# COMPACTION AND STRENGTH CHARACTERISITCS OF SOILS IN IKOLE-EKITI AREA AND THEIR INFLUENCE IN CIVIL ENGINEERING CONSTRUCTION

By

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## ABSTRACT

This study investigates the influence of two main geotechnical properties of soils in the town of Ikole-Ekiti on civil engineering construction. The strength and compaction characteristics of the natural soil were determined. The investigation was carried out through laboratory tests on disturbed and undisturbed soil samples obtained from four trial pits (Samples A, B, C & D) in the town, across Ilotin, Ijemo-Titun, Yeye-Oba and Asin respectively. The samples were collected between depths of 1.5m and 2.0m. The soils were mostly lateritic (except Sample C) and were all coarse-grained. The consistency limits indicate Liquid Limit of 29- 52 %, Plastic Limit of 12- 29%, Plasticity Index of 17- 28% and Shrinkage Limit of 7.1- 12.1%. The average specific gravity ranges from 2.31- 2.63, these make all the soils suitable as subgrade material. According to AASHTO, the soils were respectively classified as A-2-7, A-2-7, A-2-6 and A-2-7. While in the USCS, the soils were also classified as SW, SP, SW-OL and SW-SC. Standard Proctor compaction tests indicated Maximum Dry Densities of 1.73 g/$cm^3$, 1.71 g/$cm^3$, 1.79 g/$cm^3$ and 1.56 g/$cm^3$ at optimum moisture contents of 19.05%, 19.80%, 14.90% and 15.40% for soil samples obtained from Trial Pits A, B, C and D respectively. While modified Proctor compaction tests indicated Maximum Dry Densities of 1.95g/$cm^3$, 1.92g/$cm^3$, 1.99 g/$cm^3$ and 1.87 g/$cm^3$ at optimum moisture contents of 18.20%, 20.20%, 15.70% and 16.90% for soil samples obtained from Trial Pits A, B, C and D respectively, Sample A, B& D are suitable as landfill barrier materials. The direct shear tests indicate undrained cohesion within the range (11- 74.41) $kN/m^2$ and corresponding range for the undrained angle of internal friction $15.26^o$- $18.44^o$. Triaxial (Consolidated Drained) and Unconfined compression tests were carried out on Samples A, B & D. Triaxial results showing range of (50.1 – 52.2) $kN/m^2$, at an angle of internal friction of $18^o$. These soils can be used for shallow foundations for loads of the order of 20- 53 $kN/m^2$. While for Unconfined Compression, the undrained shear strength range from (21.73 – 47.43) $kN/m^2$. These values are also important in the construction of finite& infinite slopes, of which the critical height of the slope is yet to be determined. Factor of safety can also be derived, in the design process of such slopes to avoid failure later on, before/ during/ after construction process.

## DEDICATION

I dedicate this work to GOD almighty my creator, my strong pillar, my source of inspiration, the reason behind my breath, the Author of wisdom, knowledge, and understanding. I also dedicate this report to my parents and siblings for their support and their encouragement.

## CERTIFICATION

This is to certify that this project was written by **ODEWUMI MOYOSOLUWA MICHAEL (CVE/13/1063)** under my supervision and is approved for its contribution to knowledge and literary presentation. All sources of information are specifically acknowledged by means of references, in full requirements for the award of Bachelor of Engineering (B.Eng.) degree in Civil Engineering, Federal University Oye Ekiti.

______________________ 21/03/2019

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(Head of Department)

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# CHAPTER ONE

# INTRODUCTION

## 1.1 Theoretical Background

Engineering is a profession which applies the knowledge of science and technology to proffer solutions to societal problems facing humanity. The development of engineering over the years has been driven by various discoveries by scientist and inventions of modern tools by technologist which are always results of proven hypothesis through different researches with different degrees of comprehensiveness. Civil engineering structures like buildings, bridges, highways, tunnels, railways, dams, harbours, etc., are founded below or on the surface of the earth. For their stability, suitable foundation soil is required. Soil is different than any other material that we work with in construction, as it is non-uniform. Soils can be used as it is, it can be improved upon and also be replaced with better materials during construction works.

The size, shape, and arrangement of mineral grains which form the soil mass is known as soil structure, and it helps in explaining the engineering behaviour of soils. To check the suitability of soil to be used as foundation or as a construction material, its properties are assessed. This is done so as to have sufficient knowledge about the geotechnical characterization of the site on which the civil engineering structure is to stand. There are various geotechnical properties of soils which are tested for on a site: they include- specific gravity, density index, consistency limits (Liquid limit, plastic limit and shrinkage limit), particle size analysis, compaction, consolidation, permeability and strength. A good knowledge about a site including its subsurface conditions is very important in its safe and economic development. It is therefore an essential preliminary to the construction of any civil engineering work such as roads, buildings, dams, bridges, foundations, etc.

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 (Knapett& Craig; 2012). It is one of the ground improvements techniques. It is a process in which by expending compactive energy on soil, the soil grains are more closely arranged. Compaction increases the shear strength of soil and reduces its compressibility. Murthy (2000) explained that when an earth dam is compacted, thereby making the shear strength increase, the dam becomes more stable. Since the soil becomes denser, its permeability decreases. The decrease in permeability of the dam decreases the loss of the water stored through seepage. The settlement of the dam also decreases due to the increase in the density of the materials. Compaction of soils increases the bearing capacity but reduces the void ratio, porosity, permeability and settlement. Compaction is used in the construction for roads, airports, car parks, earth-dam or rock dam construction.

Strength is an important aspect of most engineering materials. It is very important in the consideration of the bearing capacity of footing and piles, the stability of natural and cut slopes, dams and embankment and the earth pressure against retaining walls. The strength of material is defined as the greatest stress it can sustain. If the stress exceeds the strength, failure occurs. Strength analysis can be performed on the basis of tensile, compressive and shear stresses. Because soil has very little or negligible tensile strength, geotechnical engineers rarely perform tensile strength analysis. The geometry of most geotechnical problems is such that even when the soil mass is in compression, it does not fail in compression. Although the introduction of large stresses may result in soil failure, the soil actually fails in shear, not in compression. Therefore, nearly all geotechnical strength analysis is done using shear strength theories. The shear resistance of soil is the result of friction and the interlocking of particles and possibly cementation or bonding at the particle contacts. The shear strength parameters are defined as cohesion and the friction angle. The shear strength of soil depends on the effective stress, drainage conditions, density of the particles, rate of strain and the direction of strain. Thus the shearing strength is affected by the consistency of materials, mineralogy, grain size distribution, shape of the particles, initial void ratio and features such as layers, joints, fissures and cementation. The shear strength parameters of a granular soil are directly correlated to the maximum particle size, the coefficient of uniformity, the density, the applied normal stress and the gravel and fines contents of the sample. The shear strength parameters are related to the frictional forces, which arise when the soil particles slide and interlock during shearing. The capability of a soil to support a loading from a structure, or to support its overburden, or to sustain a slope in equilibrium is governed by its shear strength.

Unfortunately in developing countries like Nigeria only few investors in the construction industry take time to execute subsoil investigation prior to commencement of construction activities on their projects. The result is the disastrous consequences such as failure or collapse of buildings and other massive engineering structures which often cause untold hardship and damage and sometimes even loss of lives and properties.

## 1.2 Problem Statement

The Civil Engineer is often faced with the practical problems raised by use of soil as a foundation and construction material. A careful consideration of experimental investigation and the need for simplicity in the means employed has to be attained. In Nigeria, the non-availability of relevant data in this area, particularly for initial engineering planning and designs, has been the major cause of failure of most civil engineering works. The geotechnical properties such as compaction and strength of soils across various areas in Ikole metropolis have not been adequately studied and researched upon. Hence the life-span and serviceability of the structures in the area can't be pre-determined without proper geotechnical investigation.

## 1.3 Aim of this Study

To determine the various applications in civil engineering construction of Ikole soils based on their compaction and strength characteristics. (To assess the suitability of Ikole soils for civil engineering construction)

## 1.4 Objectives of this Study

1. To obtain representative soil samples from four locations in Ikole area.

2. To classify the soils based on the Unified Soil Classification System (USCS) and AASHTO systems.

3. To carry out compaction and strength tests on the samples.

4. To analyze the results to determine the uses to which the various soils can be put for civil engineering construction.

## 1.5 Scope of this Study

This work was limited to determination of the compaction and strength characteristics of soils in Ikole area. The investigation is not extended to other geotechnical properties of soils such as consolidation, bearing capacity, etc. The extent of the study didn't go outside the metropolis of Ikole such as the neighbouring communities around Ikole. The essence of the study is also limited to the use of the soils for civil engineering construction and not for any other purpose.

## 1.6 Significance of this Study

This research helps in determining the significance of soils found in Ikole area (to different aspects) for civil engineering construction based on their compaction and strength properties using various parameters from previously established works, the British Standard and other codes of engineering practice being used in Nigeria.

## 1.7 Study Area

The study area is Ikole-Ekiti which is a town in Ikole Local Government Area of Ekiti State, which is a state in Western Nigeria (Fig 1.1 & Fig 1.2). It became a state on 1 October 1996, alongside five others by the military under General Sani Abacha. The state, carved out of the territory of the old Ondo State, covers the former twelve local government areas that made up the Ekiti-Zone of old Ondo State. On creation, it had sixteen Local Government Areas (LGAs), having had an additional four carved out of the old ones. Ekiti State is one of the thirty-six states that makes up the Federal Republic of Nigeria. The State is mainly an upland zone, rising over 250 meters above sea level. The State is dotted with rugged hills.

Ikole is located between latitudes $7.47'00^{O}$N and $7.78'33^{O}$N, and longitudes $5.31'00^{O}E$ and $5.51'67^{O}E$. The general geology of Ekiti State is well researched. The state is underlain by the Precambrian rocks of the Basement Complex of Southwestern Nigeria which covers about 50% of the land surface of Nigeria. The general digitized geological map of Ekiti State is shown in Figure 1.3. The major lithological units include the granite gneiss, migmatites gneiss and charnockite. The Basement rocks show great variations in grain size and in mineral composition. The rocks are predominantly quartz, gneisses and schists consisting essentially of quartz with small amounts of white micaceous minerals.

Ikole-Ekiti lies in an area underlain by metamorphic rock. It is a generally undulating country with a characteristic landscape that consists of old plains broken by step-sided out-crops that occur singularly or in groups or ridges. Ikole-ekiti enjoys tropical climate with two distinct seasons. These are the rainy season (April–October) and the dry season (November–March). Temperature ranges between $21^oC$ and $28^oC$ with high humidity. The south-westerly wind and the north-east trade winds blow in the rainy and dry (Harmattan) seasons respectively. Tropical forest exists in the south, while savannah occupies the northern peripheries. Ikole's land is one of the most fertile and with high degree of accessibility to water bodies.

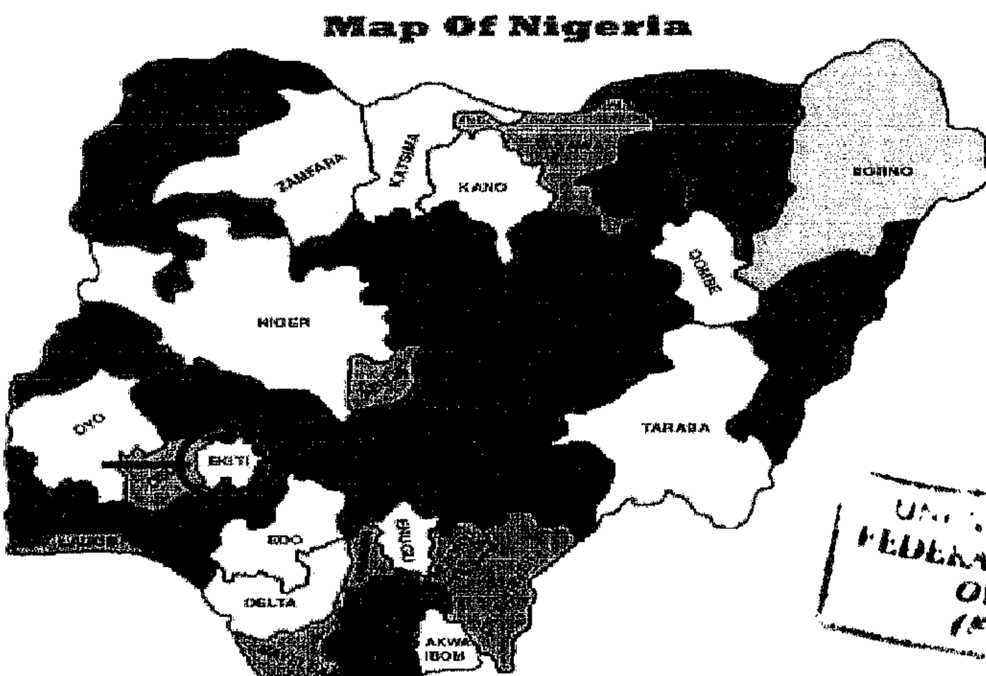





**Figure 1.1**: The Map of Nigeria showing the 36 states of the Federation& FCT (adapted from National Geographic Society, 1998)

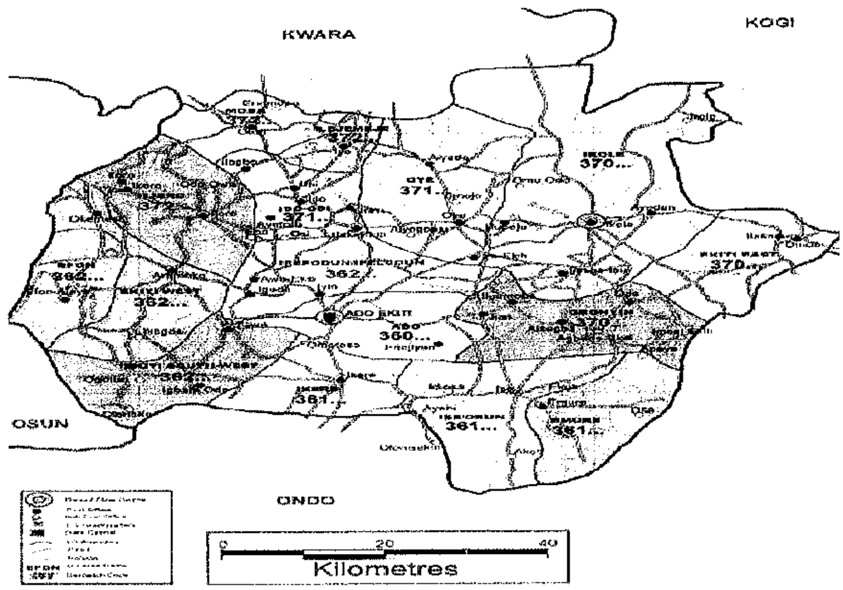



**Figure 1.2**: An enlarged map of Ekiti state showing the 16 L.G.A's (Ekiti State Directorate of ICT, *The Official Website of the Government of Ekiti State, Nigeria,* Available: https://ekitistate.gov.ng/administration/localgovt/lga , 2015)

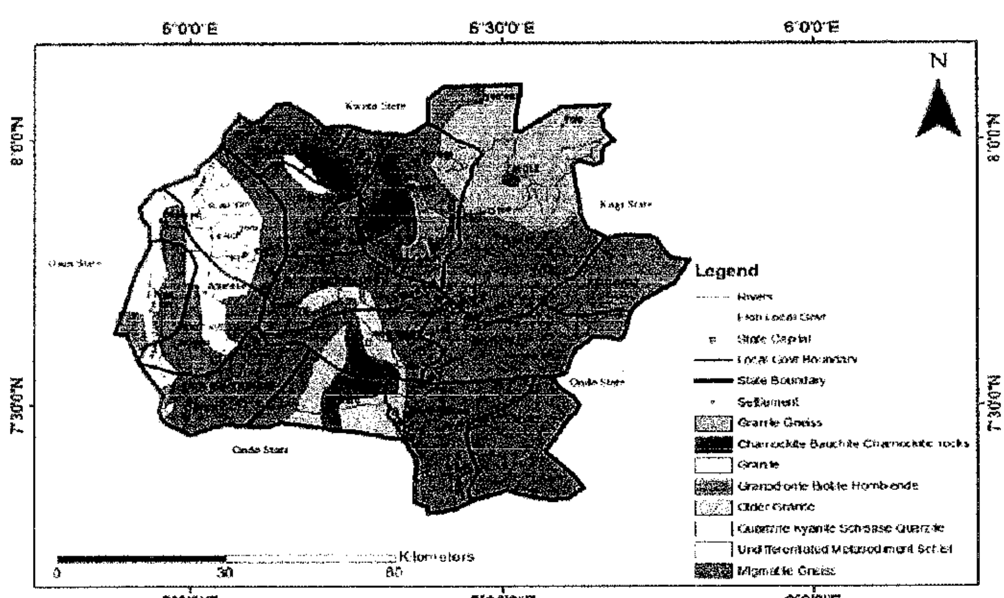



**Figure 1.3**: The Geological Map of Ekiti State. (Wikimedia Foundation Inc., *Ekiti state,* Available: https://en.wikipedia.org/wiki/EkitiState, 2015).

# CHAPTER TWO

# LITERATURE REVIEW

## 2.1 Introduction

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 (superstructure) or within (substructure) 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) involve foundation and anchor systems for offshore structures such as oil platforms.

In a study report on Compaction equipment and construction machinery by the Government of India, Ministry of Railways (2005), compaction in the field is affected by the soil type, the layer thickness, contact pressure, number of roller passes and the speed of rolling.

## 2.2 Review from Researched Works

Not much attempts have been made to study the geotechnical properties of soils around Ekiti State and in Southwestern Nigeria as a whole, as relating to these geotechnical properties (compaction and strength) and the various uses of soils in civil engineering construction. Adeyeri *et. al.,* (2017) investigated the geotechnical properties and litho-stratigraphic soil profile of soils in Ootunja axis of ikole area of Ekiti state based on analysis of laboratory tests on disturbed and undisturbed soil and water samples from the wash boring method. It revealed a subsurface stratification made up of reddish brown granitic clayey sand from existing ground level to a depth of 9m to 12.0m from 3 different boreholes. This is underlain by a layer of mottled, brown, decomposed micaceous sand to a depth of 16.5m - 18m immediately after which are layers of mottled grey, decomposing quartzite sand to about 18.0m to 19.0m depth. This is further underlain by fragments of freshly weathered granitic rock to the termination depth of 19.5m. The consistency limits indicate LL of 44 – 58% and PL of 18 – 26% while the quick undrained triaxial tests indicate undrained cohesion within the range (128.4 - 157)$kN/m^2$ and undrained angle of internal friction$(15.0 - 20.0)^o$. Groundwater was encountered between 6.5m to 7.5m below the existing ground level during the course of soil exploration. The water in the area has pH values of 5.79 to 7.05, the chloride content values range between 12.0mg/l and 16.0mg/l while sulphate content varies from 6.12mg/l to 7.10mg/l.

Bolarinwa *et. al.,* (2017).investigated the compaction and consolidation characteristics of lateritic soils in Igbona axis of Ikole area of Ekiti state. The investigation was carried out through laboratory tests on disturbed and undisturbed soil samples obtained from three borings (BH1, BH2, and BH3). The soils are all lateritic and mostly fine-grained. Compaction tests indicate maximum dry densities of 2.05$Mgm^{-3}$, 1.78 $Mgm^{-3}$, and 1.69 $Mgm^{-3}$ at optimum moisture contents of 14.3%, 20.7% and 19.6% for soil samples obtained from BH1, BH2 and BH3 respectively. Compression indices $(C_c)$ obtained from oedometer tests are 0.04816, 0.03820 and 0.04318 while the calculated coefficients of volume compressibility $(m_v)$, are $1.308\,X\,10^{-4}$, $1.065\,X\,10^{-4}$and $1.093\,X\,10^{-4}$, $m^2kN^{-1}$for samples in BH1, BH2 and BH3 respectively. The unsoaked California Bearing Ratio (CBR) value at 2.5 mm penetration ranges from 42.10% to

92.40% and CBR value at 5.0 mm penetration ranges from 52.70 to 89.10% indicative of good materials for road subgrade, sub-base and base courses.

From the work of Adetoro and Michael (2017) on the comparative analyses of performance properties of soils in various local government areas of Ekiti state, it was observed that the study didn't feature any part of Ikole L.G.A. Soils samples collected from the study area were subjected to laboratory tests (i.e. Compaction test). The results of the tests carried out on the soil samples showed that the soil samples of Emure and Ise-Orun LGAs have the highest ranges of Maximum Dry Density (MDD) values and were most suitable for subgrade, subbase and base courses materials, though their moisture content values were higher than the specified values. The soil samples of Efon -Alaaye LGA were all suitable for subgrade course materials and some of them (A and B) were suitable materials for subbase and base courses materials. Though their moisture content were higher than the specified values. The soil samples of Gbonyin, Ido-Osi, Ijero and Oye LGA's were not suitable as subgrade, subbase and base courses materials. Their moisture content values were also higher than the specified values

According to Adebisi *et. al.,* (2014) in their research on the index and strength characteristics of residual lateritic soils from south-western Nigeria, in which the study was undertaken majorly in the cities of Ibadan and Abeokuta, it was observed that the lateritic soils are predominantly well-graded silty, clayey sands with average specific gravity and clay content of 2.72 and 31.8% respectively. The colloidal activity of the clay ranged up to 2.25, signifying potential considerable volume change when wetted and large shrinkage when dried. Kaolinite and Ca-montmorillonite are the principal clay minerals present in the soils. The fines are silty clays of low to medium plasticity. The research revealed a fairly strong inverse relationship between undrained cohesion and flow index, thereby implying that the rate of loss of shearing stress of the soils is comparatively a function of increase in their moisture content.

In the work of Ogundipe (2014), in which he investigated the strength and compaction characteristics of granular soil stabilized with bitumen at 2%, 4% and 6% bitumen content considerations. It was observed that the optimum binder content required in achieving the highest MDD and CBR is 4%. But at 6% bitumen, the MDD and CBR values decreased, although the values obtained were greater than those for unstabilized soil. The reduction in MDD and CBR was due to excess bitumen in the mix which filled the voids, thus resulting in slip and weakening the bond between the aggregates. From the research,it was seen that the properties of the granular soil improved when stabilized with cutback bitumen.

In the work of Ojuri (2012), he worked upon the indirect determination of soil shear strength using basic soil properties for a lateritic soil area. The study area used was the Federal University of Technology, Akure from 6 different locations around the campus. Exploratory analysis was done using statistical analysis of principal components. Predictive modeling for the California Bearing Ratio (CBR) and undrained shear strength ($S_u$) was done via stepwise multiple regression. A quick evaluation of the shear strength of tropical lateritic soils using their maximum dry density and group index values can be done using this model: $S_u = -547.713 + 0.381MDD - 9.1.4GI$

From the work of Owoyemi and Adeyemi (2012), in which samples of foundation soil of an unstable section of the Lagos-Ibadan highway around Sagamu were investigated so as to evaluate them as highway subgrade materials and establish any geotechnical bases for the observed pavement failure in the area, parameters such as the grain size distribution, consistency limits, activity, Maximum Dry Density (MDD), Optimum Moisture Content (OMC), unconfined compressive strength, California Bearing Ratio (CBR) and permeability were determined. Parameters from the two locations were compared with each other and emphasis was placed on comparing the obtained results with existing standard specifications for highway sub-grade soils. The results of the investigations showed that the sub grade soils from both locations have the same degree of laterization and they are poorly graded being rich in fines and deficient in intermediate particles. Casagrande chart classifies soils from both locations as inorganic intermediate plastic soils, while American Association of State Highway and Transportation Officials (AASHTO) classification system groups them as fair to poor sub grade materials. Although, the investigation showed that the predominant clay mineral in both locations is kaolinite, they were found to have plasticity, liquid limits and linear shrinkage values higher than the recommended standard values. It was also discovered that while soils from the locations showed good compaction parameters, they exhibit high water retention capacities and poor drainage characteristics. The CBR (California bearing ratio) of soils from both locations are generally lower than recommended values and were reduced greatly on after soaking. The soils from both locations therefore fall short of recommended standards for good highway sub grade materials.

In the work of Ige *et. al* (2014)- four lateritic soil samples derived from different parent rocks were examined for their suitability as construction materials. All analyses were carried out in accordance with BSI standard. Results showed that soil samples from migmatite gneiss, quartz schist and amphibolites are silty-sands while the granite derived soil is silty-clay. Grain size distribution of the soils showed that they are poorly graded with amount of fines ranging between 28% and 86%. On the basis of this, the OYN soil will be suitable for use as road sub-base materials. The other soils are unsuitable for direct use in road sub-base. The soils, except the OYN soil, have fines >30% which makes them suitable for use as landfill liners. On the basis of the Atterberg consistency limits, all soils meet the requirement for use as sub-grade/fill materials, except OKG and ILS soils. All samples have low to intermediate plasticity. The cohesion values of the soils are between $50kPa$ and $80kPa$ at standard Proctor compaction energy while it is between $30kPa$ and $75kPa$ at modified Proctor compaction energy. Also, the angle of internal friction varies from $11^0$ to $24^0$ and $14^0$ to $24^0$ for standard and modified compaction energies respectively. The coefficients of permeability of the soils fall between $10^{-8}\, m/s$ and $10^{-9}\, m/s$ making them practically impermeable. The mineralogy of the soil showed that they contain no undesirable mineral constituent as they contain mainly quartz. The results of the compaction and CBR showed that the samples are suitable for use as subgrade and fill materials. The grain size distribution values, Atterberg limits and coefficient of permeability of the soils make them suitable for use as liners in waste disposal systems. The angle of internal friction and cohesion of the soils means the soils could also support shallow foundations and could also support moderately steep slopes. This geotechnical information is important for foundation design for future development of the sampled localities.

Ojuri and Ogundipe (2012) from their work studied the characteristics of a lateritic soil by simulating an oil contaminated site by mixing predetermined amounts of used engine oil with lateritic soil samples collected in Akure, south-western Nigeria. Geotechnical testing performed on the studied soils include basic index property tests, compaction tests and strength tests. Soil samples collected from the surrounding of the Engineering Workshop (Machine Shop) building in the Federal University of Technology, Akure were mixed with 0, 2, 4, 6, 8, and 10% of used engine oil by dried weight of the soil. The oil contaminated soils indicated lower Maximum Dry Density (MDD), optimum moisture content (OMC), unconfined compressive strength (UCS) and California Bearing Ratio (CBR) compared to the uncontaminated soil. Regression models for the estimation of compaction and strength characteristics for this type of ferrallitic lateritic soils were established. The MDD for the lateritic soil decreased from $1795\ kg/m^3$ to $1698\ kg/m^3$with increase in used engine oil concentration. The OMC values dropped from 15.3% to 10.9%. The CBR value of the lateritic soil decreased from 22.05 % to 14.45%. The unconfined compressive strength ($q_u$) value for uncontaminated soils was $204\,kN/m^2$which got reduced to $140\,kN/m^2$ at an oil content of 10%. These results reveal that the addition of used engine oil has adverse effects on the compaction and strength characteristics of this lateritic soil. Used engine oil contamination does not just affect the quality of the soil and ground water; it also alters the physical and geotechnical properties of the oil contaminated soil. The high indices of correlation (the coefficient of determination [$R^2$]) for the established relationships between geotechnical characteristics and the used engine oil content, suggest that these expressions are suitable for the determination of the compaction and strength characteristics for similar lateritic soils at different degrees of used engine oil contamination. They recommended proper subsoil investigation for the construction of buildings at sites with history of used oil release into the subsurface environment. Adequate factor of safety should also be used in the design of building foundations at such sites.

From the work of Okunade (2010), in which the study was on stabilization of tropical lateritic soils using self-cementing coal fly ash evaluated the effects of the addition of self-cementing coal fly ash on the engineering properties of three lateritic soils from south-western Nigeria. The engineering properties investigated were those normally involved in highway design and construction. Increasing percentages (by weight of dry soil) of coal fly ash, ranging from 0% through 15% in 2.5% increments, were added and the geotechnical properties assessed. It was observed, for all the soils, that increasing coal fly ash contents brought about increasing improvements in the plasticity and mechanical properties of the soils. When comparing the average value of the properties at 0% coal ash content to their average values at 12.5% coal ash content, there was a reduction in the liquid limits (from 39.0% to 33.3%), a reduction in the plasticity indices (from 15.3% to 9.3%), a reduction in the optimum moisture contents (from 15.8% to 9.7%) accompanied by an increase in the maximum dry densities (from 1920 to $2180kg/m^3$), and an increase in the unsoaked CBR values (from 20.0% to 53.0%). For the stabilization of lateritic soils with coal fly ash, a coal fly ash of 12.5% by weight of dry soil was recommended because the improvements in the soil's properties tapered off at about that percentage of coal ash content. The mixture of coal fly ash with lateritic soils improves the plasticity and mechanical properties of the soil, as expressed by a reduction in the liquid limit and the plasticity index. With regard to the influence of self-cementing fly ash on density and compaction, test results show that fly ash increases the compacted dry density and reduces the optimum moisture content of lateritic soils. Economically, coal fly ash stabilization is much cheaper than stabilization with the conventional material.

Ogundipe (2008) in his work studied the effect of poor soil properties on highway pavement failure on critical locations along Aramoko-Ilesha road, Nigeria. Laboratory soil tests on the sample collected showed the moisture content range of 8.44% to 21.63%, specific gravity from 2.30 to 2.80, liquid limit from 27.30 to 42.50%, linear shrinkage from 8.9% to 9.9%, maximum dry density from 1750 to 2050 $kg/m^3$, optimum moisture content from 10.00 to 18.00% and California Bearing Ratio (C.B.R.) values from 5.45 to 36.64%. The liquid limit and the plasticity index do not conform to specification and the CBR values were far below the values specified for sub-base and base course of road pavement. It was discovered that the use of materials with poor properties was responsible for the failure of these locations. Therefore, it is important that materials that conform to specifications are used in the construction of the road pavements. In some cases, it is advised to follow appropriate correction measures so as to make the materials suitable such as stabilization with bitumen, Portland cement, lime, fly ash and other pozolans. Also the road pavement can be prevented from failing before its design life by providing good drainage so as to prevent ingress of water into the road pavement.

Ige (2009) investigated the geotechnical properties of a migmatite - derived soil from south-western Nigeria for its potential use as barrier in sanitary landfill. The required parameters for soils to be considered as barrier are its' grain size distribution, Atterberg consistency limits, maximum dry density (MDD) and the coefficient of permeability. Results obtained show that the hydraulic conductivity is lower than the suggested limit (1 x $10^{-7}$cm/s) of the various waste regulatory agencies. The hydraulic conductivity value of the liner material is used as the principal indicator of its containment potential. Hydraulic conductivity behaviour of soil barrier is greatly influenced by the particle size distribution because the relative proportions of large and small particle sizes affect the size of voids conducting flow. Barrier soils should have at least 30% Fines and 15% clay to achieve hydraulic conductivity $\leq 1\times10^{-7}\ cm/s$. In addition, it has adequate basic geotechnical properties, strength and the shrinkage potential upon drying. These properties suggest the potential suitability of the soil as a barrier in containment facility for disposal of waste material.

Bello and Adegoke (2010) presented the geotechnical properties of lateritic soil found within Ilesha East Local Government Area, Southwestern Nigeria and environs. The research was achieved by carrying out the following laboratory soil tests: particle size analysis test, Atterberg limit test, British standard light compaction test, specific gravity, and California bearing ratio in accordance with British Standard 1377 (1990) and Head (1992). Grain size analysis shows that the percentages passing No. 200 BS sieve are 69%, 51%, 33%, 34%, 56%, 32%, 50%, and 64% for samples P1, P2, P3, P4, P5, P6, P7, and P8, respectively. The liquid limit ranges between 15.5% and 48.6%, plastic limit ranges between 4.66% and 25.60%, and the plasticity index ranges between 7.17% and 23%. California Bearing Ratio (unsoaked) ranges between 37% and 85%. The specific gravity ranges between 2.61 and 2.80 while the maximum dry density ranges between 2.3 $mg/cm^3$and 2.62 $mg/cm^3$with their optimum moisture contents ranging between 14.5 to 28.0%.According to USCS soil classification, samples P3, P4, and P6 can be classified as well-graded soil thus be considered as good for subgrade and subbase materials. Sample P7 (Ilerin) is considered as a very poor soil thus should not be used as highway construction materials while others are properly graded thus be used as filling materials.

### 2.3 Reviews on methodologies to be employed

#### 2.3.1 Consistency Limits

Chew et. al (2004) examined the relationship between the microstructure and engineering properties (Atterberg limits and unconfined compressive strength among others) of cement – treated marine clay. It has been concluded that the multitude of changes in the properties and behavior of cement – treated marine clay can be explained by four microstructural mechanisms.

#### 2.3.2 Soil Strength

In soils, strength is measured in terms of shear strength. Soils do not generally have much, if any, strength in tension due to the particulate composition of soils. Shear strength in soils is the resistance to shear deformation of the soil mass and is described by internal angle of friction and cohesion. Shear strength in soils results from particle interlocking, particle interference, and sliding resistance (Terzaghi and Peck 1948).

Internal angle of friction ($\varphi$) is a function of mineralogical composition, shape, gradation, void ratio, and organic content of the soil and is measured in degrees (Holtz and Kovacs 1981, Coduto 1999). The contribution of friction angle to the shear strength of a soil is a function of the vertical effective stress at a given point in the soil.

*Triaxial Test*: Day, R. W. (2001) revealed that the triaxial test is one of the most versatile and widely performed geotechnical laboratory tests, allowing the shear strength and stiffness of soil and rock to be determined for use in geotechnical design. He added that the advantages over simpler procedures, such as the direct shear test, include the ability to control specimen drainage and take measurements of pore water pressures. Primary parameters obtained from the test may include the angle of shearing resistance $\phi'$, cohesion $c'$, and undrained shear strength $c_u$, although other parameters such as the shear stiffness G, compression index $C_c$, and permeability k may also be determined. triaxial compression provides strength information at the top of a cut slope, whilst triaxial extension allows parameters for soil elements at the slope base to be determined.

### 2.3.3 Compaction

Hunde's (2003) thesis dealt with the investigation of lateritic soils. He described some properties like the compaction behavior of lateritic soils and he assessed some of the causes of dam failures. He concluded that most of the failures of these dams were due to the less awareness on the properties of the tropical soils and treated when like transported soils. He added that compaction properties of lateritic soils which are tropical soils were highly influenced on the compaction characteristics of these soil.

# CHAPTER THREE

# MATERIALS AND METHODOLOGY

## 3.1 Preliminary Investigations

### 3.1.1 Demographic Study and Site Investigation

This involved learning about the cultural& religious practices of neighbourhoods in Ikole, by engaging in proper dialogue with residents to know the viability of a site to be explored. The high chiefs in the land were contacted by having meetings with them such that the area for exploration is chosen within the scope of the study. Thereafter, topographical maps were gotten from Google Earth Maps as this provided information on the accessibility of the site (of work) and the terrain, both of which exerted a strong influence on the testing method. General characteristics of the soils were commonly revealed by the topography. A reconnaissance survey was then carried out for location and identification of major outcrops around the study area. The positions of the investigation pits were then established after pedological mapping to identify profiles. Four different locations in Ikole-Ekiti were chosen for this study, which are Ilotin (Right side of Ikole Local Government Secretariat), Ijemo-titun (off Ijesa-Isu/ Ikole road), Palace area (Beside First Bank/ NSCDC office) and Asin (Mic-Vic Hotel Area), at which one trial pit shall be dug.

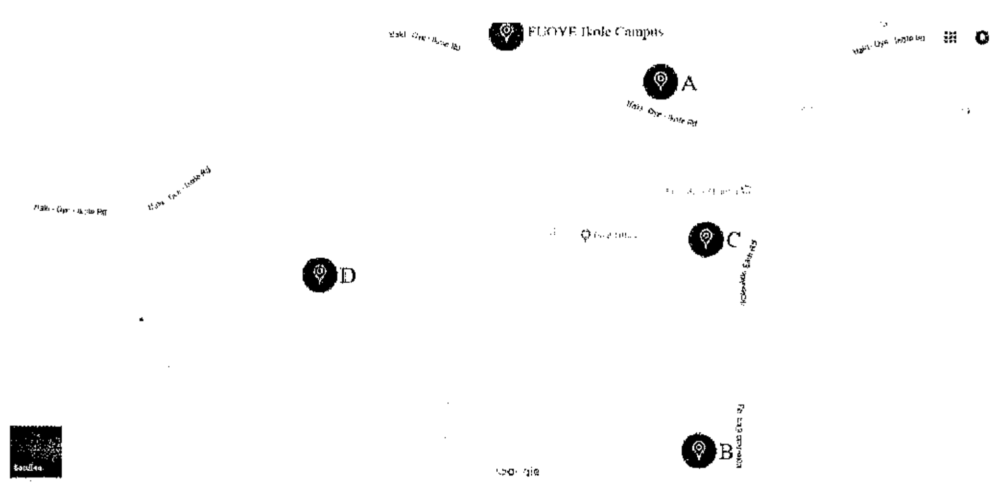



**Figure 3.1**: Map of sample location points (*From Google Earth Maps*)

**Table 3.1: Trial Pits Coordinates and depths explored**

| Sample | Northings Coordinates | Eastings Coordinates | Dug Depth (m) |
|---|---|---|---|
| A | 7.801339 N | 5.506228 E | 1.75 |
| B | 7.773403 N | 5.507344 E | 1.80 |
| C | 7.792115 N | 5.512535 E | 1.95 |
| D | 7.788680 N | 5.485370 E | 1.85 |

### 3.1.2 Field Work and Sample Collection

The scope of field work involves: exploration of four (4) geotechnical trial pits. The trial/ test pit, as an investigation technique, is selected because it provides a very quick and economical method of obtaining substantially reliable samples and information, making it very suitable in profiling the pits. A trial pit is simply a hole dug in the ground that is large enough for a ladder to be inserted, thus permitting a close examination of the sides (Plate 3.1). The samples were then taken to the laboratory where the deleterious materials such as roots were removed. The depth of each trial pit will be 1.8 m (6 ft) and about 4 ft × 4ft wide that is, 1.2m × 1.2 m × 1.8 m pit. Each trial pit was sunk by hand excavation with the aid of spade and a chisel-like digger and excavating the underlying material to a depth of 1.8- 2.0 m. In this process, soil samples were obtained between the depths of 1.7m and 2.0m below ground level. Four trial pits across various points in Ikole township area were chosen.

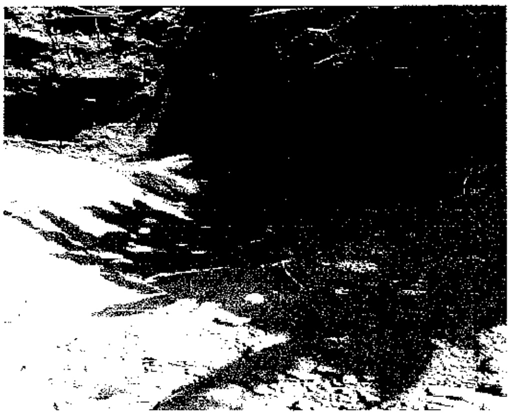


**Plate 3.1**: Site clearance & excavation **Plate 3.2:** Undisturbed block sample at Trial Pit B base

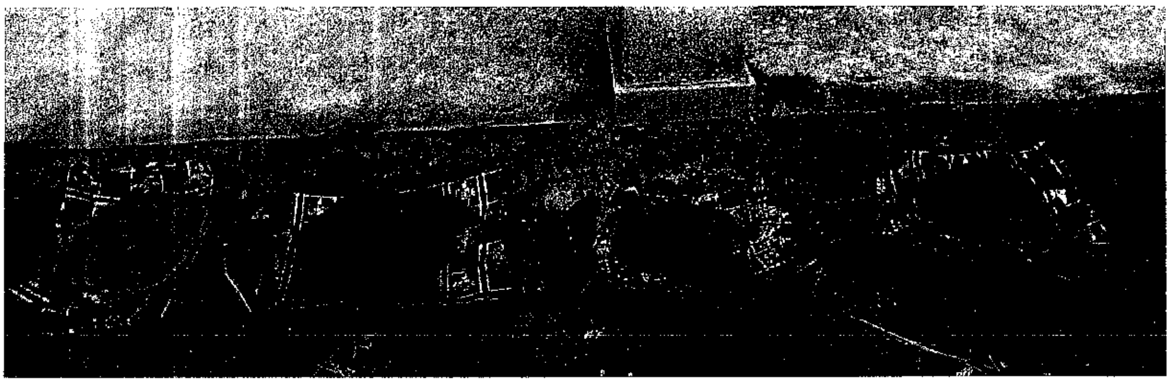


**Plate 3.3:** L- R; Disturbed Samples from Trial pit D, C, B and A on different polythene bags

The type of samples collected for the laboratory analysis were as follows:

(i) Disturbed samples.

(ii) Undisturbed samples (Plate 3.2)

a.) Disturbed Sample: Disturbed samples were used mainly for soil classification and compaction tests. They were to be collected from the trial pits with the use of hand diggers and shovel (Plate 3.3).

b.) Undisturbed Sample: Undisturbed samples were required for strength tests on the soil. During cutting and trimming, the samples are protected from water, wind, and sun to avoid any change in water content. The samples were covered with black polythene bag immediately they were brought to the surface, and the samples were carefully labelled, respectively. The block undisturbed samples collected had a size of about 225 mm×225 mm× 225 mm.

### 3.1.3 Moisture Content

The moisture content of a soil is assumed to be the amount of water within the pore space between the soil grains which is removed by oven-drying at a temperature not exceeding$110^oC$. The moisture content has a profound effect on soil behavior. The oven-drying method is regarded as standard laboratory practice. The moisture content is expressed as a percentage of the mass of dry soil.

**Equipment Required**

1. A drying oven with temperature of $105^oC$ to $110^oC$. (Plate 3.4)

2. A balance readable to 0.1g.

3. A metal container.

4. A dessicator.

**Procedure for Moisture content determination**

1. The container was cleaned and dried, its weight was then recorded to the nearest 0.1g ($m_1$).

2. A representative sample was crumbled and loosely packed in the container.

(i) For fine-grained soils the sample weight shall be minimum 30g.

(ii) For medium-grained soils the sample weight shall be minimum 300g.

(iii) For coarse-grained soils the sample weight shall be minimum 3kg.

3. The container with sample was immediately weighed ($m_2$) and placed in the oven to dry at $105^0C$ for minimum 12 hours.

4. After drying, the weight of the container and the contents ($m_3$) was determined.

$$Moisture\ content, w\ (\%) = \frac{m_2 - m_3}{m_3 - m_1}\ X\ 100\ (\%) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots . (Eq\ 3.1)$$

### 3.1.4 Atterberg Limits

The **Atterberg limits** are a basic measure of the critical water contents of a fine-grained soil: its **shrinkage limit, plastic limit**, and **liquid limit**. As a dry, clayey soil takes on increasing amounts of water, it undergoes distinct changes in behavior and consistency. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of a soil are different and consequently so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behavior. The Atterberg limits can be used to distinguish between silt and clay, and to distinguish between different types of silts and clays.

Distinctions in soil are used in assessing the soils that are to have structures built on them. Soils when wet retain water, and some expand in volume. The amount of expansion is related to the ability of the soil to take in water and its structural make-up (the type of atoms present). These tests are mainly used on clayey or silty soils since these are the soils that expand and shrink due to moisture content. Clays and silts react with the water and thus change sizes and have varying shear strengths. Thus, these tests are used widely in the preliminary stages of designing any structure to ensure that the soil will have the correct amount of shear strength and not too much change in volume as it expands and shrinks with different moisture contents.

As a hard, rigid solid in the dry state, soil becomes a crumbly (friable) semisolid when a certain moisture content, termed the shrinkage limit, is reached. If it is an expansive soil, this soil will also begin to swell in volume as this moisture content is exceeded. Increasing the water content beyond the soil's plastic limit will transform it into a malleable, plastic mass, which causes additional swelling. The soil will remain in this plastic state until its liquid limit is exceeded, which causes it to transform into a viscous liquid that flows when jarred.

**a.) <u>Liquid Limit</u>:** The liquid limit is the empirically established moisture content at which a soil passes from the liquid state to the plastic state. The liquid limit provides a means of identifying and classifying fine-grained cohesive soils especially when also the plastic limit is known. Variations in the moisture content in a soil may have significant effect on its shear strength, especially on fine-grained soils. The cone penetrometer method is the preferred method to the Casagrande test as it is essentially a static test depending on soil shear strength. The test method covers the determination of the liquid limit of a sample in its natural state, or a sample from which material retained on a 425 mm test sieve has been removed. It is based on the measurement of penetration into the soil of a standardized cone.

**Equipment Required for Liquid Limit**

(1) Test sieves of sizes 425 μm.

(2) An airtight container.

(3) A flat glass plate.

(4) Two palette knives or spatulas.

(5) A penetrometer.

(6) A cone of stainless steel, 35 mm long with a smooth, polished surface and an angle of $30^0$ having a mass of 80 g.

(7) A metal cup 55 mm in diameter and 40 mm deep with the rim parallel to the flat base.

(8) An evaporating dish or a damp cloth.

(9) Apparatus for moisture content determination.

(10) A wash bottle containing clean water.

(11) A metal straight edge.

(12) A stopwatch.

**Sample Preparation for Liquid Limit**

1. A sample of the soil of sufficient size was taken to give a test specimen weighing about 400 g which passes the 425mm sieve. This was enough material for both Plastic Limit and Linear Shrinkage tests in addition to the Liquid Limit Test.

2. The soil was transferred to a glass plate, water was added and it was mixed thoroughly with two palette knives until the mass became a thick homogenous paste.

3. The paste was placed in an airtight container and allowed to stand for 16-24 hours to enable the water permeate through the soil.

**Procedure for Liquid Limit**

1. The 400 g soil sample was taken and placed on a glass plate. The paste was mixed for at least 10 minutes using the two palette knives. (More distilled water was added if necessary, so that the first cone penetrometer reading was about 15 mm).

2. A portion of the mixed soil was pushed into the cup with a palette knife, taking care not to trap air, the cup was gently tapped against a firm surface if necessary. Excess soil was struck off with the straight edge to give a smooth level surface.

3. With the penetration cone locked in the raised position, the cone was lowered so that it just touches the surface of the soil. When the cone is in the correct position, a slight movement of cup will just mark the soil surface. The dial gauge was lowered to come in contact with the cone shaft and the reading of the dial gauge was recorded to the nearest 0.1 mm.

4. The cone was released for a period of 5±1 sec. After locking the cone in position, the dial gauge was lowered to contact the cone shaft and the reading of the dial gauge was recorded to the nearest 0.1 mm.

5. The cone was lifted out and cleaned carefully.

6. A little more wet soil was added to the cup and the process was repeated. Whenever the difference between the first and second penetration readings was less than 0.5 mm, the average of the two penetrations shall be recorded. However, whenever the second penetration was more than 0.5 mm and less than 1 mm different from the first, a third test was carried out. Whenever the overall range was not more than 1 mm, the average of the 3 penetrations was recorded. But, if the overall range is more than 1 mm, the soil was removed from the cup, remixed and the testd were repeated until consistent results were obtained.

7. The moisture content of a sample of about 20 g from the area penetrated by the cone was determined.

8. The penetration test was repeated at least three more times using the same sample of soil to which further increments of water have been added. The amount of water added was such that a range of penetration values of approximately 15 mm to 25 mm is covered by the four test runs.

9. Each time soil is removed from the cup for addition of water, the cup was washed and dried.

**Calculations**

1. The moisture content of each specimen is calculated by

$$w = \left(\frac{m_2 - m_3}{m_3 - m_1}\right) \times 100\ (\%)$$

Where,

$m_1$ is the mass of container (in g)

$m_2$ is the mass of the container and wet soil (in g)

$m_3$ is the mass of the container and dry soil (in g)

2. The relationship between the moisture content and cone penetration is to be plotted, with the moisture content as the abscissae and the cone penetration as ordinates, both on linear scales.

3. The best straight line fitting the points is then to be drawn.

4. The liquid limit ($w_L$) of the soil sample is the moisture content corresponding to a cone penetration of 20 mm and shall be expressed to the nearest whole number.

**b.) <u>Plastic Limit and Plasticity Index</u>:** The plastic limit is the empirically established moisture content at which a soil becomes too dry to be plastic. It is used together with the Liquid limit to determine the plasticity index which when plotted against the liquid limit on the plasticity chart provides a means of classifying cohesive soils. The plasticity index is the difference between the liquid limit and the plastic limit. The plasticity index, is the range of moisture content in which a soil is plastic; the finer the soil, the greater the Plasticity index. This test method covers the determination of the Liquid limit of a sample in its natural state, or a sample from which material retained on a 425 μm test sieve.

This test commonly is performed as a continuance of the Liquid Limit test, and material for the test could conveniently be prepared as part of the Liquid Limit test. Otherwise, a 40 g sample should be prepared in the same way as specified for the Liquid Limit test.

**Equipment Required for Plastic Limit**

(1) Two flat glass plates, one for mixing soil and one for rolling threads.

(2) Two palette knives or spatulas

(3) Apparatus for moisture content determination

(4) Clean water

(5) A length of rod, 3 mm in diameter and 100 mm long.

**Procedure for Plastic Limit**

1. 40 g soil paste sample was taken and placed on a glass plate.

2. The soil was allowed to dry partially until it became plastic enough to be shaped into a ball.

3. The ball of soil was moulded between the fingers and rolled between the palms of the hands until the heat of the hands dried the soil sufficiently for slight cracks to appear on its surface.

4. The sample was divided into 2 sub-samples of about 20 g each and separate determinations were carried out on each portion. (Each of the 2 sub-samples were divided into 4 more or less equal parts).

5. The soil in the fingers was moulded to equalize the distribution of moisture. A thread of soil about 6 mm diameter was then formed between the first finger and thumb of each hand.

6. The thread was rolled between the fingers, from finger-tip to the second joint, of one hand and the surface of the glass plate. Enough pressure was used to reduce the diameter of the thread to about 3 mm in 5 to 10 complete, forward and back, movements of the hand.

7. The soil was picked up, moulded between the fingers to dry it further, formed into a thread and rolled out again as specified above.

8. The procedure was repeated until the thread sheared both longitudinally and transversely when rolled to about 3 mm diameter. The metal rod was used to gauge the diameter. The first crumbling point is the Plastic Limit.

9. The pieces of the crumbled soil thread were gathered together and transferred to a suitable container for determining the moisture content and the lid was replaced immediately.

10. The Rolling procedure was repeated on the other 3 portions of the sub-sample, by placing them all in the same container for determining the moisture content.

11. The rolling procedure was repeated on the $2^{nd}$ sub-sample as described above so that 2 completely separate determinations were made.

**Calculations for Plastic Limit**

1. The moisture content of both samples was calculated. (Whenever the 2 results differ by more than 0.5% moisture content, the whole test was repeated).

2. The average of the 2 moisture content was calculated and the value was expressed to the nearest whole number. This value is the Plastic Limit ($w_P$).

**c.) <u>Shrinkage Limit</u>:** Shrinkage due to drying is significant in clays, but less so in silts and sands. If the drying process is prolonged after the plastic limit has been reached, the soil will continue to decrease in volume, which is also relevant to the converse condition of expansion due to wetting. The Linear Shrinkage value is a way of quantifying the amount of shrinkage likely to be experienced by clayey material. Such a value is also relevant to the converse condition of expansion due to wetting. Linear Shrinkage method covers the determination of the total linear shrinkage from linear measurements on a bar of soil of the fraction of a soil sample passing a 425 μm test sieve, originally having the moisture content of the Liquid Limit.

This test commonly is performed as a continuance of the Liquid Limit and Plastic Limit test, and material for the test could therefore conveniently be prepared as part of the Liquid Limit test. Otherwise, a 150 g sample should be prepared in the same way as specified for the Liquid Limit test. A sample of material passing through a 425 μm sieve, or alternatively a sample of natural soil without coarse particles, shall be thoroughly mixed with distilled water until the mass becomes a smooth homogenous paste with a moisture content at about the Liquid Limit of the soil.

**Equipment Required for Shrinkage Limit**

1. A flat glass plate

2. Two palette knives or spatulas

3. A drying oven capable of maintaining temperature of $105^oC$ to $110^oC$.

4. Clean water

5. A brass mould for Linear Shrinkage test.

6. Silicone grease or petroleum jelly.

7. Vernier calipers or steel rule with accuracy 0.5 mm.

**Procedure for Shrinkage Limit**

1. The mould was cleaned thoroughly and a thin film of silicone grease or petroleum jelly was applied to its inner faces to prevent the soil adhering to the mould.

2. 150g soil paste sample was taken at approximately the Liquid Limit.

3. The soil/ water mixture was placed in the mould such that it was slightly proud of the sides of the mould. The mould was carefully tapped against a firm surface, to remove air pockets in the mixture.

4. The soil was levelled along the top of the mould with the palette knife and all soil adhering to the rim of the mould was removed by wiping with a damp cloth.

5. The mould was placed where the paste could air dry slowly for 1– 2 days until the soil shrunk away from the walls of the mould.

6. The drying was completed at $105^oC$ to $110^oC$.

7. The mould was cooled and the mean length of the soil bar was measured by pressing it against the end of the mould where there was a better fit, while measuring the distance between the opposite side of the mould and the soil bar.

**Calculations for Shrinkage Limit**

The linear shrinkage of the soil is a percentage of the original length of the specimen, $L_O$ (in mm), from the equation:

$$Percentage\ of\ Linear\ Shrinkage = \left(1 - \frac{L_D}{L_O}\right)100 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(Eq\ 3.2)$$

Where $L_D$ is the length of the oven-dry specimen (in mm)

The Linear shrinkage of the soil is to be reported to the nearest whole percentage.

### 3.1.5 Particle Size Analysis

The disturbed sample was dried, weighed and sieved through a series of standard sieves. The portion retained on each sieve is was weighed and the percentage passing each sieve was calculated. This method of dry sieving is detailed in BS 1377.

**Equipment Required for Particle Size Analysis**

1. A balance readable to 0.1g.

2. Sample dividers.

3. A drying oven with temperature of $105^oC$ to $110^oC$.

4. A tray.

5. A scoop.

6. Sieve brushes.

7. A mechanical sieve shaker. (Plate 3.5)

**Procedure for particle size analysis by dry sieving**

1. The oven-dried sample was weighed to 0.1 % of its total mass (m).

2. The largest size was fitted to the test sieve appropriate to the maximum size of material present to the receiver and the sample was placed on the sieve.

3. The test sieve was agitated such that the sample rolls in an irregular motion over the test sieve. The particles were hand placed to see if they would fall through but were not pushed through. Only individual particles were ensured to be retained. The amount retained on the test sieve was weighed to 0.1 % of its total mass.

### 3.1.6 Specific Gravity / Particle Density

Particle density is the term used **Specific Gravity** instead of specific gravity of particles. Knowledge of the particle density is essential in relation to other test, especially for calculating porosity and voids and for computation of particle size analysis from a sedimentation procedure (Hydrometer analysis). It is also important when compaction and consolidation properties are considered.

The specific gravity is then determined using the pycnometer method. The small pycnometer method is suitable for soils consisting of particles finer than 2mm. Larger particles may be ground down to smaller than 2mm testing. The sample is then to be oven-dried at $105^oC$ and is to be weighed in the pycnometer with and without water. The test is to be carried out in accordance with BS 1377.

**Equipment required for Specific Gravity**

1. 1000mL density bottles (pycnometer) designed for a screw-top lid, fitted with the following:

(i) A corrosion-resistant screw ring;

(ii) A conical camp of corrosion-resistant metal with a cone angle of $75^o$ to $78^o$ and with a hole 6 ± 0.5 mm diameter at its apex;

(iii) A rubber or fibre sealing washer to ensure that the crew top is watertight when screwed on to the jar.

2. Constant temperature water bath.

3. Vacuum desiccator.

4. A drying oven with temperature of $105^oC$ to $110^oC$.

5. A glass rod about 300 mm long and 6 mm diameter.

6. A balance readable to 0.1g.

7. A thermometer to cover the temperature range $0^oC$ to $50^oC$, readable to $1^oC$.

**Procedure for determination of specific gravity**

1. The pycnometer was cleaned and dried and the whole assembly was weighed to the nearest 0.5g ($m_1$)

2. With the screw top removed, the first specimen was transferred from its sealed container directly into the jar. The jar and contents and the screw-top assembly was weighed to the nearest 0.5g ($m_2$).

3. Water was added at a temperature of within $\pm\ 2^oC$ of the average room temperature during the test to about half fill the jar. The mixture was stirred thoroughly with the glass rod to remove air trapped in the soil.

4. The screw cap assembly was fitted and tightened so that the reference marks coincide. The pycnometer was filled with water.

5. The pycnometer was agitated by shaking or by rolling on the bench, while holding one finger over the hole in the conical top. Air was allowed to escape so as to disperse. The pycnometer was left standing for at least 24 hours at room temperature constant to within $\pm\ 2^oC$. (Plate 3.6)

6. The pycnometer was topped with water so that the water surface was flushed with the hole in the conical cap. Air bubbles were taken care off by not been trapped under the cap.

7. The pycnometer was dried on the outside and the whole assembly was weighed to the nearest 0.5 g ($m_3$).

8. The pycnometer was emptied, washed thoroughly and completely filled with water at room temperature. The reference marks on the screw cap were ensured to coincide, that no air bubbles were entrapped and the water surface was flushed with the hole in the conical cap. (Plate 3.7)

9. The pycnometer was dried on the outside and weighed to the nearest 0.5 g ($m_4$).

10. Procedures 1 to 9 was repeated using the second specimen of the same soil so that two values of particle density were be obtained. The specific gravity was then to be calculated as shown below:

$$S.G = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_2)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (Eq\ 3.3)$$

Where,

$m_1$ . . . $Weight\ of\ empty\ bottle\ (Pycnometer)$

$m_2$ . . . $Weight\ of\ bottle + dry\ soil$

$m_3$ . . . $Weight\ of\ bottle + dry\ soil + water$

$m_4$ . . . $Weight\ of\ bottle + water$

## 3.2 Main Investigations

### 3.2.1 Compaction Tests

The dry density which can be achieved for a soil depends upon the degree of compaction applied and the moisture content. The moisture content which gives the highest dry density is called the optimum moisture content for that type of compaction. In general the optimum moisture content is less than the Plastic Limit. The compaction characteristics of a soil can be assessed by means of standard laboratory tests, such as standard proctor test, modified proctor test and west African method. The tests selected for this study were the standard proctor test and modified proctor test.

In the standard proctor test, the soil was mixed with varying amounts of water and then compacted in three equal layers by a 2.5kg rammer which had a drop of 30.5cm that delivered 25 blows to each layer. While for the modified proctor test, the soil is compacted in five layers by a 4.5kg rammer which had a drop of 45.7 cm that delivered 25 blows to each layer. The objective of these tests was to obtain relationships between compacted dry density and soil moisture content. The test was used to provide a guide for specifications on field compaction.

**Equipment Required for Compaction Tests**

1. A cylindrical compaction mould with internal diameter of 105 mm and internal height of 115 mm and a volume of 1.0 L ($1000cm^3$)- The mould shall be fitted with a detachable baseplate and a removable extension (collar) approximately 50 mm height. (Plate 3.8))

2. Subsidiary mould (CBR mould), diameter 152 mm, height 127 mm.

3. A metal rammer having a 50 mm diameter circular face and weighing 2.5Kg. The rammer shall be equipped with an arrangement for controlling the height of drop to 300 mm. (Plate 3.9)

4. A weighing balance readable to 1g.

5. Palette knives or spatulas.

6. A straight edge, e.g. a steel strip.

7. A 20 mm and 37.5 mm test sieves and receiver.

8. A container suitable for mixing the quantity of material to be used.

9. Water proof containers and scoop.

10. A large metal tray.

11. Measuring cylinder, 200 ml or 500 ml.

12. Suitable tools for extracting specimen from mould.

13. Apparatus for moisture content determination.

**Preparation of Sample for Compaction Tests**

The method of preparation of samples for these tests, and the quantity of soil required, depended on the size of the largest particles present and on whether or not the soil particles were susceptible to crushing during compaction.

For soils containing particles not susceptible to crushing, one sample was required for test and it was used several times after progressively increasing the amount of water.

For soils containing particles that were susceptible to crushing, it was necessary to prepare separate batches of soil at different moisture contents, each for compacting once only, otherwise the characteristics of the material will progressively change after each application of compaction. Consequently, a much larger sample was required.

For stiff cohesive soils, which need to be shredded or chopped into small lumps, the result of a compaction test depends on the size of the resulting pieces. Furthermore, the densities obtained in the test will not necessarily be directly related to densities obtained in-situ.. Suggested methods are to shred the soil so that it could pass through a 5 mm test sieve, or to chop it into pieces, e.g. to pass a 20 mm test sieve.

*Preliminary assessment*: The initial soil sample for testing shall be obtained in accordance with the procedure described in Sections 7.6.1 to 7.6.3 of BS 1377-1:1990. The procedures to be used for sample preparation and for carrying out the compaction test shall be selected on the basis of the following assessment.

1. The soil particles needed to be ascertained whether they were susceptible or not to crushing during compaction. To erase doubt, I assumed that they are susceptible. [The soil were considered susceptible to crushing during compaction since the sample contains granular material of a soft nature, e.g. soft limestone, sandstone, etc., which would be reduced in size by the action of the 2.5 kg rammer].

2. The approximate percentages were determined (to an accuracy of ±5 %) by mass of particles in the soil sample passing the 20 mm and 37.5 mm test sieves. The material used for this assessment was not dried, and the dry mass of soil finer than 20 mm was determined by measuring the moisture content using a representative portion. Since enough soil was

available to meet the requirements of Clause 9 of BS 1377-2:1990 a separate sample was used for this sieving operation.

3. On the basis of these percentages the soil was assigned to one of the grading zones (1) to (5) in Table 2 of BS 1377-2:1990, which are also shown diagrammatically in Figure 1 of BS 1377-2:1990. If a grading curve passes through more than one zone the highest-numbered zone applies. A soil with a grading curve passing through zone X is not suitable for these tests.

**1.** Preparation of soils not susceptible to crushing for compaction in 1L mould

**A.** *Grading zone (1) for soils passing the 20 mm test sieve.*

(i) The initial sample was prepared and subdivided by the procedures described in clause 7.6 of BS 1377-1:1990 to produce a representative sample of about 6 kg of the soil.

(ii) Suitable amounts of water was added depending on the soil type and thoroughly mixed.

(iii) The amount of water to be mixed with soil at the commencement of the test varied with the type of soil under test. In general, with sandy and gravelly soils a moisture content of 4 % to 6 % below the plastic limit of the soil was suitable for this research.

(iv) It was important that the water was mixed thoroughly and adequately with the soil, since inadequate mixing could give rise to variable test results. This was particularly important with cohesive soils when adding a substantial quantity of water. With clays of high plasticity, or where hand mixing is used, storage of the mixed sample in a sealed container for a minimum period of 24 hours before continuing with the test was the most satisfactory way of distributing the water uniformly.

(iv) The soil initially contained too much water, so it was allowed to partially air dry to the lowest moisture content at which the soil was to be compacted, and mixed thoroughly.

**B.** *Grading zone (2) for soils passing the 37.5 mm test sieve with at least 95 % passing the 20 mm test sieve*

(i) The whole sample was weighed to 0.1 % by mass and it was recorded.

(ii) The material retained on the 20 mm test sieve was removed and weighed to 0.1 % by mass.

(iii) The removal of small amounts of stone (up to 5 %) retained on a 20 mm test sieve was likely to affect the density obtainable only by amounts comparable with the experimental error involved in measuring the maximum dry density.

(iv) The finer material was subdivided and proceeded as described in part **A.**

**2.** Preparation soils susceptible to crushing for compaction in 1L mould

A. *Grading zone (1) for soils passing the 20 mm test sieve.*

(i) The initial sample was subdivided to produce five or more representative samples, each of about 2.5 kg, using the procedure described in clause 7.6 of BS 1377-1:1990.

(ii) Each sample was mixed thoroughly with a different amount of water to give suitable ranges of moisture contents. The range of moisture contents was such that at least two values lie either side of the optimum at which the maximum dry density occurs.

(iii) The water added to each sample was such that a range of moisture contents was obtained which includes the optimum moisture content. In general, increments of 1 % to 2 % are suitable for sandy and gravelly soils and of 2 % to 4% for cohesive soils. To increase the accuracy of the test it was desirable to prepare samples with smaller increments of water in the region of the optimum moisture content.

(iv) When the soil initially contained more water than it was required for the compaction at the lower moisture contents, I allowed these samples to partially dry to the desired moisture contents, and mixed thoroughly.

(v) Whenever the soil was found to be cohesive, each sample was sealed in an airtight container and stored for at least 24 hours.

**B.** *Grading zone (2) for soils passing the 37.5 mm test sieve with at least 95 % passing the 20 mm test sieve*

(i) The whole sample was weighed and the mass was recorded.

(ii) The material retained on the 20 mm test sieve was removed and discarded.

(iii) The finer material was subdivided, and proceeded as described in part **A.**

**Procedure for Compaction Tests**

A. *Compaction procedure for soil particles not susceptible to crushing*

1. The mould with the baseplate attached was weighed to an accuracy of 1g ($m_1$). The internal dimensions were measured to an accuracy 0.1 mm.

2. The extension was attached to the mould and the mould assembly was placed on a solid base, e.g. a concrete floor.

3. A quantity of moist soil was placed in the mould such that when compacted it occupied a little over one-third of the height of the mould body.

4. 27 blows were applied from the rammer dropped from a height of 300 mm above the soil as controlled by the guide tube. The blows were uniformly distributed over the surface and the rammer was ensured to always fall freely and wasn't obstructed by soil in the guide tube.

5. Steps **3** and **4** was repeated twice more, such that the amount of soil used was sufficient to fill the mould body, with the surface not more than 6 mm proud of the upper edge of the mould body.

6. The extension was removed, the excess soil was struck off and leveled off the surface of the compacted soil carefully to the top of the mould using the straightedge. Any coarse particles removed in the levelling process was replaced by finer material from the sample, well pressed in.

7. The soil and mould with baseplate attached was weighed to an accuracy of 1g ($m_2$).

8. The compacted soil was removed from the mould and placed on the metal tray. A representative sample of the soil was taken for determination of its moisture content as described in **3.1.3.**

9. The remainder of the soil was broken up, rubbed it through the 20 mm test sieve and mixed with the remainder of the prepared test sample.

10. Suitable increments of water were added and mixed thoroughly into the soil.

11. Steps **3** to steps **10** were repeated to give a total of at least five determinations. The moisture contents was such that the optimum moisture content, at which the maximum dry density occurs, lies near the middle of the range.

B. *Compaction procedure for soil particles susceptible to crushing*

1. The mould was weighed, measured and prepared as described in part A.1 and part A.2.

2. Compaction test was carried out on each of the prepared samples as described in part A.

3. The remainder of each compacted sample was discarded.

**Calculations, plotting and expression of results for Compaction Tests**

1. The internal volume was calculated, $V$ (in $cm^3$) of the mould.

2. The bulk density, $\rho$ (in $g/cm^3$), of each compacted specimen was calculated from the equation

$$\rho = \frac{m_2 - m_1}{V} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (Eq\ 3.4)$$

Where,

$m_1$ is the mass of mould and baseplate (in g);

$m_2$ is the mass of mould, baseplate and compacted soil (in g).

3. The dry density, $\rho_d$ (in $g/cm^3$), of each compacted specimen was calculated from the equation

$$\rho_d = \frac{\rho}{1 + \omega} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (Eq\ 3.5)$$

Where$\rho_d = dry\ density, \rho = Bulk\ density$ aand $\omega = water\ content$

A representative sample of the specimen was taken and the moisture content determined. From the graph of the dry density against moisture content, the maximum dry density (MDD) and optimum moisture content (OMC) will be determined. The test procedure is described in BS 1377.

### 3.2.2 Triaxial Test

The triaxial compression test, introduced by Casagrande and Terzaghi in 1936, is by far the most popular and extensively used shearing strength test, both for field application and for purposes of research. As the name itself suggests, the soil specimen is subjected to three compressive stresses in mutually perpendicular directions, one of the three stresses being increased until the specimen fails in shear. Its great advantage is that the plane of shear failure is not predetermined as in the shear box test.

In the triaxial test, three identical samples of soils are subjected to uniformly-distributed fluid pressure around the cylindrical surface (Plate 3.10). The sample is sealed in a watertight rubber membrane. Then axial load is applied to the soil sample until it fails (Plate 3.11). Although only compressive load is applied to the soil sample, it fails by shear on internal faces (Murthy, 2000).

Usually a cylindrical specimen with a height equal to twice its diameter is used. The desired three-dimensional stress system is achieved by an initial application of all-round fluid pressure or confining pressure through water. While this confining pressure is kept constant throughout the test, axial or vertical loading is increased gradually and at a uniform rate. The axial stress thus constitutes the major principal stress and the confining pressure acts in the other two principal directions, the intermediate and minor principal stresses being equal to the confining pressure.

There are three versions to this: Unconsolidated Un-drained (UU) shear test, Consolidated Un-drained (CU) shear test and Consolidated Drained (CD) shear test. For the Unconsolidated Un-drained (UU) test, there is no permission of water drainage during the entire test. Excess pore pressures (either positive or negative) will result from the application of both normal and shear stresses. For the consolidated un-drained (CU) shear test, drainage is permitted and full primary consolidation is allowed to take place under the initially applied normal stresses only. There is no drainage permitted during subsequent applications of either normal or shear stresses. For the consolidated drained (CD) shear test, drainage is permitted during the application of both normal and shearing stresses. No excess pore pressures (either positive or negative) are allowed to develop at any time during the test.

From the triaxial test data, it was possible to extract fundamental material parameters about the sample, including its angle of shearing resistance, apparent cohesion, and dilatancy angle. During the shearing, a granular material will typically have a net gain or loss of volume. If it had originally been in a dense state, then it typically gains volume, a characteristic known as Reynolds' dilatancy. If it had originally been in a very loose state, then contraction may occur before the shearing begins or in conjunction with the shearing. For this research, consolidated drained test shall be employed.

### 3.2.3 Direct Shear Test

The specimen, of cross-sectional area A, is confined in a metal box (known as the shear box or direct shear apparatus, {Plate 3.12}) of square or circular cross- section split horizontally at mid- height, a small clearance being maintained between the two halves of the box. At failure within an element of soil under principal stresses $\sigma'_1$ and $\sigma'_3$, a slip plane will form within the element at an angle $\theta$. The shearbox was designed to represent the stress conditions along this slip plane. Porous plates were placed below and on top of the specimen if it is fully or partially saturated to allow free drainage: if the specimen is dry, solid metal plates may be used {Plate 3.13}. A vertical force (N) is applied to the specimen through a loading plate, under which the sample is allowed to consolidate i.e., Vertical load was applied to the sample and was held constant during the test. The vertical load divided by the area of the sample gives the applied vertical stress. Shear displacement was then gradually applied on a horizontal plane by causing the two halves of the box to move relative to each other, the shear force required (T) being measured together with the corresponding shear displacement ($\Delta l$) with the aid of a load transducer which is connected to a computer. The induced shear stress within the sample on the slip plane is equal to that required to shear the two halves of the box. The test was repeated with a few more samples having the same initial conditions as the first sample. Each sample was tested with a different vertical load. The Direct shear apparatus can be used for both drained and undrained tests.

The advantages of the direct shear test over other shear tests are the simplicity of setup and equipment used, and the ability to test under differing saturation, drainage, and consolidation conditions. These advantages were weighed against the difficulty of measuring pore-water pressure when testing in undrained conditions, and possible spuriously high results from forcing the failure plane to occur in a specific location.

### 3.2.4 Unconfined Compression Strength Test

The primary purpose of the Unconfined Compression Strength (UCS) Test was to quickly determine a measure of the unconfined compressive strength of rocks or fine-grained soils that possess sufficient cohesion to permit testing in the unconfined state. This measure was then used to calculate the unconsolidated un-drained shear strength of the soil under unconfined conditions. In general, The UCS test can be conducted on rock samples or on undisturbed, reconstituted or compacted cohesive soil sample.

In the unconfined compression test, the sample was placed in the loading machine between the lower and upper plates. Before starting the loading, the upper plate is adjusted to be in contact with the sample and the deformation is set as zero. The test then starts by applying a constant axial strain of about 0.5 to 2% per minute. The load and deformation values are recorded as needed for obtaining a reasonably complete load-deformation curve. The loading is continued until the load values decrease or remain constant with increasing strain, or until reaching 20% (sometimes 15%) axial strain. At this state, the samples is considered to be at failure. The sample is then removed for measurement of the water content as in **3.1.3**. As for the results, the axial stress was plotted versus the axial strain. The maximum axial stress, or the axial stress at 20% (sometimes 15%) axial strain if it occurs earlier, is reported as the unconfined compressive strength$\sigma_c$. The undrained shear strength then reads:

$$S_u = \frac{\sigma_c}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (Eq\ 3.6)$$

$$S_u = Undrained\ shear\ strength$$

$$\sigma_c = Unconfined\ compressive\ strength$$

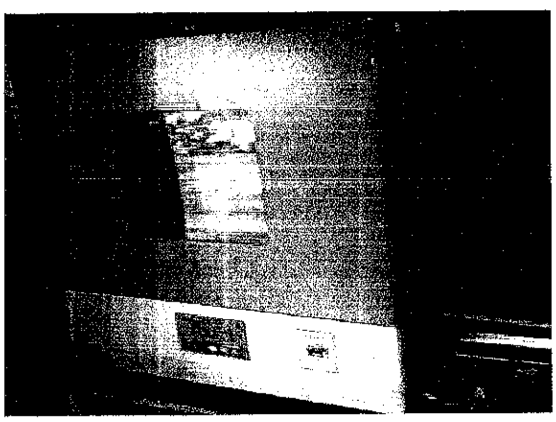


**Plate 3.4**: Thermostatically controlled oven

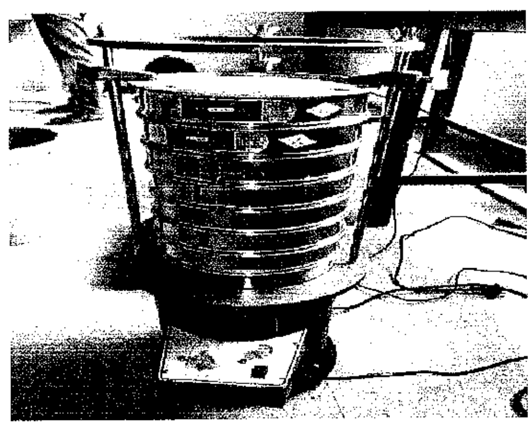


**Plate 3.5:** Mechanical sieve shaker

**Plate 3.6**: Pycnometer filled with soil solution

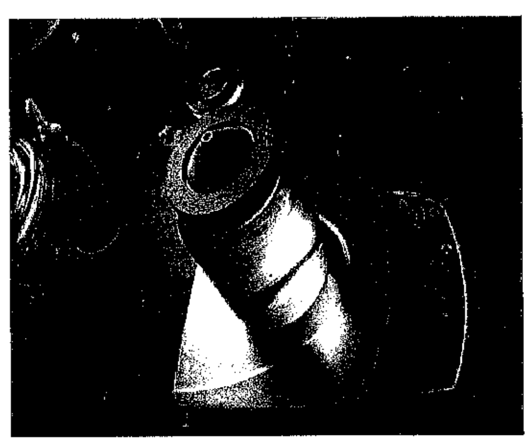


**Plate 3.7**: Pycnometer filled with water

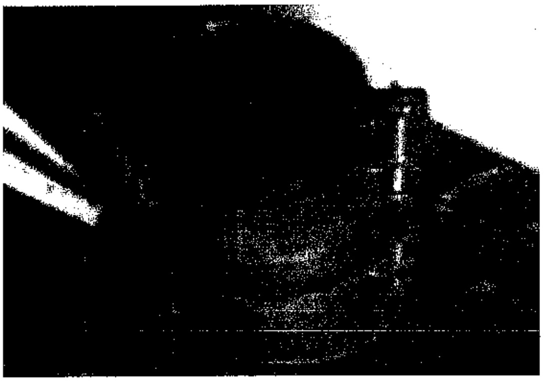


**Plate 3.8**: Compaction mould

**Plate 3.9**: 2.5kg and 4.5kg Rammers

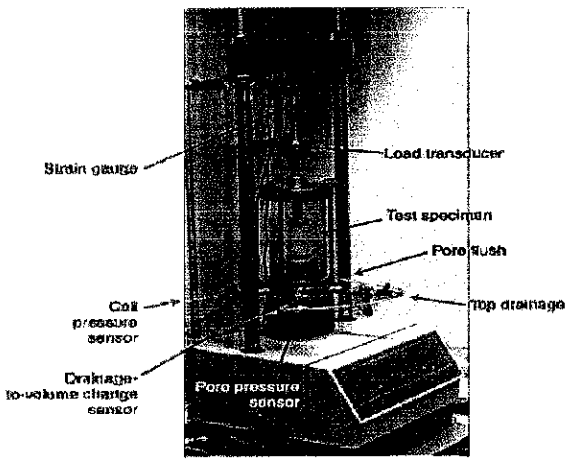



**Plate 3.10**: Labelled Triaxial test apparatus

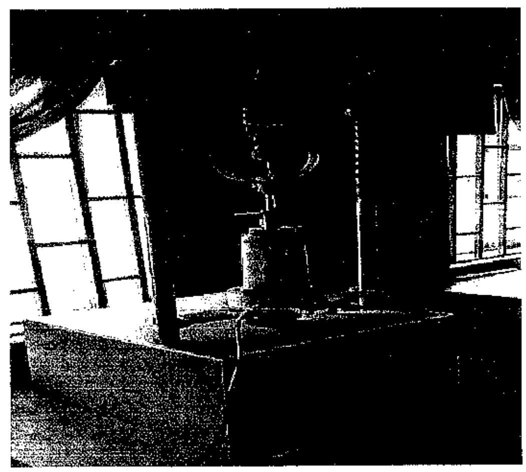


**Plate 3.11**: Laboratory Triaxial test apparatus

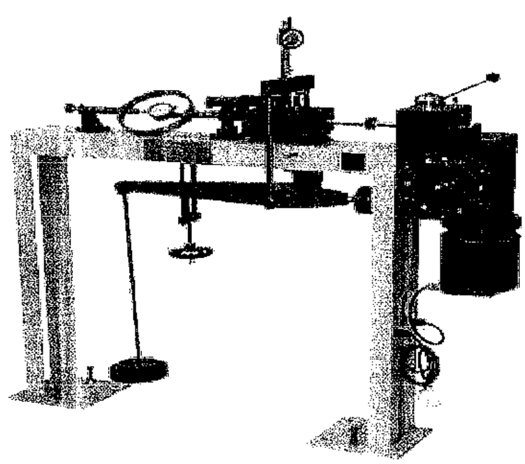


**Plate 3.12**: Direct shear box apparatus

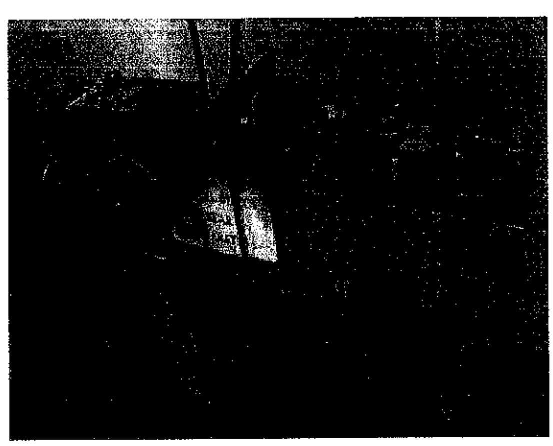


**Plate 3.13**: Laboratory Direct Shear Apparatus

# CHAPTER FOUR

# RESULTS AND DISCUSSIONS

## 4.1 Results

This Chapter presents the result of various geotechnical tests carried out on the four different soil samples used.

**Sample A- Ilotin:** The results of the dry sieve analysis is presented in Table 4.1. It has a $C_u = 5.398$, $C_c = 1.137$ from the particle size distribution curve of Figure 4.1. The liquid limit and plastic limit readings are shown in Tables 4.2 & 4.3 respectively, from which it has a Liquid limit of 52 (Figure 4.2), a plastic limit of 24 and a plasticity index of 28. According to Unified Soil Classification System (USCS), it belongs to SW, i.e. well-graded sand with little fines. According to AASHTO, it falls under A-2-7, which represents clayey sand. The readings from the standard proctor and modified proctor tests are shown in Tables 4.4& 4.5 respectively and the curve set is presented in Figure 4.3. From the laboratory tests on the soil specimen, the readings from the direct shear, triaxial test and unconfined compression test are shown in Tables 4.7, 4.8& 4.9 respectively, of which the direct shear and triaixal test have been plotted in Figures 4.4& 4.5 respectively.

**Table 4.1: Sample A Particle Size Distribution**

| SIEVE NUMBER | SIEVE OPENING (mm) | MASS OF SIEVE (gm) | MASS OF SIEVE + SOIL RETAINED (gm) | SOIL RETAINED (gm) | PERCENT RETAINED | CUMULATIVE % RETAINED | PERCENT PASSING |
|---|---|---|---|---|---|---|---|
| 4 | 5 | 290 | 300 | 10 | 3.333 | 3.333 | 96.667 |
| 8 | 2.36 | 340 | 345 | 5 | 1.667 | 5 | 95.000 |
| 10 | 2 | 355 | 365 | 10 | 3.333 | 8.333 | 91.667 |
| 18 | 1 | 255 | 295 | 40 | 13.333 | 21.667 | 78.333 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **30** | 0.63 | 255 | 305 | 50 | 16.667 | 38.333 | 61.667 |
| **50** | 0.315 | 250 | 335 | 85 | 28.333 | 66.667 | 33.333 |
| **100** | 0.15 | 255 | 310 | 55 | 18.333 | 85.000 | 15.000 |
| **200** | 0.075 | 245 | 295 | 40 | 13.333 | 98.333 | 1.667 |
| | Pan | 210 | 215 | 5 | 1.6667 | 100.00 | 0 |
| | | | TOTAL | 300 | | | |

**Table 4.2: Sample A Liquid Limit Readings**

| **DESCRIPTION** | **1** | **2** | **3** | **4** |
|---|---|---|---|---|
| **Number of Blows** | 36 | 22 | 14 | 9 |
| **Container No.** | A11 | A12 | A13 | A14 |
| **Mass of Container (g)** | 29.8 | 20.2 | 26.9 | 19.7 |
| **Mass of Container + wet soil (g)** | 52.7 | 43.8 | 53.8 | 41.8 |
| **Mass of Container + dry soil (g)** | 45.1 | 35.6 | 44.2 | 33.5 |
| **Mass of water (g)** | 7.6 | 8.2 | 9.6 | 8.3 |
| **Mass of dry soil (g)** | 15.3 | 15.4 | 17.3 | 13.8 |
| **Moisture Content (%)** | 49.673 | 53.25 | 55.491 | 60.145 |

**Table 4.3: Sample A Plastic Limit Readings**

| DESCRIPTION | 1 | 2 |
|---|---|---|
| **Container No.** | A15 | A16 |
| **Mass of Container (g)** | 16.7 | 27 |
| **Mass of Container + wet soil (g)** | 32.6 | 44.3 |
| **Mass of Container + dry soil (g)** | 29.5 | 41.1 |
| **Mass of water (g)** | 3.1 | 3.2 |
| **Mass of dry soil (g)** | 12.8 | 14.1 |
| **Moisture Content (%)** | 24.21875 | 22.69504 |

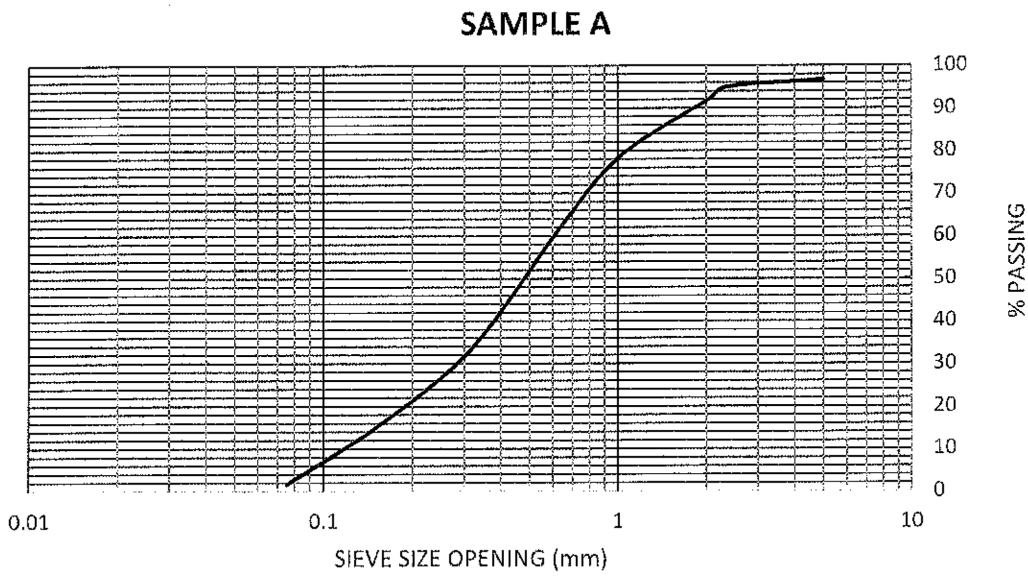
**Table 4.4: Sample A Standard Proctor Test Readings**

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 5650 | 5850 | 6050 | 5700 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 1650 | 1850 | 2050 | 1700 |
| Bulk density (g/cm*3) | 1.65 | 1.85 | 2.05 | 1.7 |
| Container No. | A1 | A2 | A3 | A4 |
| Mass of Container (g) | 20 | 14.9 | 13.9 | 12.8 |
| Mass of Container + wet soil (g) | 49.8 | 46.1 | 44.6 | 45.4 |
| Mass of Container + dry soil (g) | 46.5 | 41.7 | 39.6 | 39.7 |
| Mass of water (g) | 3.3 | 4.4 | 5 | 5.7 |
| Mass of dry soil (g) | 26.5 | 26.8 | 25.7 | 26.9 |
| Moisture Content (%) | 12.453 | 16.418 | 19.455 | 21.190 |
| Dry Density (g/cm*3) | 1.467 | 1.5891 | 1.716 | 1.403 |

**Table 4.5: Sample A Modified Proctor Readings**

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 6000 | 6200 | 6300 | 5900 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 2000 | 2200 | 2300 | 1900 |
| Bulk density (g/cm*3) | 2 | 2.2 | 2.3 | 1.9 |
| Container No. | A5 | A6 | A7 | A8 |
| Mass of Container (g) | 20 | 20.4 | 20 | 20.1 |
| Mass of Container + wet soil (g) | 51 | 52.7 | 52.8 | 53.7 |
| Mass of Container + dry soil (g) | 47.4 | 48.3 | 47.4 | 47.2 |
| Mass of water (g) | 3.6 | 4.4 | 5.4 | 6.5 |
| Mass of dry soil (g) | 27.4 | 27.9 | 27.4 | 27.1 |
| Moisture Content (%) | 13.1387 | 15.7706 | 19.708 | 23.9852 |
| Dry Density (g/cm*3) | 1.7677 | 1.9003 | 1.9213 | 1.5324 |

**Figure 4.1: Sample A Particle Size Distribution Curve**




$$D_{10} = 0.113$$

$$D_{30} = 0.28$$

$$D_{60} = 0.61$$

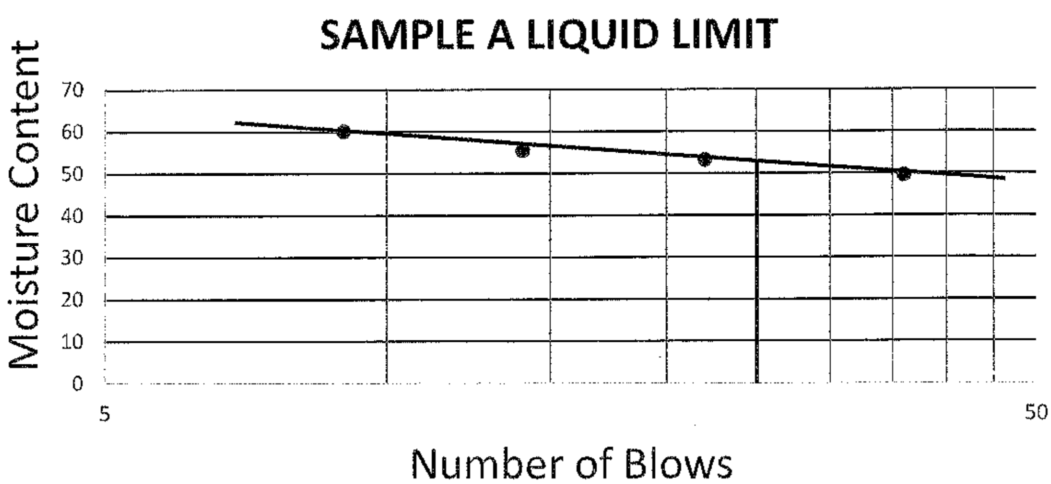



**Figure 4.2: Sample A Liquid Limit Plot**

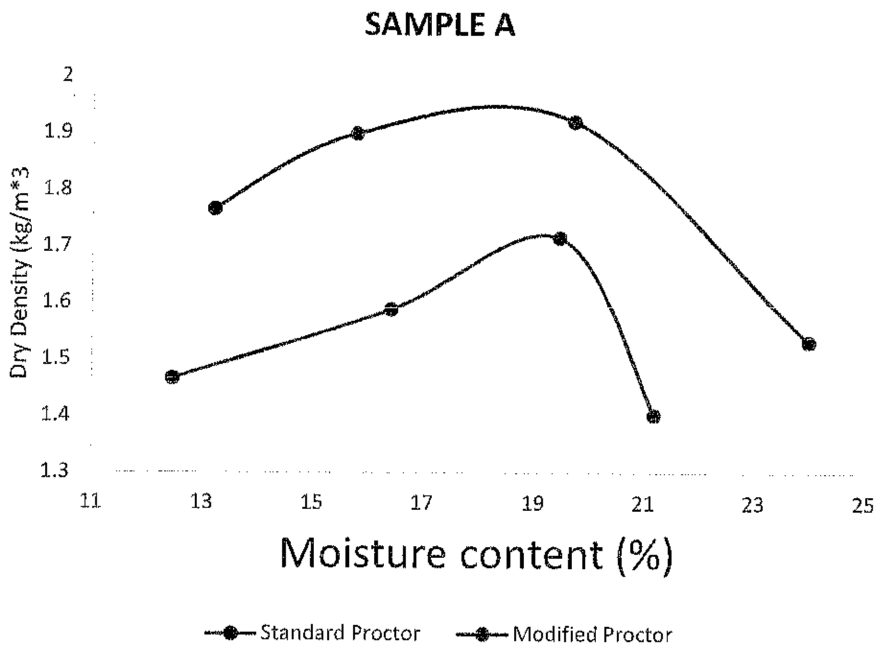



**Figure 4.3: Sample A Compaction Set of Curves**

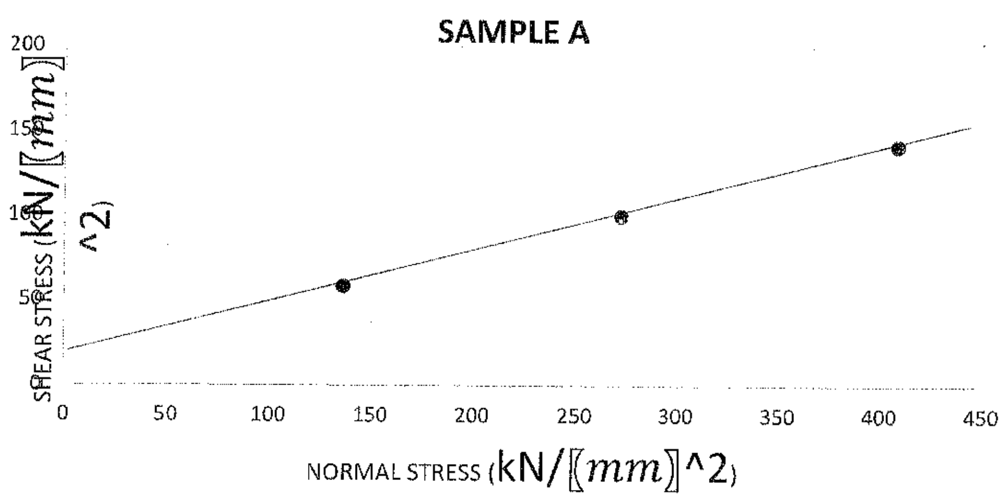



**Figure 4.4: Sample A Direct shear Plot**

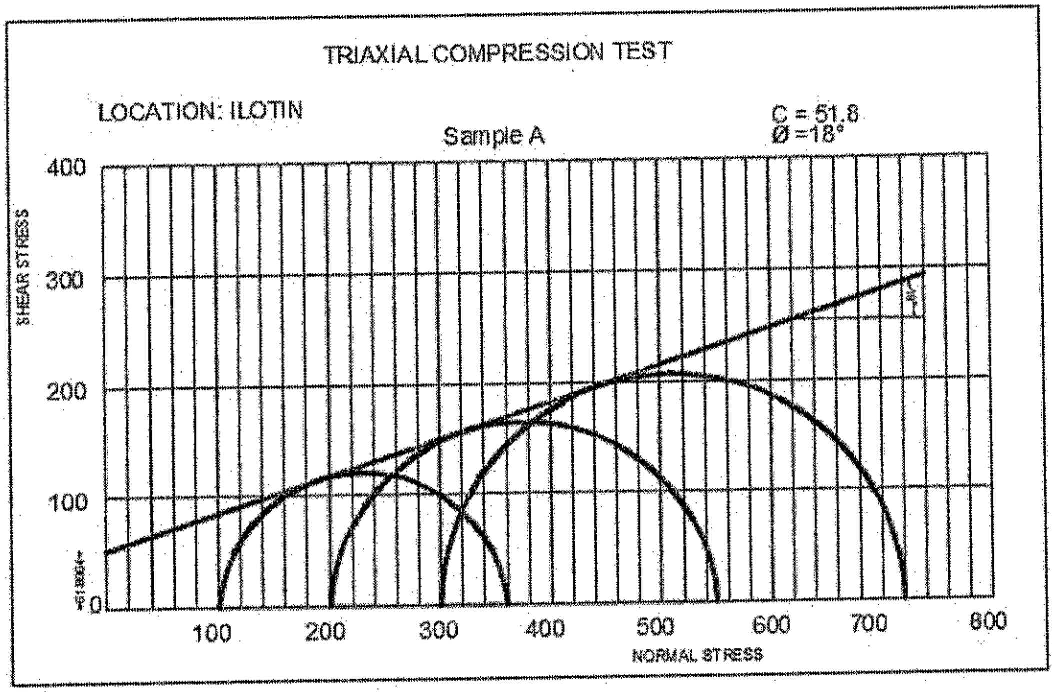



**Figure 4.5: Sample A Triaxial Test Graph**

**Table 4.6: Sample A Direct Shear Test Readings**

| TEST NUMBER | VERTICAL LOAD (KG) | NORMAL FORCE (kN) | NORMAL STRESS kN/⟦mm⟧^2 | MAX. D.R DIV | SHEAR FORCE (kN) | SHEAR STRESS (kN/⟦mm⟧^2) |
|---|---|---|---|---|---|---|
| 1 | 50 | 0.4905 | 136.25 | 35.1 | 0.2106 | 58.5 |
| 2 | 100 | 0.981 | 272.5 | 60 | 0.36 | 100 |
| 3 | 150 | 1.4715 | 408.75 | 85 | 0.51 | 141.667 |

**Table 4.7: Sample A Triaxial (Consolidated Drained) Test Readings**

| TEST NUMBER | CELL PRESSURE, σ_3 (kN/m^2) | DEFORM-ATION DIAL READING (mm) | CHANGE IN LENGTH (mm) | STRAIN | CORRECTED AREA (m^2) | LOAD DIAL READING | DEVIATOR FORCE (kN) | DEVIATOR STRESS, σ_1-σ_3 (kN/m^2) | σ_1 (X 10) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 100 | 375 | 3.75 | 0.05 | 0.0012 | 151 | 0.302 | 261.2 | 36 |
| 2 | 200 | 425 | 4.25 | 0.05 | 0.0012 | 207.2 | 0.414 | 345 | 54 |
| 3 | 300 | 475 | 4.75 | 0.06 | 0.0012 | 254.5 | 0.509 | 420.6 | 72 |

**Table 4.8: Sample A Unconfined Compression Test Readings**

| TEST NUMBER | DGR (mm) | LENGTH (mm) | STRAIN (ΔL/L_o ) | LOAD DGR | FORCE (kN) | AREA ([mm]^2) | CORRECTED AREA ([mm]^2) | STRESS, (kN/m^2) |
|---|---|---|---|---|---|---|---|---|
| 1 | 200 | 76 | 2 | 5 | 0.125 | 0.0019 | 0.002 | 62.406 |
| 2 | 150 | 76 | 1.5 | 4.5 | 0.113 | 0.0019 | 0.002 | 56.167 |

**Sample B- Ijemo-Titun:** The results of the dry sieve analysis is presented in Table 4.9. It has a $C_u = 5.948$, $C_c = 1.05$ from the particle size distribution curve of Figure 4.6. The liquid limit and plastic limit readings are shown in Tables 4.10 & 4.11 respectively, from which it has a Liquid limit of 47 (Figure 4.7), a plastic limit of 26 and a plasticity index of 21. According to Unified Soil Classification System (USCS), it belongs to SP, i.e. poorly-graded sand with little fines. According to AASHTO, it falls under A-2-7, which represents clayey sand. The readings from the standard proctor and modified proctor tests are shown in Tables 4.12& 4.13 respectively and the curve set is presented in Figure 4.8. From the laboratory tests on the soil specimen, the readings from the direct shear, triaxial test and unconfined compression test are shown in Tables 4.14, 4.15& 4.16 respectively, of which the direct shear and triaixal test have been plotted in Figures 4.9& 4.10 respectively.

**Table 4.9: Sample B Particle Size Distribution**

| SIEVE NUMBER | SIEVE OPENING (mm) | MASS OF SIEVE (gm) | MASS OF SIEVE + SOIL RETAINED (gm) | SOIL RETAINED (gm) | PERCENT RETAINED | CUMULATIVE % RETAINED | PERCENT PASSING |
|---|---|---|---|---|---|---|---|
| 4 | 5 | 290 | 295 | 5 | 1.351 | 1.351 | 98.649 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **8** | 2.36 | 340 | 355 | 15 | 4.054 | 5.405 | 94.595 |
| **10** | 2 | 355 | 375 | 20 | 5.405 | 10.810 | 89.189 |
| **18** | 1 | 255 | 300 | 45 | 12.162 | 22.972 | 77.027 |
| **30** | 0.63 | 255 | 330 | 75 | 20.270 | 43.243 | 56.758 |
| **50** | 0.315 | 250 | 340 | 90 | 24.324 | 67.567 | 32.432 |
| **100** | 0.15 | 255 | 325 | 70 | 18.919 | 86.487 | 13.514 |
| **200** | 0.075 | 245 | 280 | 35 | 9.460 | 95.946 | 4.054 |
| | Pan | 270 | 285 | 15 | 4.054 | 100.00 | 0 |
| | | | TOTAL | 370 | | | |

**Table 4.10: Sample B Liquid Limit Readings**

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Number of Blows | 44 | 30 | 23 | 12 |
| Container No. | B11 | B12 | B13 | B14 |
| Mass of Container (g) | 19.9 | 21.6 | 22.4 | 27 |
| Mass of Container + wet soil (g) | 37.8 | 40.5 | 43.3 | 48.8 |
| Mass of Container + dry soil (g) | 32.4 | 34.5 | 36.4 | 41.3 |
| Mass of water (g) | 5.4 | 6 | 6.9 | 7.5 |
| Mass of dry soil (g) | 12.5 | 12.9 | 14 | 14.3 |
| Moisture Content (%) | 43.20 | 46.51 | 49.29 | 52.44 |

**Table 4.11: Sample B Plastic Limit Readings**

| DESCRIPTION | 1 | 2 |
|---|---|---|
| Container No. | B15 | B16 |
| Mass of Container (g) | 20 | 19.9 |
| Mass of Container + wet soil (g) | 31.5 | 34.3 |
| Mass of Container + dry soil (g) | 29.1 | 31.3 |
| Mass of water (g) | 2.4 | 3 |
| Mass of dry soil (g) | 9.1 | 11.4 |
| Moisture Content (%) | 26.3736264 | 26.316 |

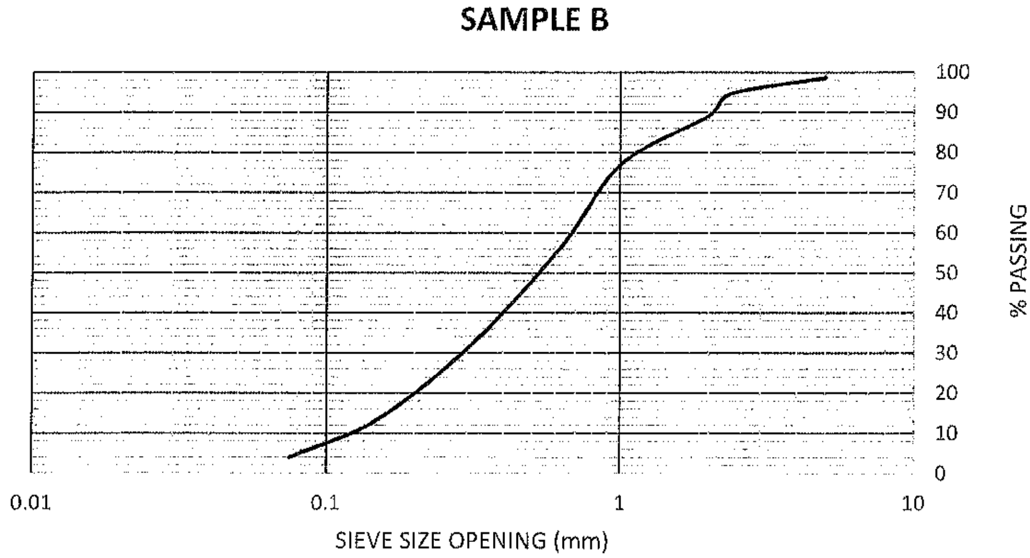



**Figure 4.6: Sample B Particle Size Distribution Curve**

$$D_{10} = 0.116$$

$$D_{30} = 0.29$$

$$D_{6\ 0}= 0.69$$

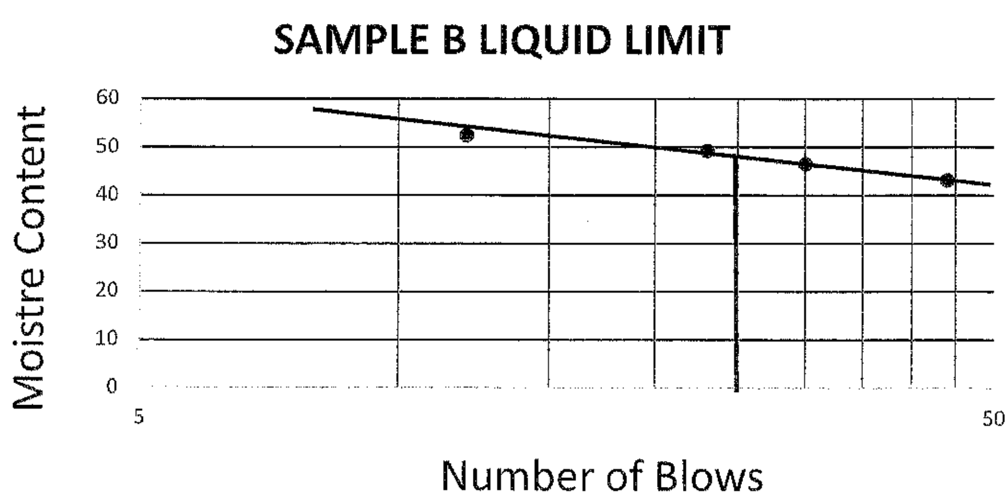



**Figure 4.7: Sample B Liquid Limit Plot**

**Table 4.12: Sample B Standard Proctor Readings**

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 5750 | 5850 | 6050 | 5900 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 1750 | 1850 | 2050 | 1900 |
| Bulk density (g/cm*3) | 1.75 | 1.85 | 2.05 | 1.9 |
| Container No. | B1 | B2 | B3 | B4 |
| Mass of Container (g) | 27 | 20.3 | 16.6 | 20 |
| Mass of Container + wet soil (g) | 54.9 | 58.8 | 55.2 | 56.5 |
| Mass of Container + dry soil (g) | 51.6 | 53.4 | 48.8 | 49.8 |
| Mass of water (g) | 3.3 | 5.4 | 6.4 | 6.7 |
| Mass of dry soil (g) | 24.6 | 33.1 | 32.2 | 29.8 |
| Moisture Content (%) | 13.41 | 16.31 | 19.87 | 22.483 |
| Dry Density (g/cm*3) | 1.543 | 1.591 | 1.710 | 1.5512 |

**Table 4.13: Sample B Modified Proctor Readings**

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 5450 | 5600 | 5700 | 5650 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 1450 | 1600 | 1700 | 1650 |
| Bulk density (g/cm*3) | 1.45 | 1.6 | 1.7 | 1.65 |
| Container No. | B5 | B6 | B7 | B8 |
| Mass of Container (g) | 21.2 | 21.6 | 18.6 | 21.7 |
| Mass of Container + wet soil (g) | 44.2 | 46.4 | 38.8 | 57.4 |
| Mass of Container + dry soil (g) | 41.5 | 42.9 | 35.4 | 50.6 |
| Mass of water (g) | 2.7 | 3.5 | 3.4 | 6.8 |
| Mass of dry soil (g) | 20.3 | 21.3 | 16.8 | 28.9 |
| Moisture Content (%) | 13.300 | 16.432 | 20.238 | 23.529 |
| Dry Density (g/cm*3) | 1.587 | 1.718 | 1.913 | 1.7 |

**Table 4.14: Sample B Direct Shear Test Readings**

| TEST NUMBER | VERTICAL LOAD (KG) | NORMAL FORCE (kN) | NORMAL STRESS (kN/m^2) | MAX. D.R DIV | SHEAR FORCE (kN) | SHEAR STRESS (kN/m^2) |
|---|---|---|---|---|---|---|
| 1 | 50 | 0.491 | 136.25 | 40.9 | 0.2034 | 56.5 |
| 2 | 100 | 0.981 | 272.5 | 61.5 | 0.369 | 102.5 |
| 3 | 150 | 1.4715 | 408.75 | 89 | 0.534 | 148.3 |

**Table 4.15: Sample B Triaxial (Consolidated Drained) Test Readings**

| TEST No. | CELL PRESSURE , σ_3 (kN/m^2) | DEFORM-ATION DIAL READING (mm) | CHANGE IN LENGTH (mm) | STRAIN | CORRECTED AREA (m^2) | LOAD DIAL READING | DEVIATOR FORCE (kN) | DEVIATOR STRESS, σ_1-σ_3 (kN/m^2) | σ_1 (X 10) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 100 | 450 | 4.5 | 0.059 | 0.0012 | 132.8 | 0.265 | 220.4 | 32 |
| 2 | 200 | 500 | 5 | 0.066 | 0.0012 | 196 | 0.392 | 322.9 | 52 |
| 3 | 300 | 350 | 3.5 | 0.046 | 0.0012 | 274.2 | 0.548 | 460.1 | 76 |

**Table 4.16: Sample B Unconfined Compression Test Readings**

| TEST NUMBER | DGR (mm) | LENGTH (mm) | STRAIN (ΔL/L_o ) | LOAD DGR | FORCE (kN) | AREA ([mm]^2) | CORRECTED AREA ([mm]^2) | STRESS, (kN/m^2) |
|---|---|---|---|---|---|---|---|---|
| 1 | 150 | 76 | 1.5 | 8 | 0.2 | 0.00197 | 0.002 | 99.850 |
| 2 | 150 | 76 | 1.5 | 7.2 | 0.18 | 0.00197 | 0.002 | 89.865 |

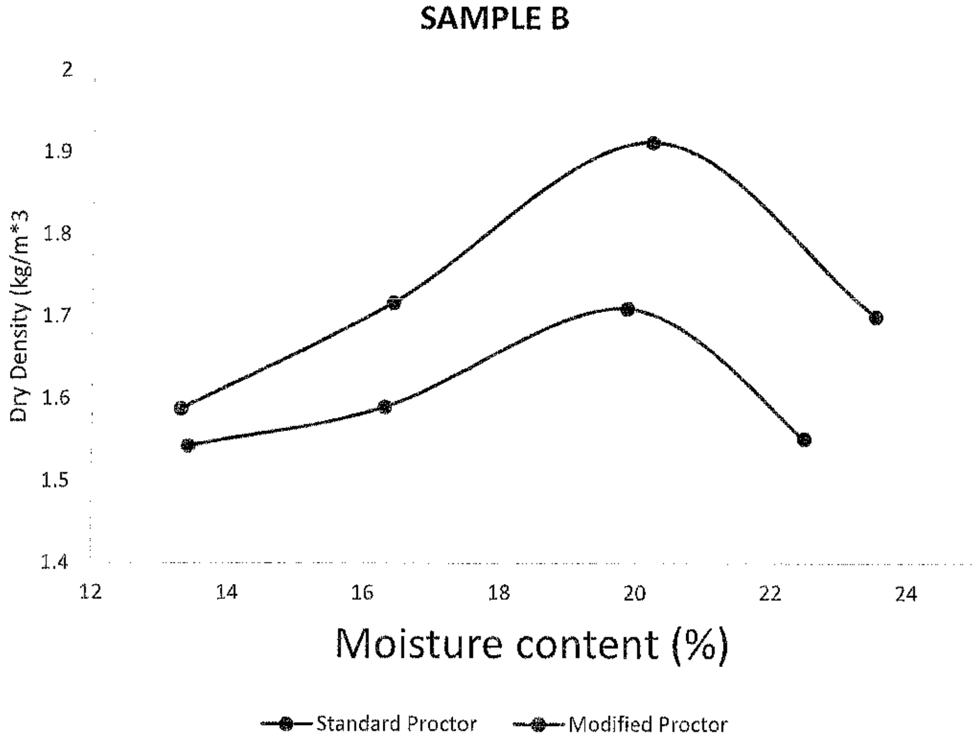



**Figure 4.8: Sample B Compaction Set of Curves**

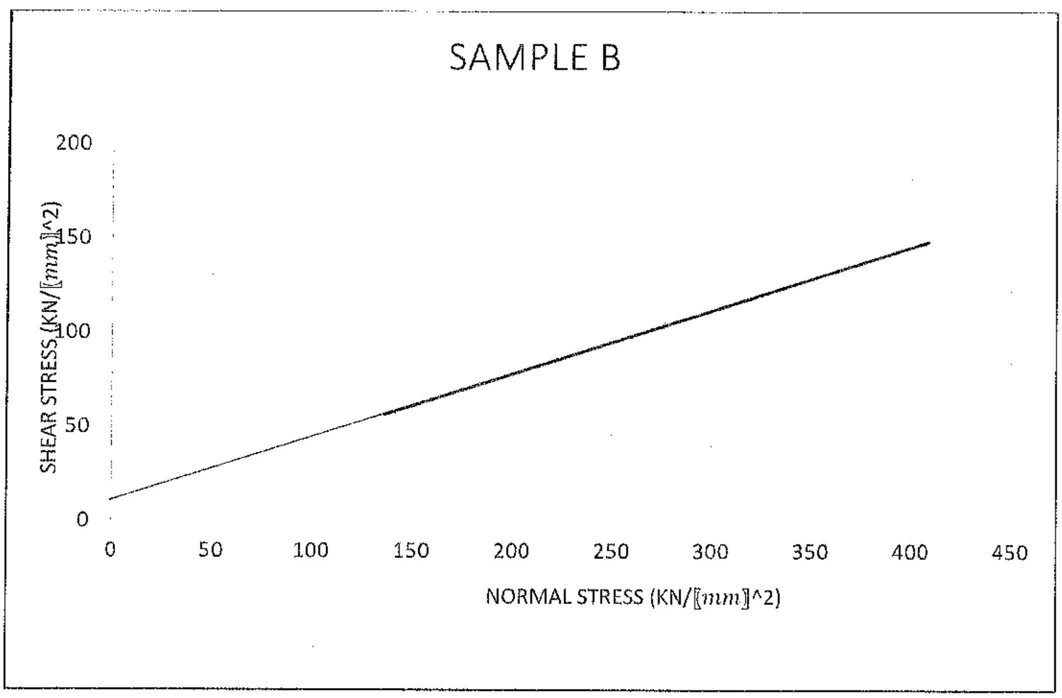



**Figure 4.9: Sample B Triaxial Test Plot**

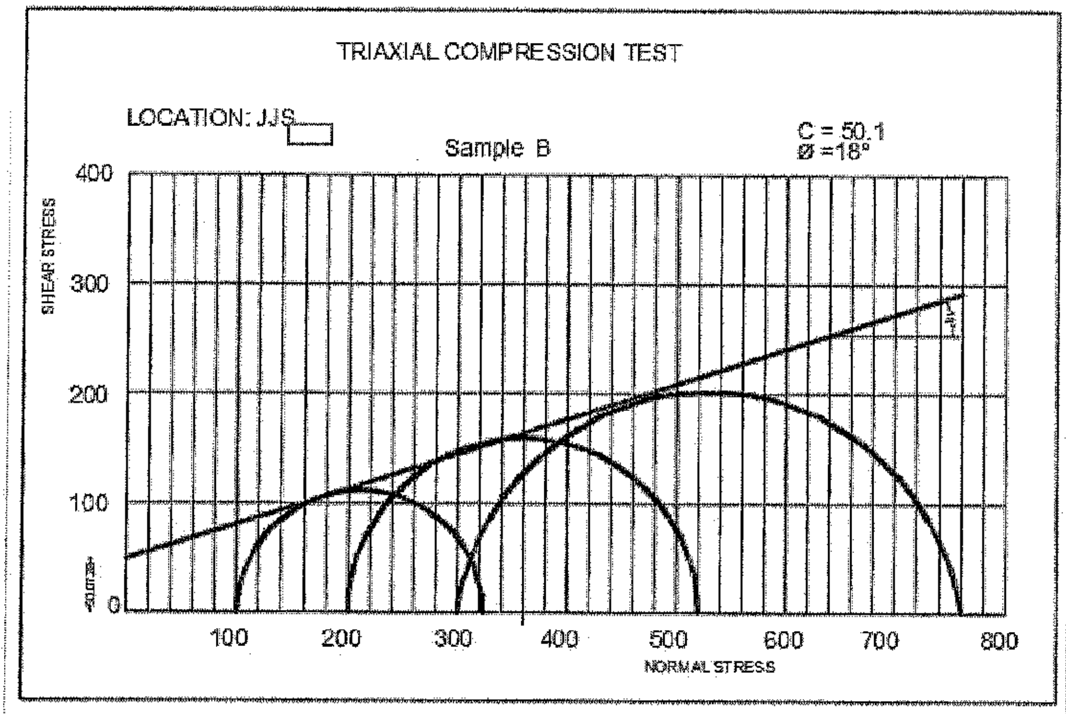



**Figure 4.10: Sample B Triaxial Test Plot**

**Sample C- Yeye-Oba:** The results of the dry sieve analysis is presented in Table 4.17. It has a $C_u = 5.09$, $C_c = 1.372$ from the particle size distribution curve of Figure 4.11. The liquid limit and plastic limit readings are shown in Tables 4.18 & 4.19 respectively, from which it has a Liquid limit of 29 (Figure 4.12), a plastic limit of 12 and a plasticity index of 17. According to Unified Soil Classification System (USCS), it belongs to SW-OL, i.e. well-graded sand with organic fines of low plasticity. According to AASHTO, it falls under A-2-6, which represents clayey sand. The readings from the standard proctor and modified proctor tests are shown in Tables 4.20& 4.21 respectively and the set of curves is presented in Figure 4.13. From the laboratory tests on the soil specimen, the readings from the direct shear test is shown in Tables 4.22, of which the direct shear test has been plotted in Figures 4.14. It was difficult to obtain undisturbed block samples for other strength tests.

Table 4.17: Sample C Particle Size Distribution

| SIEVE NUMBER | SIEVE OPENING (mm) | MASS OF SIEVE (gm) | MASS OF SIEVE + SOIL RETAINED (gm) | SOIL RETAINED (gm) | PERCENT RETAINED | CUMULATIVE % RETAINED | PERCENT PASSING |
|---|---|---|---|---|---|---|---|
| **4** | 5 | 290 | 295 | 5 | 1.25 | 1.25 | 98.75 |
| **8** | 2.36 | 340 | 355 | 15 | 3.75 | 5 | 95 |
| **10** | 2 | 355 | 370 | 15 | 3.75 | 8.75 | 91.25 |
| **18** | 1 | 255 | 285 | 30 | 7.5 | 16.25 | 83.75 |
| **30** | 0.63 | 255 | 325 | 70 | 17.5 | 33.75 | 66.25 |
| **50** | 0.315 | 250 | 385 | 135 | 33.75 | 67.5 | 32.5 |
| **100** | 0.15 | 255 | 330 | 75 | 18.75 | 86.25 | 13.75 |
| **200** | 0.075 | 245 | 280 | 35 | 8.75 | 95 | 5 |
| | Pan | 200 | 255 | 20 | 5 | 100 | 0 |
| | | | | 400 | | | |

Table 4.18: Sample C Liquid Limit Readings

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| **Number of Blows** | 49 | 38 | 22 | 13 |
| **Container No.** | C11 | C12 | C13 | C14 |
| **Mass of Container (g)** | 12 | 12 | 16 | 21.2 |
| **Mass of Container + wet soil (g)** | 31.2 | 46.4 | 39.8 | 46.4 |
| **Mass of Container + dry soil (g)** | 27.4 | 40.3 | 34.2 | 40 |
| **Mass of water (g)** | 3.8 | 6.1 | 5.6 | 6.4 |
| **Mass of dry soil (g)** | 15.4 | 28.3 | 18.2 | 18.8 |
| **Moisture Content (%)** | 24.675 | 21.555 | 30.769 | 34.043 |

**Table 4.19: Sample C Plastic Limit Readings**

| DESCRIPTION | 1 | 2 |
|---|---|---|
| Container No. | C15 | C16 |
| Mass of Container (g) | 21.6 | 20 |
| Mass of Container + wet soil (g) | 35.9 | 38.2 |
| Mass of Container + dry soil (g) | 34.1 | 36.5 |
| Mass of water (g) | 1.8 | 1.7 |
| Mass of dry soil (g) | 12.5 | 16.5 |
| Moisture Content (%) | 14.4 | 10.303 |

**Table 4.20: Sample C Standard Proctor Readings**

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 5650 | 5850 | 6050 | 5800 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 1650 | 1850 | 2050 | 1800 |
| Bulk density (g/cm*3) | 1.65 | 1.85 | 2.05 | 1.8 |
| Container No. | C1 | C2 | C3 | C4 |
| Mass of Container (g) | 20.1 | 22.1 | 21.2 | 14.3 |
| Mass of Container + wet soil (g) | 45.1 | 51 | 56 | 49.4 |
| Mass of Container + dry soil (g) | 43 | 48 | 51.4 | 43.6 |
| Mass of water (g) | 2.1 | 3 | 4.6 | 5.8 |
| Mass of dry soil (g) | 22.9 | 25.9 | 30.2 | 29.3 |
| Moisture Content (%) | 9.170 | 11.583 | 15.232 | 19.795 |
| Dry Density (g/cm*3) | 1.512 | 1.658 | 1.779 | 1.503 |

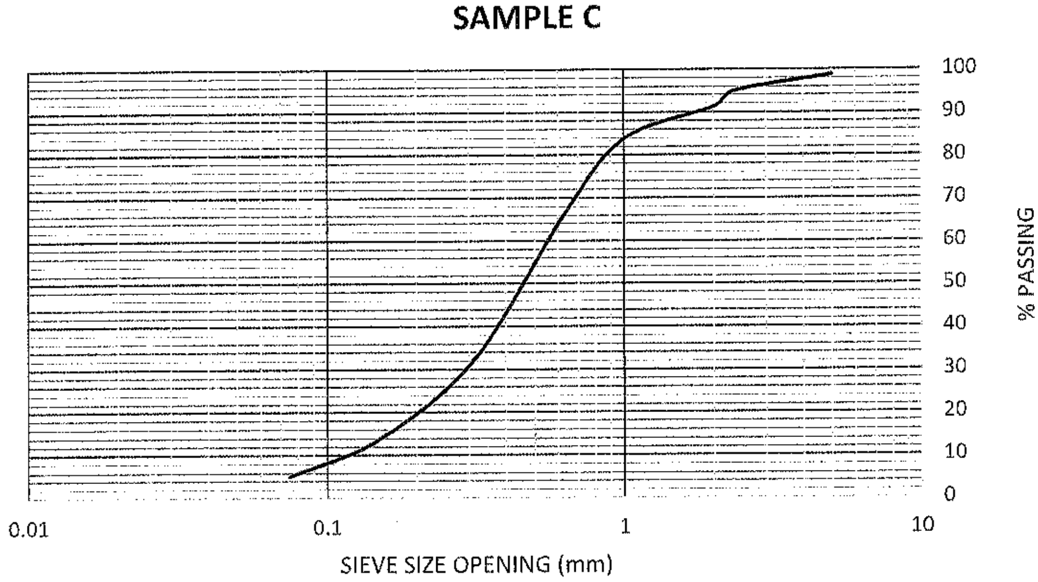



**Figure 4.11: Sample C Particle Size Distribution Curve**

$$D_{10} = 0.112$$

$$D_{30} = 0.296$$

$$D_{60} = 0.57$$

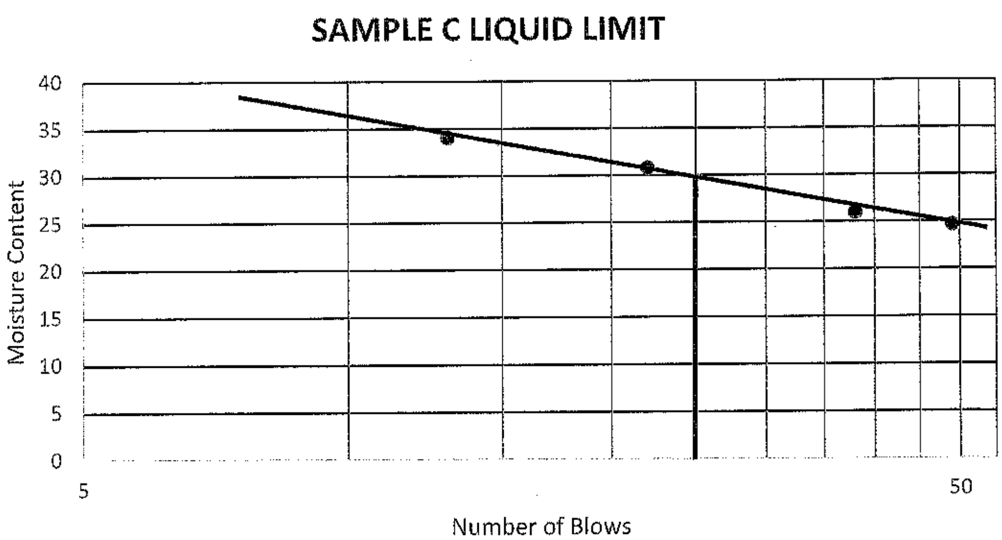



**Figure 4.12: Sample C Liquid Limit Plot**

Table 4.21: Sample C Modified Proctor Readings

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 5900 | 6150 | 6300 | 6100 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 1900 | 2150 | 2300 | 2100 |
| Bulk density (kg/m*3) | 1.9 | 2.15 | 2.3 | 2.1 |
| Container No. | C5 | C6 | C7 | C8 |
| Mass of Container (g) | 15.6 | 10.2 | 20.7 | 18.5 |
| Mass of Container + wet soil (g) | 40.6 | 30.6 | 53.1 | 49.8 |
| Mass of Container + dry soil (g) | 38.8 | 28.2 | 48.6 | 44.8 |
| Mass of water (g) | 1.8 | 2.4 | 4.5 | 5 |
| Mass of dry soil (g) | 23.2 | 18 | 27.9 | 26.3 |
| Moisture Content (%) | 7.758 | 13.33 | 16.129 | 19.011 |
| Dry Density (kg/m*3) | 1.763 | 1.897 | 1.981 | 1.764 |

Table 4.22: Sample C Direct Shear Test Readings

| TEST NUMBER | VERTICAL LOAD (KG) | NORMAL FORCE (N) | NORMAL STRESS (kN/m^2) | MAX. D.R DIV | SHEAR FORCE (kN) | SHEAR STRESS (kN/m^2) |
|---|---|---|---|---|---|---|
| 1 | 50 | 0.4905 | 136.25 | 69.5 | 0.414 | 115 |
| 2 | 100 | 0.981 | 272.5 | 91.2 | 0.547 | 152 |
| 3 | 150 | 1.4715 | 408.75 | 114 | 0.684 | 190 |

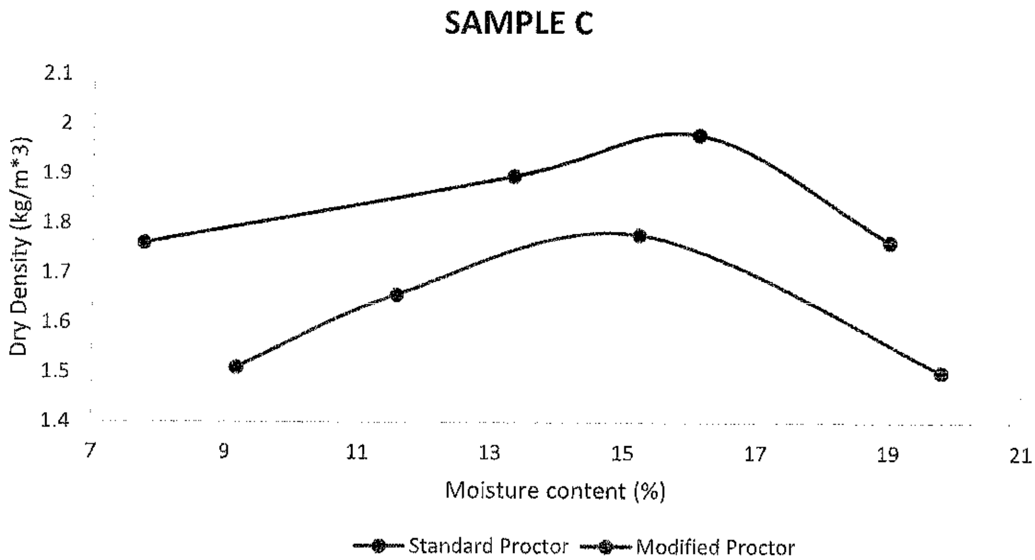



**Figure 4.13: Sample C Compaction Set of Curves**

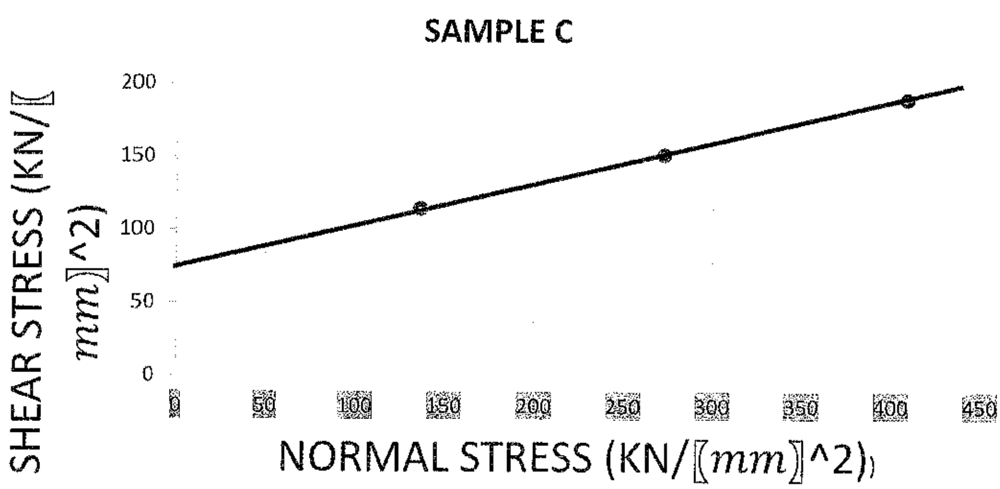



**Figure 4.14: Sample C Direct shear Plot**

**Sample D- Asin:** The results of the dry sieve analysis is presented in Table 4.23. It has a $C_u = 8.646$, $C_c = 1.9285$ from the particle size distribution curve of Figure 4.15. The liquid limit and plastic limit readings are shown in Tables 4.24 & 4.25 respectively, from which it has a Liquid limit of 46 (Figure 4.16), a plastic limit of 29 and a plasticity index of 17. According to Unified Soil Classification System (USCS), it belongs to SW-SC, i.e. well-graded sand with 7% fines of clay content. According to AASHTO, it falls under A-2-7, which represents clayey sand. The readings from the standard proctor and modified proctor tests are shown in Tables 4.26& 4.27 respectively and the curve set is presented in Figure 4.17. From the laboratory tests on the soil specimen, the readings from the direct shear, triaxial test and unconfined compression test are shown in Tables 4.28, 4.29& 4.30 respectively, of which the direct shear and triaixal test have been plotted in Figures 4.18& 4.19 respectively

**Table 4.23: Sample D Particle Size Distribution**

| SIEVE NUMBER | SIEVE OPENING (mm) | MASS OF SIEVE (gm) | MASS OF SIEVE + SOIL RETAINED (gm) | SOIL RETAINED (gm) | PERCENT RETAINED | CUMULATIVE % RETAINED | PERCENT PASSING |
|---|---|---|---|---|---|---|---|
| **4** | 5 | 290 | 300 | 10 | 2 | 2 | 98 |
| **8** | 2.36 | 340 | 355 | 15 | 3 | 5 | 95 |
| **10** | 2 | 355 | 405 | 50 | 10 | 15 | 85 |
| **18** | 1 | 255 | 335 | 80 | 16 | 31 | 69 |
| **30** | 0.63 | 255 | 375 | 120 | 24 | 55 | 45 |
| **50** | 0.315 | 250 | 385 | 100 | 20 | 75 | 25 |
| **100** | 0.15 | 255 | 295 | 45 | 9 | 84 | 16 |
| **200** | 0.075 | 245 | 290 | 45 | 9 | 93 | 7 |
| **Pan** | 0 | 255 | 290 | 35 | 7 | 100 | 0 |
| | | | TOTAL | 500 | | | |

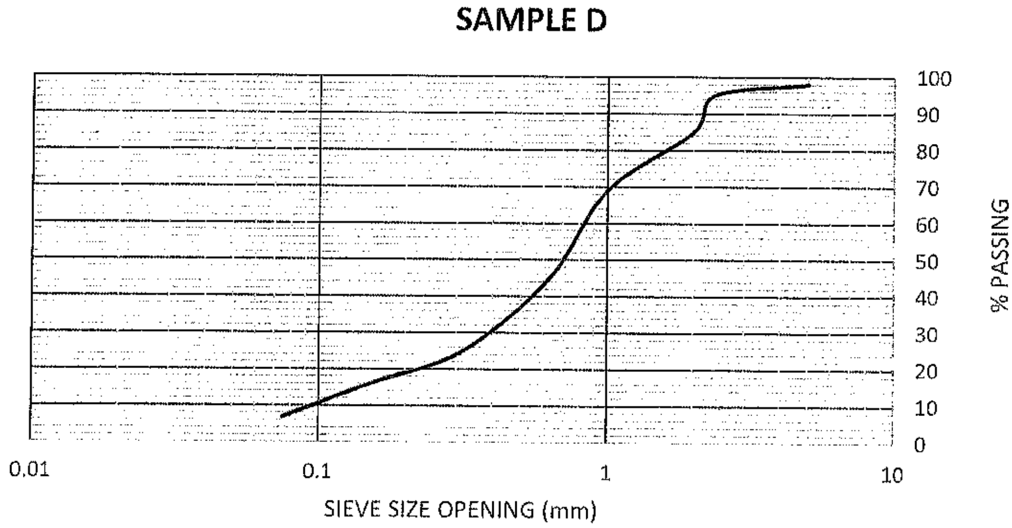



**Figure 4.15: Sample D Particle Size Distribution Curve**

$$D_{10} = 0.096$$

$$D_{30} = 0.392$$

$$D_{6\ 0} = 0.83$$

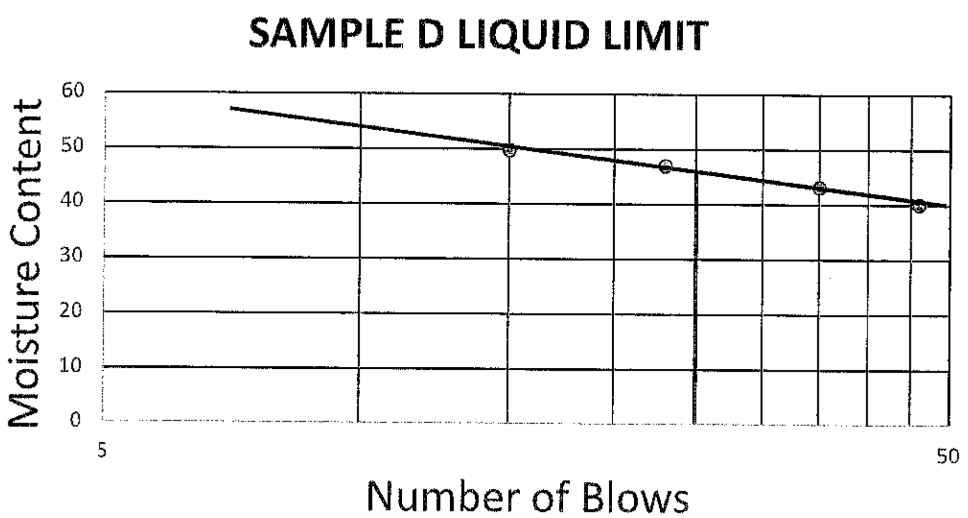



**Figure 4.16: Sample D Liquid Limit Plot**

**Table 4.24: Sample D Liquid Limit Readings**

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Number of Blows | 46 | 35 | 23 | 15 |
| Container No. | D11 | D12 | D13 | D14 |
| Mass of Container (g) | 20.3 | 19.5 | 22.1 | 19.9 |
| Mass of Container + wet soil (g) | 40.6 | 41.4 | 46.2 | 45.8 |
| Mass of Container + dry soil (g) | 34.8 | 34.8 | 38.5 | 37.2 |
| Mass of water (g) | 5.8 | 6.6 | 7.7 | 8.6 |
| Mass of dry soil (g) | 14.5 | 15.3 | 16.4 | 17.3 |
| Moisture Content (%) | 40 | 43.137 | 46.951 | 49.711 |

**Table 4.25: Sample D Plastic Limit Readings**

| DESCRIPTION | 1 | 2 |
|---|---|---|
| Container No. | D15 | D16 |
| Mass of Container (g) | 18.6 | 26.8 |
| Mass of Container + wet soil (g) | 29.6 | 41.9 |
| Mass of Container + dry soil (g) | 27.1 | 38.5 |
| Mass of water (g) | 2.5 | 3.4 |
| Mass of dry soil (g) | 8.5 | 11.7 |
| Moisture Content (%) | 29.41 | 29.06 |

Table 4.26: Sample D Standard Proctor Readings

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 5600 | 5700 | 5800 | 5700 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 1600 | 1700 | 1800 | 1700 |
| Bulk density (g/cm*3) | 1.6 | 1.7 | 1.8 | 1.7 |
| Container No. | D1 | D2 | D3 | D4 |
| Mass of Container (g) | 20.1 | 22.1 | 21.2 | 17.7 |
| Mass of Container + wet soil (g) | 49.5 | 58 | 56.8 | 49.9 |
| Mass of Container + dry soil (g) | 47 | 54 | 51.9 | 44.8 |
| Mass of water (g) | 2.5 | 4 | 4.9 | 5.1 |
| Mass of dry soil (g) | 26.9 | 31.9 | 30.7 | 27.1 |
| Moisture Content (%) | 9.293 | 12.539 | 15.961 | 18.819 |
| Dry Density (g/cm*3) | 1.464 | 1.511 | 1.552 | 1.431 |

Table 4.27: Sample D Modified Proctor Readings

| DESCRIPTION | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Mass of Mould + wet soil (g) | 5950 | 6100 | 6200 | 6000 |
| Mass of Mould (g) | 4000 | 4000 | 4000 | 4000 |
| Mass of compacted soil (g) | 1950 | 2100 | 2200 | 2000 |
| Bulk density (g/cm*3) | 1.95 | 2.1 | 2.2 | 2.0 |
| Container No. | D5 | D6 | D7 | D8 |
| Mass of Container (g) | 20.1 | 10 | 17.3 | 11.2 |
| Mass of Container + wet soil (g) | 43.9 | 44.3 | 50.3 | 44.1 |
| Mass of Container + dry soil (g) | 41.4 | 40.1 | 45.2 | 38.1 |
| Mass of water (g) | 2.5 | 4.2 | 5.1 | 6 |
| Mass of dry soil (g) | 21.3 | 30.1 | 27.9 | 26.9 |
| Moisture Content (%) | 11.737 | 13.954 | 18.279 | 22.305 |
| Dry Density (g/cm*3) | 1.745 | 1.843 | 1.86 | 1.635 |

**Table 4.28: Sample D Direct Shear Test Readings**

| TEST NUMBER | VERTICAL LOAD (KG) | NORMAL FORCE (kN) | NORMAL STRESS (kN/m^2) | MAX. D.R DIV | SHEAR FORCE (kN) | SHEAR STRESS (kN/m^2) |
|---|---|---|---|---|---|---|
| 1 | 50 | 0.4905 | 136.25 | 48.6 | 0.2556 | 71 |
| 2 | 100 | 0.981 | 272.5 | 72.2 | 0.4332 | 120 |
| 3 | 150 | 1.4715 | 408.75 | 97.8 | 0.5868 | 163 |

**Table 4.29: Sample D Triaxial (Consolidated Drained) Test Readings**

| **TEST NO.** | **CELL PRESSURE, $\sigma_3$ (kN/m^2)** | **DEFORM-ATION DIAL READING (mm)** | **CHANGE IN LENGTH (mm)** | **STRAIN** | **CORRECTED AREA (m^2)** | **LOAD DIAL READING** | **DEVIATOR FORCE (kN)** | **DEVIATOR STRESS, $\sigma_1-\sigma_3$ (kN/m^2)** | **$\sigma_1$ (X 10)** |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 100 | 325 | 3.25 | 0.043 | 0.0012 | 145.3 | 0.291 | 244.8 | 34 |
| 2 | 200 | 450 | 4.5 | 0.059 | 0.0012 | 214.0 | 0.428 | 400.0 | 60 |
| 3 | 300 | 275 | 2.75 | 0.036 | 0.0012 | 271.6 | 0.543 | 461.0 | 76 |

**Table 4.30: Sample D Unconfined Compression Test Readings**

| **TEST NUMBER** | **DGR (mm)** | **LENGTH (mm)** | **STRAIN ($\Delta L/L_o$)** | **LOAD DGR** | **FORCE (kN)** | **AREA ([mm]^2)** | **CORRECTED AREA ([mm]^2)** | **STRESS, (kN/m^2)** |
|---|---|---|---|---|---|---|---|---|
| 1 | 150 | 76 | 1.0 | 3 | 0.075 | 0.002 | 0.002 | 37 |
| 2 | 150 | 76 | 0.75 | 4 | 0.1 | 0.002 | 0.002 | 49.9 |

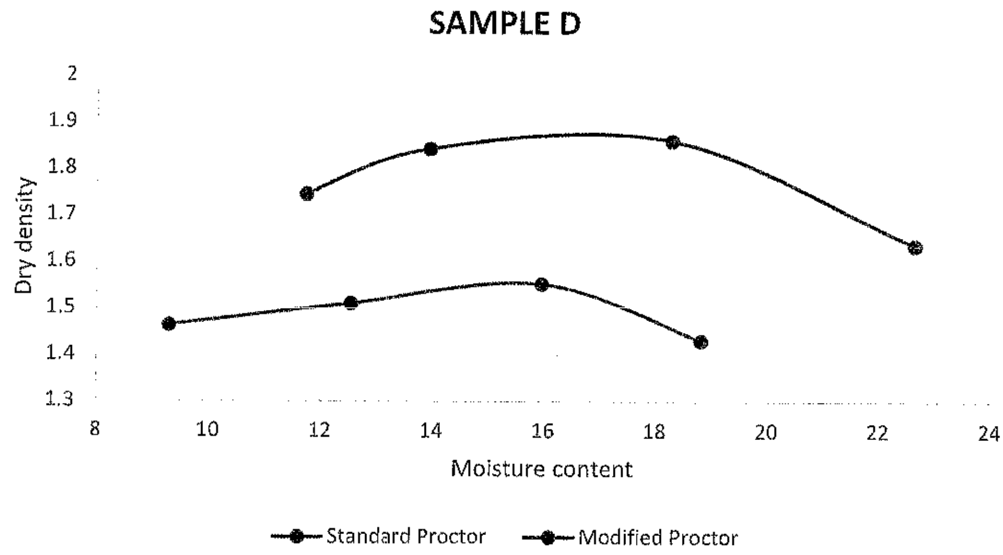



**Figure 4.17: Sample D Compaction Set of Curves**

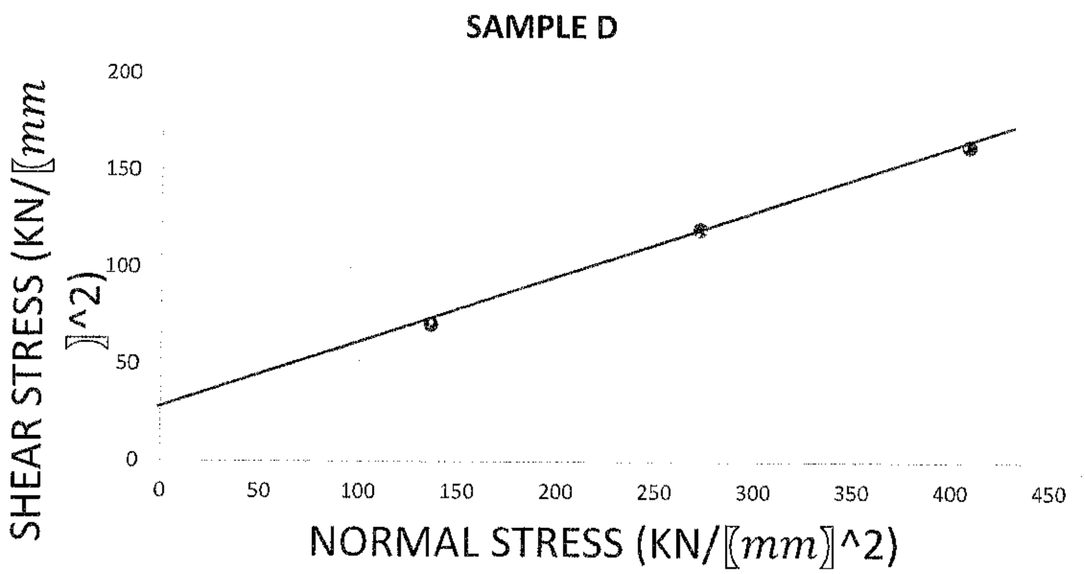



**Figure 4.18: Sample D Direct shear Plot**

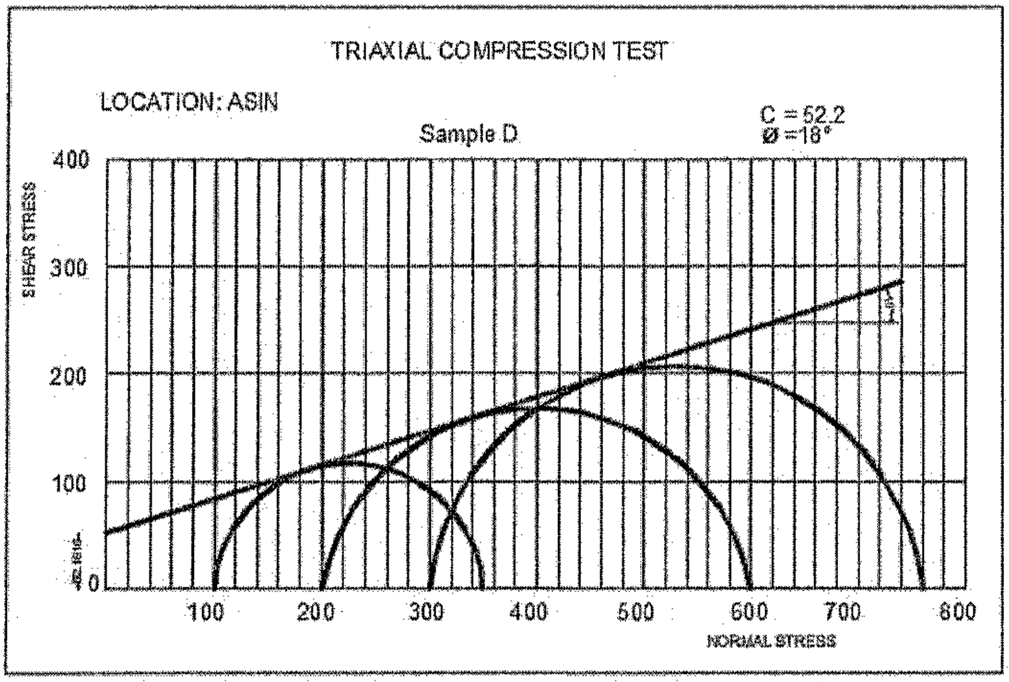



**Figure 4.19: Sample D Triaxial Test Plot**

The results from the natural moisture content, atterberg limits and specific gravity tests has been summarized into Table 4.31. While for the particle size distribution curves, a concise review is shown in Table 4.32. Due to the nature of the study, a lot of compaction tests were carried using the two methods earlier stated, Table 4.33 shows the brief overview. Table 4.34 indicates major parameters gotten from the various strength tests undergone.

**Table 4.31: Summary of Results of Natural Moisture Content, Atterberg Limit and Specific Gravity Tests**

| Sample Point | Average Natural Moisture Content (%) | Atterberg's Limit | | | | Average Specific Gravity |
|---|---|---|---|---|---|---|
| | | **Liquid Limit (LL)** | **Plastic limit (PL)** | **Plasticity Index (PI)** | **Shrinkage limit (SL)** | |
| **A** | 23.17 | 52 | 24 | 28 | 12.1 | 2.63 |
| **B** | 21.64 | 47 | 26 | 21 | 10.0 | 2.31 |
| **C** | 16.03 | 29 | 12 | 17 | 7.1 | 2.35 |
| **D** | 20.07 | 46 | 29 | 17 | 10.0 | 2.56 |

**Table 4.32: Summary of Results of Particle Size Distribution (Sieve Analysis)**

| Sample Point | Sieve analysis [% Passing Sieve] | | | Coefficient of Uniformity $(C_u)$ | Coefficient of Curvature $(C_c)$ | Soil Classification | |
|---|---|---|---|---|---|---|---|
| | **10** | **40** | **200** | | | **AASHTO** | **USCS** |
| **A** | 91.67 | 47.45 | 1.67 | 5.398 | 1.137 | A-2-7 | SW |
| **B** | 89.19 | 44.59 | 4.05 | 5.948 | 1.05 | A-2-7 | SP |
| **C** | 91.25 | 49.38 | 5.00 | 5.09 | 1.372 | A-2-6 | SW-OL |
| **D** | 85 | 35.00 | 7.00 | 8.646 | 1.9285 | A-2-7 | SW-SC |

**Table 4.33: Summary of Results for Compaction tests**

| Sample Point | Standard Proctor Test | | Modified Proctor Test | |
|---|---|---|---|---|
| | **Maximum Dry Density ($g/cm^3$)** | **Optimum Moisture Content {OMC %}** | **Maximum Dry Density ($g/cm^3$)** | **Optimum Moisture Content {OMC %)** |
| **A** | 1.73 | 19.05 | 1.95 | 18.20 |
| **B** | 1.71 | 19.80 | 1.92 | 20.20 |
| **C** | 1.79 | 14.90 | 1.99 | 15.70 |
| **D** | 1.56 | 15.40 | 1.87 | 16.90 |

**Table 4.34: Summary of Results for Strength tests**

| Sample Point | Direct Shear Test | | Triaxial Test | | Unconfined Compression | |
|---|---|---|---|---|---|---|
| | **Cohesion, $c$ $(kN/m^2)$** | **Angle of internal friction, $\emptyset$** | **Cohesion, $c$ $(kN/m^2)$** | **Angle of internal friction, $\emptyset$** | **Average Unconfined compressive Strength, $\sigma_c$ $(kN/m^2)$** | **Undrained Shear Strength, $S_u$ $(kN/m^2)$** |
| **A** | 15.00 | $16.22^o$ | 51.8 | $18^o$ | 57.29 | 28.64 |
| **B** | 11.00 | $17.95^o$ | 50.1 | $18^o$ | 94.85 | 47.43 |
| **C** | 74.41 | $15.26^o$ | - | - | - | - |
| **D** | 26.21 | $18.44^o$ | 52.2 | $18^o$ | 43.45 | 21.73 |

## 4.2 Discussions

### 4.2.1 Particle size distribution& other Physical Properties

According to the FMWH (1997), ≤35% of fine material in a soil mass was recommended to be useable as sub-base material. All the sample soils meet this requirement. The natural moisture content for all soil samples ranges between 16 and 24%. Specific gravity ranges from 2.31 to 2.63. The soils can therefore be classified to have relatively poor to fair drainage characteristics. The high value could be because the sample was collected during the raining season.

### 4.2.2 Atterberg Limits

The Atterberg limits values of the soils are contained in table 4.31. Standard for road works recommend liquid limits of 50% maximum for subbase and base materials (FMWH, 1997). All the samples except Sample A fall within this range, thus making them suitable for subbase and base materials. Liquid limit less than 30% indicates low plasticity, between 35% and 50% indicates intermediate plasticity, between 50% and 70% high plasticity, between 70% and 90% indicates very high plasticity and greater than 90% indicates extremely high plasticity (Whitlow, 1995). On the basis of this, Sample C is of low plasticity, Sample B& D is of intermediate plasticity while Sample A is of high plasticity.

Liquid limit is an important index property since it is correlated with various engineering properties (Ige, 2010). The Atterberg limit tests on soil samples from the site revealed that the following variations: liquid limits; 29 – 52, plastic limits; 12 – 29 and plasticity indices; 17 – 28. According to FMWH (1997), subgrade/fill material should have liquid limit ≤50% and plasticity index ≤30% while for sub-base, liquid limit should be ≤ 30% and plasticity index ≤ 12%. Also, according to Wright (1986), the liquid limit values of 40% and above are assumed high in pavement construction. They pointed out further that plasticity index value of 10% and above are also assumed high in pavement design. All the soils meet the requirement for use as subgrade/ fill materials, although the Liquid limit of Sample A is slightly higher than the standard. While Sample C is the only one suitable as Sub-base material, although its plasticity index is higher than the standard.

Atterberg limits are also important in the selection of materials for use as liners in landfill systems. Benson et al. (1994) recommended that the liquid limit of the liner material must be at least 20%. Most of the specification for soil liners proposed by various researchers or waste regulatory agencies does not generally prescribe any limit (maximum value) for the liquid limit (Ige and Ogunsanwo, 2009; Ige 2010). As long as it does not create any working problem, soils with high liquid limit are generally preferable because of their low hydraulic conductivity (Ige, 2010). The liquid limit of all the studied soil is higher than the minimum prescribed value. The value also falls within the range obtained by Ige (2010) and Ige et al. (2014) and hence could be promising for use as barriers in landfill systems.

### 4.2.3 Compaction Tests

Compaction tests carried out on samples using the standard proctor method indicate maximum dry densities of 1.73$gcm^{-3}$, 1.71 $gcm^{-3}$, 1.79 $gcm^{-3}$ and 1.56 $gcm^{-3}$, for soil samples obtained from Trial Pits A, B, C and D respectively. Similarly, corresponding optimum moisture contents are 19.05%, 19.80%, 14.90% and 15.40% respectively. While using the Modified Proctor Method, the tests indicated maximum dry densities of

$1.95gcm^{-3}$, $1.92gcm^{-3}$, $1.99gcm^{-3}$ and $1.87gcm^{-3}$ for soil samples obtained from the various trial pits A, B, C and D respectively; of which the corresponding Optimum moisture contents are.18.20%, 20.20%, 15.70% and 16.90% respectively. The soil of sample B showed the highest moisture content of 19.80%, at standard proctor energy and 20.20% at modified proctor energy. The compaction curves are single peaked and parabolic in shape.

Compaction places soils in a denser state and hence decreases further settlement, increases shear strength and decreases permeability. It is possible to control the dry density and moisture content so that the soils produced exhibit most of the properties desired, such as consolidation, permeability, etc.(Gidigasu, 1976). The compaction values of these soils are considered good, if 100% of the MDD and OMC are attained during field compaction (Ogundipe, 2012). The relatively good values of the compaction properties possessed by these soils makes them good engineering construction materials.

In general, they will be suitable for use as fill materials and sub-grade materials in road construction. As materials in Highway construction, soils can be used either in the base, sub-base, and subgrade sections. Since, compaction is achieved in construction with the use of rollers, the standard proctor test gives a possible minimum set of values while the modified proctor test gives a possible maximum set of values. During construction, compaction is equally dependent on the available construction machinery and the number of passes made, asides the nature of soil and its moisture content.

In the construction of barriers, compaction is done to achieve a soil layer of improved engineering properties. Barriers are natural soils used in sanitary landfills to prevent the migration of waste leachate into groundwater body. The barrier is placed within the top sealing system to prevent percolation of run-off and precipitation into the waste column and within the bottom sealing system to prevent migration of generated leachate into the groundwater bodies. Although hydraulic conductivity is the key design parameter when evaluating the acceptability of a barrier material. According to Taha et.al. (2003), Barriers should have a maximum dry density of at least $1.71gcm^{-3}$ at a high compactive effort (modified proctor), while Kabir and Taha (2006) suggested a MDD > $1.6\ gcm^{-3}$, but their liquid limit should be greater than 30 while the plasticity index should be above 15. Soil samples A, B& D at the energy of the modified proctor showed MDD within this range and will therefore be useful as landfill barrier materials.

Compaction is the least expensive methods of soil improvement such that it applies for both cohesionless and cohesive soils. In projects where excavation and replacement are confined to a narrow site, only tampers and surface vibrators may be used. On the other hand, if the whole area of the project is to be excavated and replaced in layers suitable roller types of heavy equipment can be used.

Compaction of soil results in homogenous mass that is free of large, continuous inter-clods voids; increase their density and strength, and reduce their hydraulic conductivity. The soils in Ikole-ekiti can be used to construct earthen embankment dams either via zoning or homogenous strata which incorporates the use of drain and toe filters. The soils are to be placed in 0.15m to 0.3m layers thick, with 4 to 6 passes of a 10 to 15 Ton steel-wheeled vibratory rollers.

### 4.2.4 Strength Tests

A. *Direct Shear Test:* The test was carried out on 3 undisturbed samples (A, B and D) and one remoulded sample (C). Sample A has a cohesion of 15 $kN/m^2$ with an angle of internal friction of $16.22^o$, sample B has a cohesion of 11 $kN/m^2$ with an angle of internal friction of $17.95^o$, while for sample C has a cohesion of $74.41kN/m^2$ with an angle of internal friction of $15.26^o$ and sample D has a cohesion of $26.21$ $kN/m^2$ with an angle of internal friction of $18.44^o$. The soil sample D was remoulded and closely packed into the shear box, as disturbed samples were used for the research which is why the direct shear value was very high. It predicted the shear strength parameters quickly. The values are used while designing based on residual shear strengths because of the longer travel distances. These can be used values are sufficient in the design of earth dams as such dams resisting thrust action by internal friction.

B. *Consolidated drained Triaxial Compression Test:* The CD test was carried out on samples A, B& D obtained from the superficial layer (i.e. 1.5m to 2.0m). The tests produced drained cohesion (50.1 – 52.8) $kN/m^2$ and drained angle of internal friction $18^o$ . This shows that soils in the area have cohesive and frictional properties, an indication of their residual origin. Table 4.34 gives a summary of the triaxial test results while the Mohr circle stress plots are shown in Figures 4.5, 4.10& 4.19. The bearing capacity for shallow foundations can be estimated from these given parameters. The use of shallow spread footings (pad, strip, and or raft foundation) is quite feasible for loads of the order of $20kN/m^2$– $53\ kN/m^2$ . Deep foundations like piles are recommended for projects with higher structural loads but would require further investigations at depth below 2 metres. In any case proper subsurface investigation should be carried out before the commencement of the project. The drained shear strength is used in stability analyses for loading conditions during which shear-induced pore pressures are zero, due to the slow application of the shear stresses by the loading with respect to the permeability and drainage boundary conditions of the materials involved.

The bearing capacity of the soil can be determined using Terzaghi's equation for ultimate bearing capacity ($q_u$) for a strip footing of breadth $B$ and depth $Z$, soil cohesion $C$ and unit weight of soil $\gamma$ which is given by:

$$q_u = cN_c + \gamma ZN_q + 0.5\gamma BN_\gamma \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (Eq\ 4.1)$$

Given that the average unit weight ($\gamma$) of soils in Ikole metropolis (Bada, 2018) is$18\ kN/m^3$, and for $\emptyset = 18^o$, $N_c = 13.10$, $N_q = 5.26$ and $N_\gamma = 2.009$. For the construction of a strip footing, at a depth (Z) of 1.5m below ground surface with a width $B$ of 0.675m, for a cohesion of $50.1\ kN/m^2$: The minimum bearing capacity is $810.53\ kN/m^2$. While at a factor of safety of 3, the safe baring capacity is $270.18\ kN/m^2$.

C. *Unconfined Compression Test*: The test was carried out on two soil samples each of A, B& D obtained from the required layer. It is a form of unconsolidated undrained (UU) test where the lateral confining pressure is equal to the zero (atmospheric pressure). It was used to measure the undrained shear strength. The UCS tests produced an average of $28.64kN/m^2$, $47.43kN/m^2$ and $21.73kN/m^2$ for samples A, B and D respectively. These values from these samples exceeds the minimum strength value ($18\ kN/m^2$) required for use of soils as bricks materials

Direct shear results and the seepage parameters are to be used as input data for stability analyses and should represent the range and variations that exist in foundation abutments and embankment materials. The selection of the proper input parameters and their correct use in stability analysis are generally of greater importance than the specific method of stability analysis used. The shear strength available to resist failure along any particular failure surface depends on the loading conditions applied, and the rate of change of the loading conditions. Shear strength values used in stability analysis for these loading conditions depend on consolidation conditions and on the shear-induced excess pore pressures generated by the loadings.

These (shear strength parameters) values can be used in determining the critical height of a slope given that the soil is to be used homogenously, at a particular angle of β to the horizontal with respect to the Taylor's stability number $m$, which is given from charts.

$$H_{cr} = \frac{c}{\gamma m} \dots\dots (Eq\ 4.2)$$

Also, these strength parameters can also be used to determine respective factor of safety ($F_s$) with respect to strength and sliding, where $\gamma_{sat}$ is the saturated unit weight.

$$F_s = \frac{c}{\gamma_{sat}\ H\ cos^2\beta \tan\beta} + \frac{\gamma \tan\emptyset}{\gamma_{sat} \tan\beta} \dots\dots (Eq\ 4.3)$$

It was observed that in the direct shear test, the failure plane is horizontal. This forced failure plane can sometimes coincide with the natural layering or planes of weakness. While for the triaxial test, the failure plane is not predetermined by the configuration of the testing apparatus and horizontal planes of weakness would not be expected to control the measured shear strength. Therefore, for each embankment and foundation layer, design shear strength should be selected such that two-thirds of the test values exceed the design value.

# CHAPTER FIVE

# CONCLUSION AND RECOMMENDATIONS

## 5.1 Conclusion

The strength and compaction properties of Ikole-Ekiti soils were investigated. The study showed that lateritic soils developed over most of the areas in Ikole-ekiti excluding areas bordering the Elekole's Palace (Yeye-Oba), also because they possess cohesive soil properties, i.e. they have an high degree of laterization, for they are well graded and very low in fines. All the soils in the study areas have low potential of water retention with their natural moisture content not exceeding 23.17%.

Differential Settlements or other forms of settlement are not anticipated if imposed structural loads are not more than 53 $kN/m^2$. The soils from trial pits A, B, C and D are grouped into A-2-7, A-2-7, A-2-6 and A-2-7 respectively, according to AASHTO classification system (1986). While according to the Unified Soil Classification System (USCS), samples A, B, C and D are classified as SW, SP, SW-OL and SW-SC respectively. All the soil samples tested in this research can be used as Subgrade in Highway and airfield construction while only sample C can be used as Subbase in such constructions, other samples (A, B& D) can as well be used but would require stabilization with additives.

Maximum compaction of soil samples can be achieved at the obtained MDD and respective OMC's. At standard proctor energy, 1.73$gcm^{-3}$, 1.71 $gcm^{-3}$, 1.79 $gcm^{-3}$ and 1.56 $gcm^{-3}$ were the maximum dry densities, for soil samples obtained from Trial Pits A, B, C and D respectively. Similarly, corresponding optimum moisture contents are 19.05%, 19.80%, 14.90% and 15.40% respectively. While at the Modified Proctor energy, the tests indicated maximum dry densities of 1.95$gcm^{-3}$, 1.92$gcm^{-3}$, 1.99$gcm^{-3}$ and 1.87$gcm^{-3}$ for soil samples obtained from the various trial pits A, B, C and D respectively; of which the corresponding Optimum moisture contents are.18.20%, 20.20%, 15.70% and 16.90% respectively. Soil samples A, B& D can be used as landfill barriers since the compaction values exceeds the minimum requirements in previous works. Also in the construction of earth dams, zoning can be employed or the dam can be built up homogenously with the use of steel-wheeled vibratory rollers.

The values gotten from Strength tests can be used to calculate bearing capacities of different shapes of footing for various soils in Ikole metropolis. Also for the construction of homogenous dams, direct shear values are used in design. The knowledge of shear strength is required in order to assess waste slope stability. Also, the soil samples A, B& D exceeds the required value for minimum value for use as brick materials.

Also, it is observed that the shear strength of a soil increases with the amount of compaction applied. The more the soil is compacted, the greater is the value of cohesion and the angle of shearing resistance (internal friction). By comparing the shearing strength with the moisture content for a given degree of compaction, it is found that the greatest shear strength is attained at a moisture content lower than the optimum moisture content for maximum dry unit weight. It might be inferred that it would be an advantage to carry out compaction at the lower value of the moisture content. However, soils compacted this way tend to take up more moisture and become saturated with a consequent loss of strength.

## 5.2 Recommendations

Development occurs due to a rise in civilization, in this case, Civil Infrastructure. For this study, not all areas in Ikole-Ekiti Township were studied for this research, as the exploration didn't go occur in Isaba, Usin, Temidire and Ootunja areas of the town. I recommend for further studies, the areas earlier mentioned should be worked upon to have a full overview of the whole town.

Further studies (finite element modeling) are to be carried out to evaluate the impact an improvement in the MDD and lowering of the OMC (Compaction data) could have in terms of savings from the reduction in thickness of pavement surfacing material by adding locally-sourced materials so as to stabilize it. It is recommended that, future research on the soil samples of the study area should also include permeability (hydraulic conductivity) and consolidation, as this would greatly help in the design and construction of earth dams as these form of dams resist pressure by the internal friction of the soil particles. Also for the construction of high-rise residential hostels foundations, extensive research should be done at depths beyond those reached in this project.

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