oye-Ikole Road

Table 4.5 Results of Atterberg Limits Test

S/N	Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage (%)	Shrinkage Limit (%)	Linear Index (%)
1	Ijesa Isu-Ikole Road	58.50	24.70	33.80	5.20	9.30	0.49
2	Ikole centre town	31.80	15.10	16.70	6.10	6.40	6.20
3	Omuo-Ikole Road	40.80	23.60	17.20	3.80	7.90	5.30
4	Oye-Ikole Road	42.00	28.30	13.70	6.50	7.10	0.51

4.6 California Bearing Ratio Test

This was performed to BS 1377: Methods of test for Soils for civil engineering purposes — Part 4: Compaction-related tests. The CBR with respect to the moisture content that corresponds to the maximum dry density in the compaction test was determined from the graph shown by figures 4.7; to get the mechanical strength of the soil sample, the CBR curve was superimposed on the Proctor curve both using the same horizontal axis (moisture content axis). Table 4.7 shows the CBR value at 2.5 and 5.0 of the samples from the stated locations. The Figures below show the graph representations of the result of CBR tests.

Table 4.6 Result of California Bearing Ratio

S/N	CBR Value	Ijesa Isu-Ikole Road (%)	Ikole Centre town (%)	Omuo-Ikole Road (%)	Oye-Ikole Road (%)
1	2.5	19.30	42.10	78.40	92.40
2	5.0	16.70	52.70	82.40	89.10

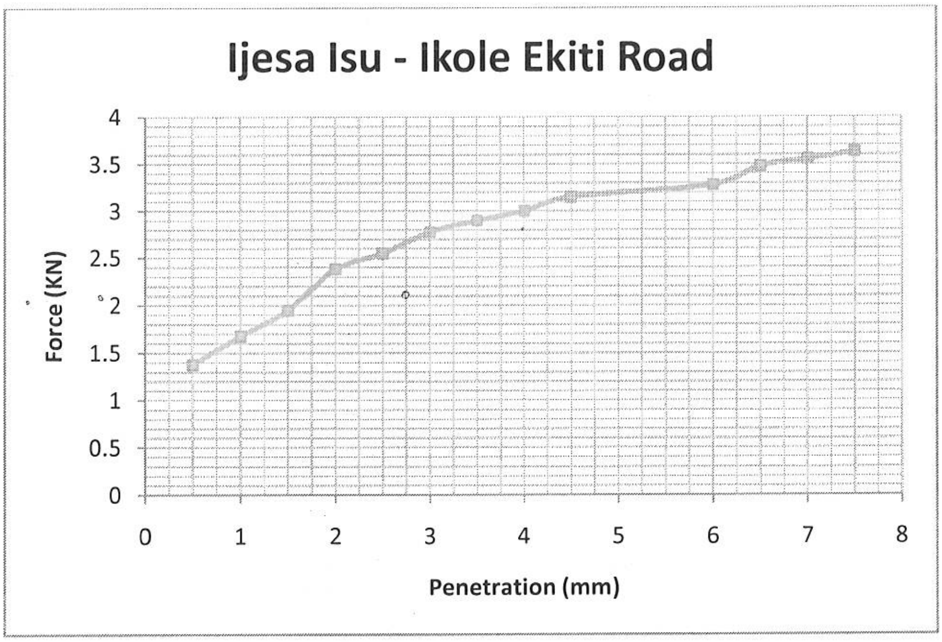


Figure 4.6.1: Graph of Penetration against Force for IjesIsu-Ikole Road

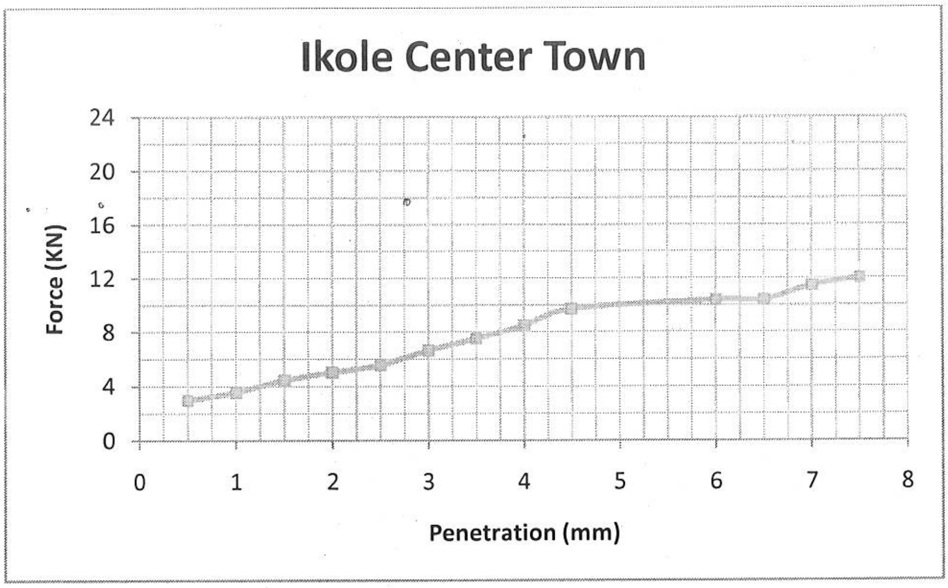


Figure 4.6.2: Graph of Penetration against Force for Ikole Center Town

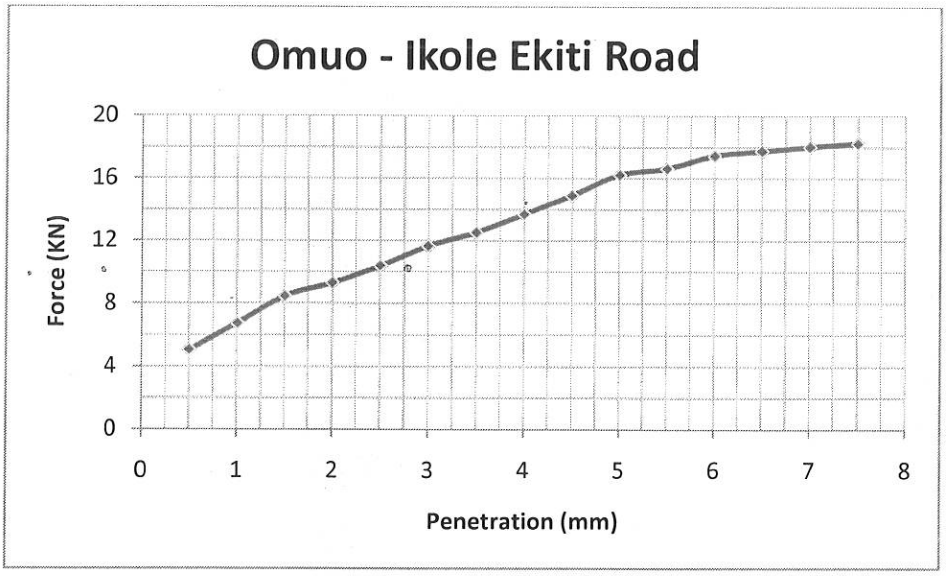


Figure 4.6.3: Graph of Penetration against Force for Omuo - Ikole Road

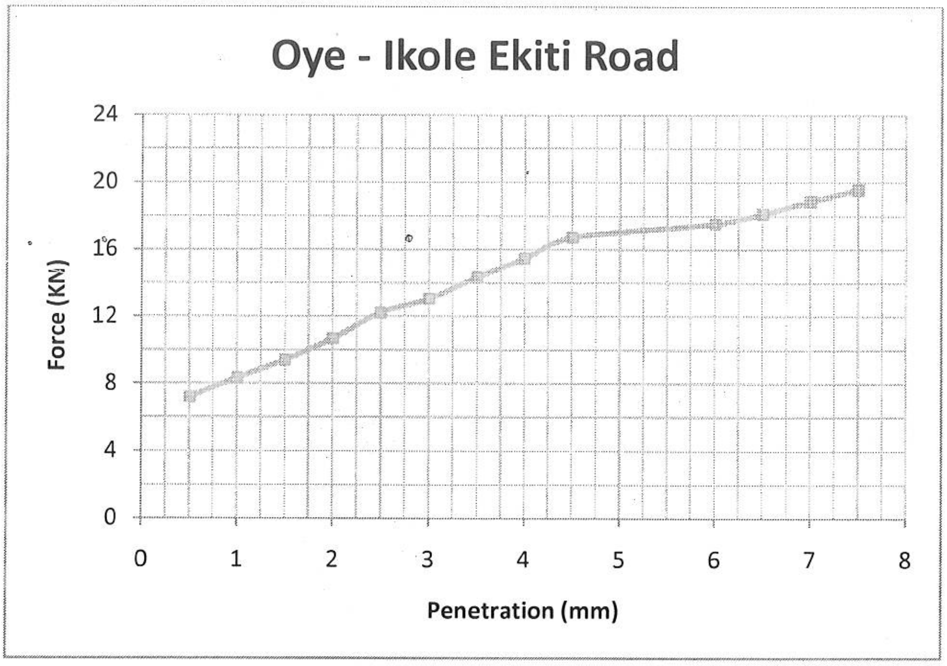


Figure 4.6.4: Graph of Penetration against Force for Oye – Ikole Road

Outcome of the CBR value shown that sample from Oye-Ikole road and Omuo-Ikole road meet the standard specification of BS 1377 and FMW Nig. (1997) for sub-base > 80%.

Oye-Ikole road has the highest value with 92.4% at 2.5 and 89.1% for 5.0 follow with Omuo-Ikole road 78.4% and 82.4% while Ijesa Isu- Ikole road has the less value of 19.3% and 16.7% both for 2.5 and 5.0

4.7 Direct Shear Strength

The direct shear test was conducted on the samples to determine the Cohension, C, and Angle of Internal Friction, Phi. It was observed that there are variations in the properties of the soils at different locations.

Table 4.7: Results of Direct Shear Test

S/N	Location	Ijesa Isu- Ikole Road	Ikole Center Town	Omuo –Ikole Road	Oye – Ikole Road
1	Slope	0.518225	0.606	0.719	0.8405
2	Intercept C	7.333986	31.7	60.83333	41.86667
3	Phi	27.39433	31.21586	35.71614	40.04705

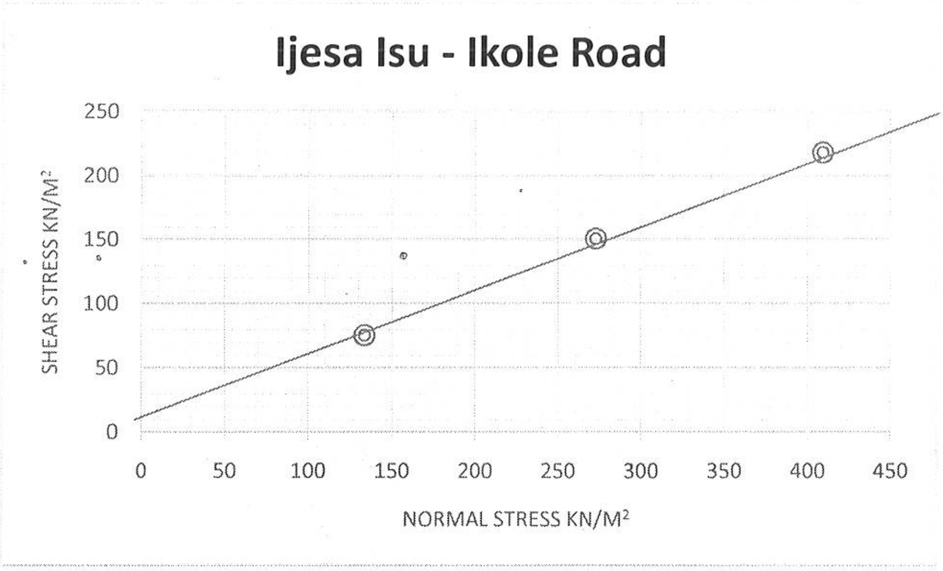


Figure 4.7.1: Graph of Normal Stress against Shear Strength for IjesaIsu – Ikole Road

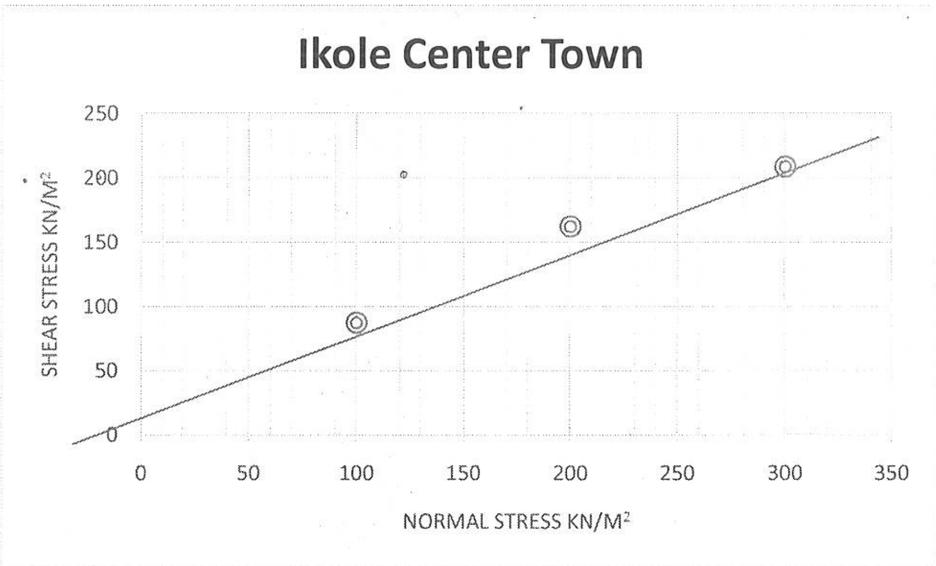


Figure 4.7.2: Graph of Normal Stress against Shear Strength for Ikole Center Town

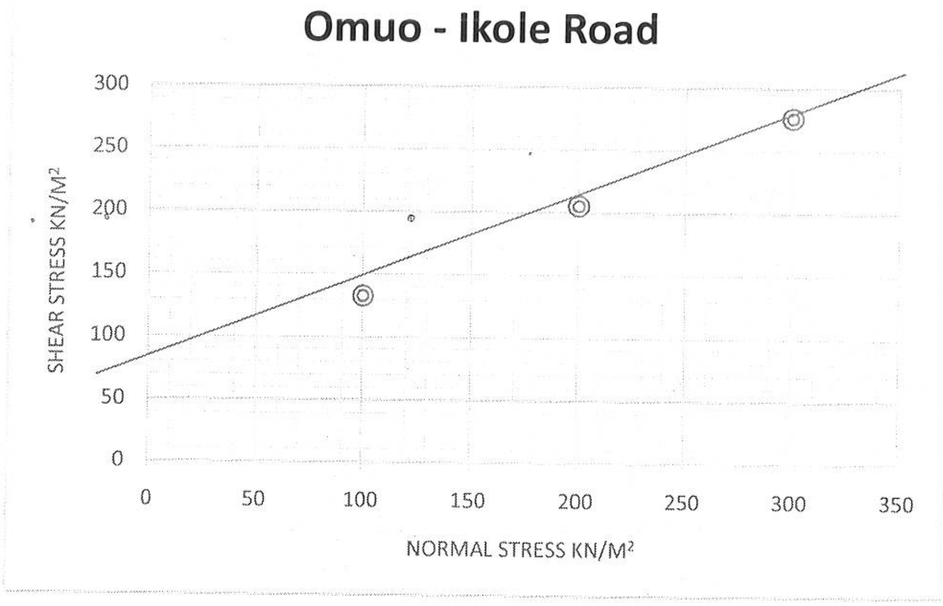


Figure 4.7.3: Graph of Normal Stress against Shear Strength for Omuo – Ikole Road

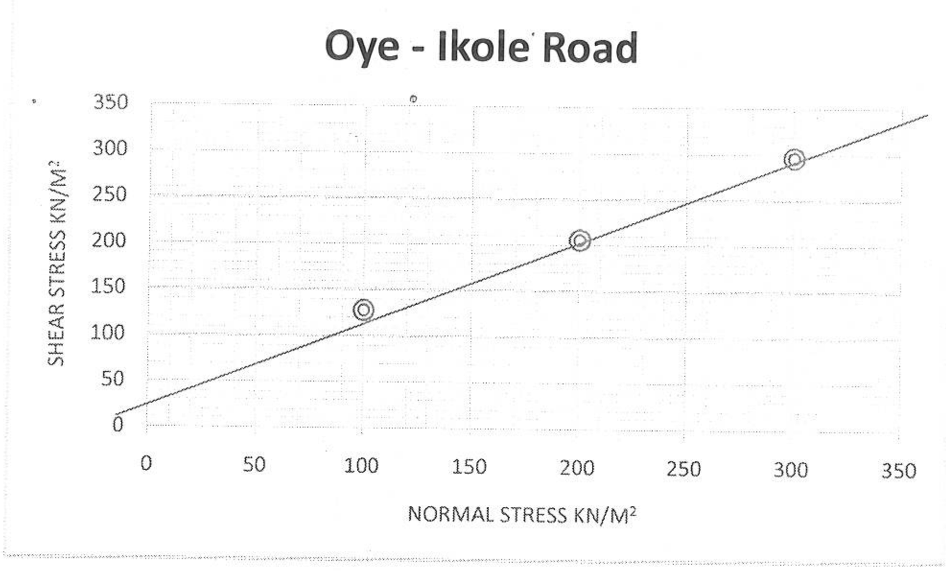


Figure 4.7.4: Graph of Normal Stress against Shear Strength for Oye – Ikole Road

4.8 Permeability Test (Falling Head)

The results of the permeability characteristics using coefficient of uniformity for the studied soil are shown on the table below, showing their grading according to USBR 1952;

Table 4.8: Permeability Test Results

S/N	Sample Location	K (mm/sec)	Grading Type
1	Ijesa Isu - Ikole Road	1.12×10^{-4}	Low
2	Ikole Center Town	1.35×10^{-3}	Medium
3	Omuo- Ikole Road	8.38×10^{-3}	Medium
4	Oye – Ikole Road	2.06×10^{-3}	Medium

The results varies from location to location

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The sample analysis results from various locations in Ikole-Ekiti, Ekiti State show very high percentage of fine soil (i.e. silts clays and clay) with gravel sized particles for two of the locations. This is usually classified under the Unified Soil Classification Scheme (USC) as poorly graded since it does not contain particles of all sizes. This high percentage of fines and shows high porosity, low permeability with decreasing stability. The average specific gravity is 2.4 which is below the standard 2.6 for lateritic soils.

The moisture content (MC) values range from 12.6 - 17.8%. This shows that some areas are well drained while others are not. The significance of the moisture content is that, the greater the amount of water a soil contains, the less interaction there will be between adjacent particles and the more the more the soil will behave like a liquid, i.e. decreasing shear strength. The liquid limit values range between 31.80 - 58.50%. This shows that the soil is of intermediate plasticity, and is an indication of low strength. The plasticity index values range from 13.70 – 33.80%. This indicates that the soil is of low to medium swelling potential. Samples according to AASHTO Classification of highway subgrade material are given as; Ijesa Isu-Ikole Road is A7, Ikole center town is A2-6, Omuo-Ikole Road is A7 and Oye-Ikole Road is A2-7.

The larger the plasticity index, the greater is the engineering problem associated with using the soil as an engineering material.

The soil samples tested from the study area indicate a general cohesive high MDD and low OMC, with sample from Oye-Ikole Road being the sample with the highest MDD and lowest OMC, an indication that the sample is good for good for road construction.

5.2 Recommendations

These valuable data obtained from the geotechnical analysis can be useful for civil engineers in the design and construction of roads in Ikole-Ekiti and environs for maximum durability and efficiency. It is recommended that engineering confirmatory tests be carried out before embarking on any construction of roads. Locations with clayey soil particles should be stabilized with either cement, sand; crushed stone (gravels) of ½ and 3/4 inch size in order to meet the subbase or base course requirement. Further Research should be done on other areas in Ikole-Ekiti to have a concrete characterization of the community;

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APPENDIX 1

(MOISTURE-CONTENT TEST)

	IJESA ISU – IKOLE RD		IKOLE CENTER RD		OMUO – IKOLE RD		OYE – IKOLE RD	
	1	2	1	2	1	2	1	2
Can number	A	B	C	D	E	F	G	H
Can + wet soil	56.2	57.5	56.5	59.5	41.4	40.4	58.7	51.6
Can + dry soil	50.4	51.3	52.3	55.1	37.1	35.9	53.9	47.9
Wt of can	17.5	12.2	19.6	19.5	12.9	11.3	17.8	18.9
Ww	5.8	6.2	4.2	4.4	4.3	4.5	4.8	3.7
Wds	32.9	39.1	32.7	35.6	24.8	24.6	36.1	29.0
Mc	17.6	15.9	12.8	12.4	17.3	18.3	13.3	12.8

AV .MC = 17.8%   13.1%   16.8%   12.6%

APPENDIX 2

(GRAIN SIZE ANALYSIS)

	IJESA ISU – IKOLE ROAD			IKOLE CENTER TOWN			OMUO-IKOLE ROAD			OYE – IKOLE ROAD		
S/S	Wt. Ret	% Ret	% Passing	Wt. Ret	% Ret	% Passing	Wt. Ret	% Ret	% Passing	Wt. Ret	% Ret	% Passing
9.50	7.8	1.56	98.4	23.7	4.74	95.3	3.9	0.78	99.2	20.1	5.02	96.0
4.75	58.8	9.76	88.7	22.0	4.40	90.9	52.6	10.52	88.7	52.9	10.58	85.4
2.36	43.4	8.68	80.0	34.8	6.96	83.9	47.4	9.48	79.2	54.4	10.88	74.5
1.18	21.5	4.30	75.7	50.7	10.14	73.8	45.5	9.10	70.1	52.8	10.56	64.0
600	32.3	6.46	69.2	61.5	12.30	61.5	48.5	9.70	60.4	60.7	12.14	51.9
350	49.5	9.90	59.3	63.5	12.70	48.8	43.5	8.70	51.7	54.0	10.8	41.1
150	29.4	5.88	53.5	53.1	10.62	38.2	35.5	7.00	44.7	42.7	8.54	32.5
75	12.4	2.46	51.0	278	5.56	32.6	35.0	7.00	39.9	26.7	5.34	27.2

APPENDIX 3

(SPECIFIC GRAVITY TEST)

Specific Gravity is given by

$$\frac{M_2 - M_1}{(M_2 - M_2) - (M_3 - M_2)}$$

(1) Oye-Ikole

$$A = \frac{40.3 - 15.8}{(98.9 - 40.3) - (71.1 - 15)} = \frac{24.5}{58.6 - 53.3} = 4.623$$

$$B = \frac{40.3 - 15.8}{(98.9 - 40.3) - (71.1 - 15)} = \frac{24.5}{58.6 - 53.3} = 4.623$$

$$Av.GS = \frac{2.284+2.328}{2} = 2.305$$

(2) Omuo-IkoleEkiti

$$A = \frac{39.5 - 14.6}{(92.1 - 14.6) - (106.1 - 39.5)} = \frac{24.9}{10.9} = 2.284$$

$$B = \frac{49.0 - 17.1}{(110.7 - 17.1) - (128.7 - 49.0)} = \frac{31.9}{13.7} = 2.328$$

$$Av.GS = \frac{2.284 + 2.328}{2} = 2.305$$

(3)

$$A = \frac{43.4 - 20.5}{(72.8 - 20.5) - (85.5 - 43.4)} = \frac{22.9}{9.9} = 2.313$$

$$B = \frac{55.0 - 17.1}{(69.3 - 17.1) - (92.2 - 55.0)} = \frac{37.3}{15} = 2.487$$

$$Av.GS = \frac{2.313 + 2.487}{2} = 2.385\ 76.7\ 66.7$$

(4)

$$A = \frac{45.1 - 21.6}{(92.3 - 21.6) - (107.6 - 45.1)} = \frac{23.5}{8.2} = 2.867$$

$$B = \frac{39.3 - 17.1}{(93.8 - 17.1) - (107.0 - 39.3)} = \frac{22.2}{10} = 2.220$$

$$Av.GS = \frac{2.867 + 2.220}{2} = 2.544$$

APPENDIX 4

(COMPACTION TEST)

IJESA ISU – IKOLE ROAD

Trial Number	1	2	3	4	5
Wt of mould + wet soil	5500	5650	5850	5900	5700
Wt of mould empty	3800	3800	3800	3800	3800
Wet density (g)	1700	1850	2050	2100	2000
Wet density (kg)	1.70	1.85	2.05	2.10	2.00
Can number	K_1	K_2	K_3	K_4	K_5
Can wt empty (g)	23.7	9.5	15.9	16.1	14.2
Wt can + soil wet	89.6	58.7	97.0	68.9	69.6
Wt soil + can dry (g)	87.2	50.3	87.1	58.8	58.4
Wt of water (ww)	6.9	8.4	9.9	10.1	11.2
Wt of dry soil (wds)	63.5	78	71.2		44.2
Mc	6.9	9.8	13.9	19	25.3
Dry density (kg/m^3)	1.59	1.68	1.80	1.76	1.60

IKOLE CENTER TOWN

Trial Number	1	2	3	4
Wt of mould + wet soil	5100	5250	5350	5200
Wt of mould empty	3250	3250	3250	3250
Wet density (g)	1850	2000	2100	1950
Wet density (kg)	1.85	2.00	2.10	1.95
Can number	C_1	C_2	C_3	C_4
Can wt empty (g)	15.4	10.4	20.0	10.2
Wt can + soil wet	53.6	62.6	71.3	62.4
Wt soil + can dry (g)	49.9	56.8	62.9	52.8
Wt of water (ww)	3.7	5.8	8.4	9.6
Wt of dry soil (wds)	34.5	46.4	42.9	42.6
Mc	10.7	15.7	19.6	22.5
Dry density (kg/m^3)	1.67	1.73	1.86	1.59

OMUO – IKOLE ROAD

Trial Number	1	2	3	4
Wt of mould + wet soil	5500	5950	5950	5850
Wt of mould empty	3800	3800	3800	3800
Wet density (g)	1750	1950	2150	2050
Wet density (kg)	1.75	1.95	2.15	2.05
Can number	A_1	A_2	A_3	A_4
Can wt empty (g)	13.9	20.4	14.3	11.8
Wt can + soil wet	61.4	70.8	64.4	59.5
Wt soil + can dry (g)	56.1	63.8	55.8	50.0
Wt of water (ww)	5.3	7.0	8.6	9.5
Wt of dry soil (wds)	42.2	42.4	41.5	38.2
Mc	12.6	16.5	20.7	24.9
Dry density (kg/m^3)	1.55	1.67	1.78	1.64

OYE-IKOLE ROAD

Trial Number	1	2	3	4
Wt of mould + wet soil	5700	6000	6100	5950
Wt of mould empty	3800	3800	3800	3800
Wet density (g)	1900	2200	2300	2150
Wet density (kg)	1.90	2.20	2.30	2.15
Can number	B_1	B_2	B_3	B_4
Can wt empty (g)	19.5	9.5	15.5	10.3
Wt can + soil wet	82.3	71.1	71.4	75.9
Wt soil + can dry (g)	77.5	64.7	64.0	65.5
Wt of water (ww)	5.8	6.4	7.4	10.4
Wt of dry soil (wds)	58.0	55.3	48.5	55.2
Mc	8.3	11.6	15.2	18.8
Dry density (kg/m^3)	1.75	1.97	2.00	1.81

APPENDIX 5

(ATTERBERG LIMITS)

IjesaIsu – Ikole Road

	LL				PL	
	1	**2**	**3**	**4**		
Number of blows	48	37	23	12		
Can number	A_1	B_1	C_1	D_1	E_1	F_1
Wt of empty can (g)	15.5	14.2	19.0	23.0	20.3	10.6
Wt of can + soil wet	35.4	37.4	33.6	49.5	36.7	22.9
Wt of can + soil dry	28.5	29.1	24.6	39.4	33.4	20.5
Wt of water (g)	6.9	8.3	9.0	10.1	3.3	2.4
Wt of dry soil	13.0	14.9	15.6	16.4	13.1	9.9
Mc	53.1	55.7	57.7	61.6	25.2	24.2

PL =

24.7%

(SL) =1.3

SL =

9.3%

Ikole Centre Town

	LL				PL	
	1	**2**	**3**	**4**		
Number of blows	48	34	24	15		
Can number	A_4	B_4	C_4	D_4	E_4	F_4
Wt of empty can (g)	20.9	19.1	19.7	15.4	16.1	17.1
Wt of can + soil wet	42.8	41.8	43.0	41.1	46.5	50.2
Wt of can + soil dry	38.4	37.1	37.8	35.2	42.6	46.7
Wt of water (g)	4.2	4.7	5.2	5.9	3.9	3.5
Wt of dry soil	18.3	17.3	18.1	19.3	26.5	22.5
Mc	24.0	26.1	28.7	30.6	14.7	15.6

PL =

15.1%

(SL) = 0.9

SL =

6.4%

Omuo – Ikole Road

	LL				PL	
	1	2	3	4		
Number of blows	44	34	21	11		
Can number	A_2	B_2	C_2	D_2	E_2	F_2
Wt of empty can (g)	9.2	16.4	12.7	11.5	8.8	9.2
Wt of can + soil wet	25.6	32.5	33.6	36.8	26.0	21.3
Wt of can + soil dry	21.7	28.2	27.1	28.5	22.4	18.7
Wt of water (g)	3.9	4.3	6.5	8.3	2.6	2.6
Wt of dry soil	12.5	15.6	14.4	17.0	13.6	9.5
Mc	31.2	36.4	45.1	48.8	19.1	27.4

PL =

23.3%

(SL) =1.1

SL =

7.9%

Oye – Ikole Road

	LL				PL	
	1	**2**	**3**	**4**		
Number of blows	48	36	20	11		
Can number	A_3	B_3	C_3	D_3	E_3	F_3
Wt of empty can (g)	7.3	7.4	10.3	6.8	1.2	10.5
Wt of can + soil wet	25.1	25.7	31.6	31.2	28.4	24.8
Wt of can + soil dry	20.7	20.2	25.0	23.3	25.1	21.6
Wt of water (g)	4.4	5.5	6.6	7.9	3.3	3.2
Wt of dry soil	13.4	12.8	14.7	16.5	11.9	11.1
Mc	32.8	43.0	44.9	47.9	27.7	28.8

PL =

28.3%

(SL) = 1.0

SL =

7.1%

APPENDIX 6

(C.B.R. TEST)

	IJESA ISU- IKOLE ROAD		IKOLE CENTER TOWN		OMUO – IKOLE ROAD		OYE – IKOLE ROAD	
Penetration	**DR**	**LOAD**	**DR**	**LOAD**	**DR**	**LOAD**	**DR**	**LOAD**
50	55	1.38	119	2.98	202	5.05	288	7.20
100	67	1.68	143	3.58	269	6.73	333	8.33
150	78	1.95	178	4.45	338	8.45	376	9.40
200	95	2.38	201	5.03	371	9.28	427	10.68
250	102	2.55	223	5.58	415	10.38	489	12.23
300	111	2.78	265	6.63	466	11.65	522	13.05
350	116	2.90	300	7.50	501	12.53	573	14.33
400	120	3.00	339	8.48	548	13.70	618	15.45
450	126	3.15	388	9.70	596	14.9	669	16.73
500	131	3.28	415	10.38	649	16.22	702	17.55
550	139	3.48	437	10.93	665	16.63	725	18.13
600	142	3.55	458	11.45	698	17.45	755	18.88
650	145	3.63	481	12.03	710	17.75	783	19.58
700	148	3.70	508	12.70	721	18.03	801	20.03
750	151	3.78	520	13.00	730	18.25	811	20.28