GEOTECHNICAL CLASSIFICATION AND CHARACTERIZATION OF SOILS IN FEDERAL UNIVERSITY OYE-EKITI IKOLE CAMPUS

BY

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(CVE/13/1050)

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A project report submitted to the department Of Civil Engineering Federal University Oye Ekiti in partial fulfillment of the requirements for the award of the B. Eng. (Hons) In Civil Engineering.

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ABSTRACT

This research project investigates the geotechnical characteristics of soils in Federal University Oye Ekiti, Ikole campus. In this research project, disturbed soil samples were obtained from five labelled TP 1, TP 2, TP 3, TP 4, TP 5 at different locations and at various intervals along the university and were taken to laboratory for relevant soil engineering tests. The relevant soil tests include; particle size analysis, consistency limits (liquid, plastic, and shrinkage limits), natural moisture contents, specific gravity, compaction test, direct shear test, CBR test. There were provisions for spacing sampling to obtain variation of soil properties. Compaction test showed a maximum dry density range of 1.55kg/m^3 to 1.75kg/m^3 and an optimum moisture content range between 17% and 22.5%. The values of the California Bearing Ratio (CBR) of un-soaked soil samples were within the range of 15.5% and 81.7%. Furthermore, the sieve analysis revealed that a substantial percentage of the soil samples passed through the No. 200 BS sieve suggesting the soil consist mostly of silty clayey material, which translates to a fair material rating according to AASHTO design standard (1986) and may require stabilization before a construction is commenced in such areas.

DEDICATION

I dedicate this work to GOD almighty my creator, my strong pillar, my source of inspiration, the reason behind my breath... 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 report was written by ADETUNMBI AYOOLUWA. J (CVE/13/1050) 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 partial requirements for the award of Bachelor of Engineering (B.Eng.) degree in Civil Engineering, Federal University Oye Ekiti.

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CHAPTER ONE

INTRODUCTION

1.1 General Background

The long-term performance of any construction project depends on the soundness of the underlying soils. Construction works often encounter problems originating from weak engineering properties of underlying soils such as low bearing capacity, excessive settlements and ground movements. It is therefore of paramount importance that there be an in-depth investigation on the subsoil conditions so as to provide a detailed geological soil report of the geographical location or the study area, hence, the reason for this study. The design of civil engineering projects requires the determination of physical, mechanical and strength characteristics of in-situ soils. Successful engineering projects often involve the use of engineering principles in the appropriate manner which in turn answers concerns such as safety and economy. Engineering geologist, geotechnical engineers, geomorphologist among other professionals play an integral role in modern engineering project this is because report on geotechnical analysis make them aware of problem- soil with a view to avoid structural failure, defects or collapse of civil engineering projects. Kekere et al (2012) proposed that the behaviour of a structure depends upon the properties of the soil materials on which the structure rests. The properties of the soil materials depend upon the properties of the rocks from which they are derived. A brief discussion of the parent rocks is, therefore, quite essential in order to understand the properties of soil materials, Murthy (2004).

In terms of soil texture, soil type usually refers to the different sizes of mineral particles in a particular sample. Soil is made up in part of finely ground rock particles, grouped according to size as sand and silt in addition to clay, organic material such as decomposed plant matter. Each component, and their size, plays an important role. For example, sand, determine aeration

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and drainage characteristics, while the tiniest, sub-microscopic clay particles are chemically active, binding with water and plant nutrients. The ratio of these sizes determines soil type: clay, loam, clay-loam, silt-loam, and so on. In addition to the mineral composition of soil, humus (organic material) also plays an important role in soil characteristics and fertility for plant life. Soil may be mixed with larger aggregate, such as pebbles or gravel.

1.2 Statement of Problem

The crucial role of soil cannot be over emphasized in the construction of Civil engineering projects such as buildings, roads etc. Poor engineering properties of soil has detrimentally affected so many structures and FUOYE being a developing school in terms of infrastructure, would often indulge in some civil engineering construction projects hence, there is need for geotechnical evaluation of the soil before any civil engineering construction.

1.3 Aim and Objectives

This study intends to appraise the geotechnical characteristics of soil in Federal University Oye Ekiti, Ikole campus Ekiti state.

To achieve this stated aim, the objectives are:

1. To obtain soil samples at 5 different locations
2. To subject the soil samples to certain geotechnical tests
3. To assess geotechnical properties of the studied soil samples
4. To determine the class of the studied soil samples
5. To ascertain the geotechnical status of the studied area which may be useful for future construction works within the university campus
6. To draw informed conclusion and give recommendations based on the results obtained from the tests

1.4 Significance Of Research

The study is considered to be very important as it will investigate the properties of soil in Federal University, Oye-Ekiti Ikole Campus and signify the classification of the soil, which in turns guides the use of the soil. This research work would aid future works on the soils of this area perhaps with respect to research, construction uses, etc.

1.5 Scope and Limitations of Study

The samples of disturbed soils will be collected from five (5) locations within the university environment and will be subjected to the following tests;

1. **Index properties tests;**

a. Specific gravity

b. Sieve analysis

c. Moisture content

d. Atterberg limits

2. **Strength tests;**

a. Compaction test

b. California Bearing Ratio (CBR) test

c. Direct Shear Test

The research is limited to only five (5) locations for the whole area of the University campus namely; School main gate, Faculty of Agriculture, Faculty of Engineering, School Hostel, and School Health Centre.

1.6 Description of Study Area

The study area is situated at the Federal University Oye Ikole campus, Ikole local government area of Ekiti state, Ikole is located between longitude 5°30'52.17" East of Greenwich and latitude 7°47'53.76" North of the Equator (distancetos.com). Ikole-Ekiti is the Headquarters of the old Ikole District Council, the defunct Ekiti North Division and the Headquarters of defunct Ekiti North Local Government and now Headquarter of Ikole Local Government. Ikole is about 65 kilometres from Ado Ekiti, the capital of Ekiti State of Nigeria. The town is situated on a very plain and well-drained land on the northern part of the State – about 40 kilometres from the boundary of Kwara State. The population of the town according to the 1963 census is about 52,000. The town is gifted with good fertile farmlands which ensure future expansion of agriculture and allied industries as well as a high swell in its population growth.

The geology of the area is underlain by the Precambrian rocks of the basement complex of southwestern Nigeria which covers about 50% of the land surface in Nigeria (Ekiti State Government,2017). The basement rocks show great variation in size and in mineral composition, Oladapo and Ayeni (2013).

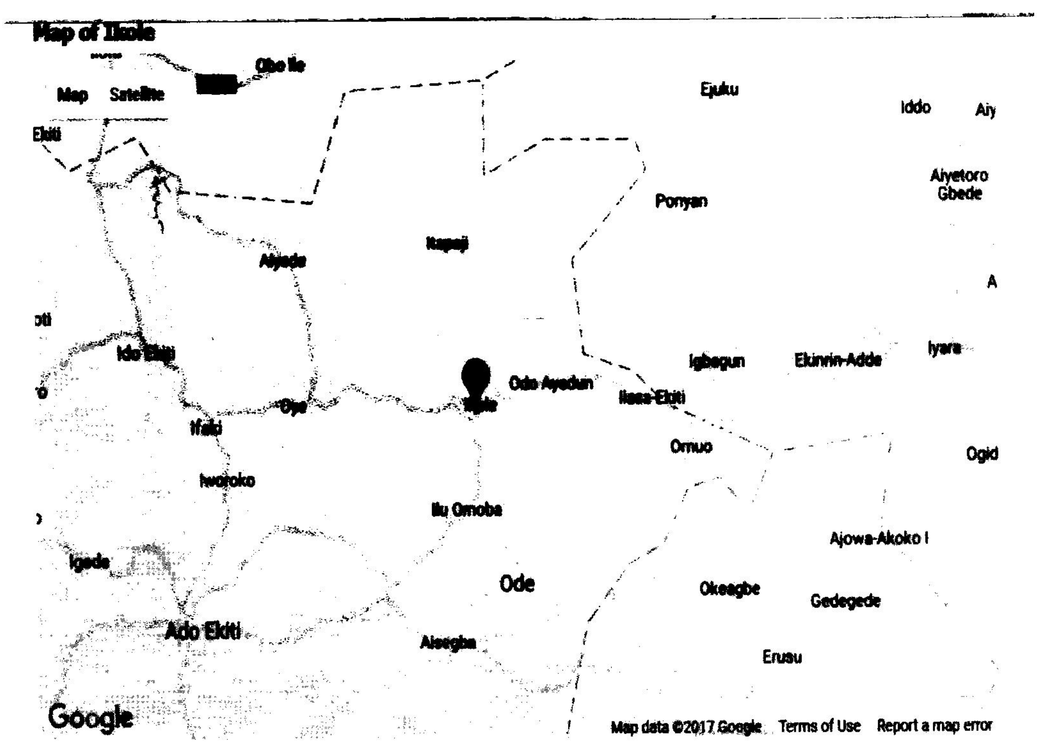

Figure 1.1: Map indicating study area (Digitized from Ademilua 2014)

CHAPTER TWO

LITERATURE REVIEW

2.1 Overview

A site investigation or soil survey is an essential part of the preliminary design work on any important structure in order to obtain information regarding the sequence of soil strata and the ground water level and also to collect samples for identification and testing. According to Adeyeri (2015); 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.

2.2 Soil

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

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

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

2.3 The Origin of Soils

Soil can be defined as an assemblage of nonmetallic solid particles (mineral grains), and it consists of three phases: solid, liquid (water), and gas (air). Commonly used terms such as

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gravel, sand, silt, and clay are the names of soils based on their particle grain sizes. The names quartz, mica, feldspar, etc. are based on their crystal names. The rock cycle in Figure 2.1 illustrates the origins of a variety of soils on the earth, Isao and Hemantha (2015). In the investigation of Knapett and Craig (2012), soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks. Weak cementation can be due to carbonates or oxides precipitated between the particles, or due to organic matter. Subsequent deposition and compression of soils, combined with cementation between particles, transforms soil into sedimentary rocks (a process known as lithification). If the products of weathering remain at their original location, they constitute a residual soil. If the products are transported and deposited in a different location they constitute a transported soil, the agents of transportation being gravity, wind, water and glaciers. During transportation, the size and shape of particles can undergo change and the particles can be sorted into specific size ranges. In their own work, Isao and Hemantha (2015) said particle sizes in soils can vary from over 100 mm to less than 0.001mm. Figure 2.1 explains the rock cycle.

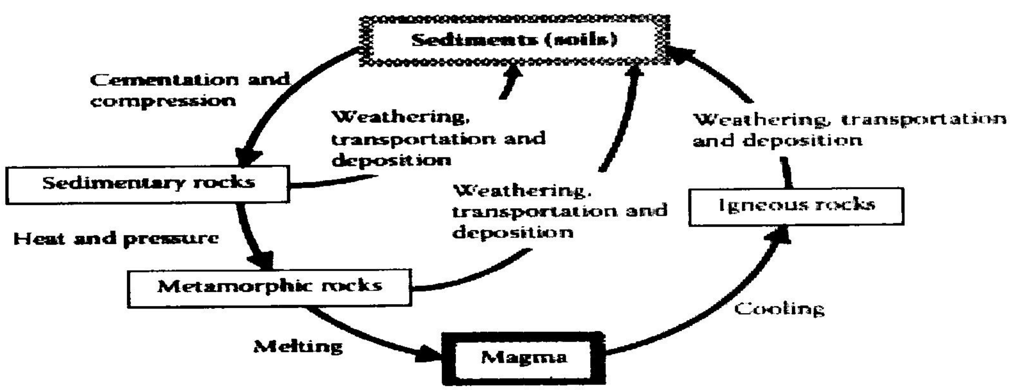

FIGURE 2.1 Rock cycle.

Source: BS 1924: Part 1: 1990

2.4 Soil Description and Classification

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

In the investigation conducted by McCarthy (1982), a description of mass characteristics should include an assessment of in-situ compactive state (coarse soils) or stiffness (fine soils) and details of any bedding, discontinuities and weathering. The arrangement of minor geological details, referred to as the soil macro-fabric, should be carefully described, as this can influence the engineering behaviour of the in-situ soil to a considerable extent. Examples of macro-fabric features are thin layers of fine sand and silt in clay, silt-filled fissures in clay, small lenses of clay in sand, organic inclusions and root holes. The name of the geological formation, if definitely known, should be included in the description; in addition, the type of deposit may be stated (e.g. till, alluvium, river terrace), as this can indicate, in a general way, the likely behaviour of the soil. It is important to distinguish between soil description and soil classification.

Cernica (1995) also noted that soil description includes details of both material and mass characteristics, and therefore it is unlikely that any two soils will have identical descriptions. In soil classification, on the other hand, a soil is allocated to one of a limited number of groups on the basis of material characteristics only. Soil classification is thus independent of the in-situ condition of the soil mass. If the soil is to be employed in its undisturbed condition, for example to support a foundation, a full soil description will be adequate and the addition of the soil classification is discretionary. However, classification is particularly useful if the soil in question is to be used as a construction material, for example in an embankment. Engineers can also draw on past experience of the behaviour of soils of similar classification according to Townsend (1973). Figure 2.2 describes the stratigraphic information of the soil profile.

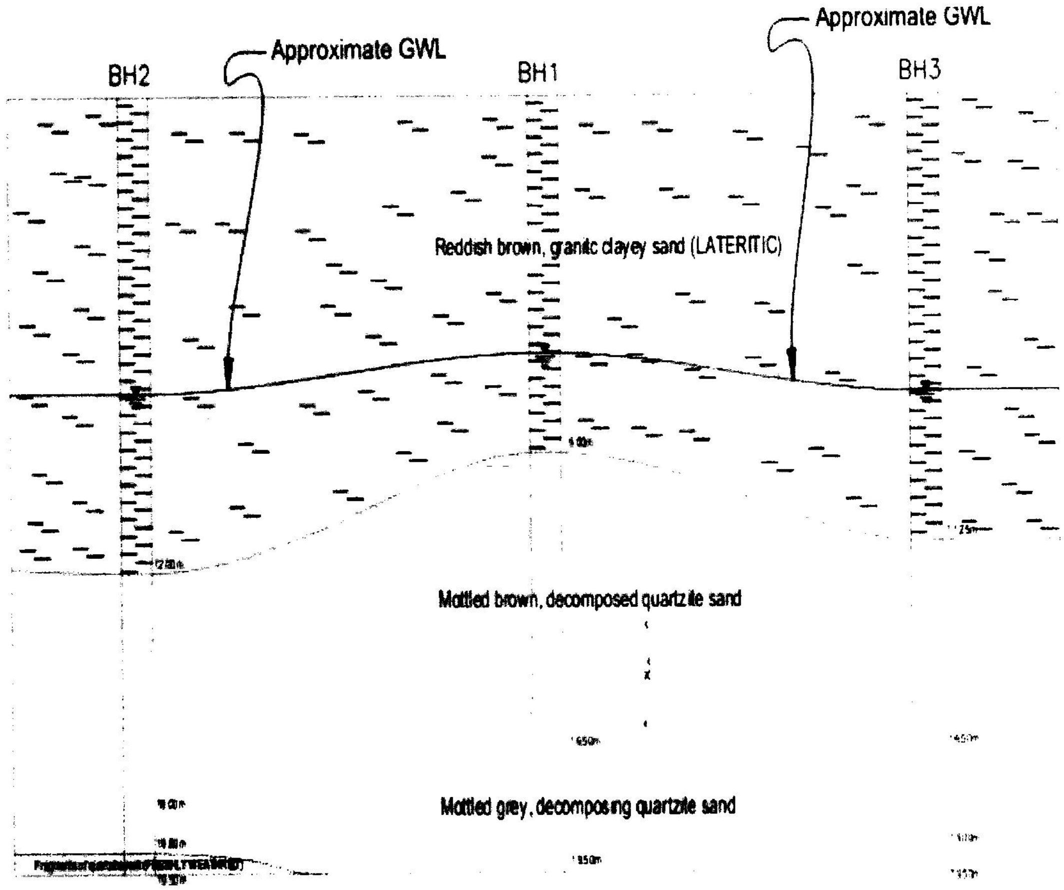

Figure 2.2 Stratigraphic Description of the Soil Profile Bolarinwa et al (2017)

2.5 Lateritic Soils

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

2.5.1 Formation of lateritic soils

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

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

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

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

2.5.2 Chemical and mineralogical composition

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

2.5.3 Colour

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

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

2.5.4 Geotechnical properties of lateritic soils

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

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

2.5.4.1 Particle size distribution

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

(1) A basis for identification and classification of the soil

(2) The compactibility characteristics

(3) Permeability

(4) Sweallability and

(5) A rough idea of deformation characteristics of the soil mass.

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

2.6 Geotechnical Properties Of Soil

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

2.6.1 Soil Sampling

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

2.6.2 Geotechnical and Physical Properties of Soil

2.6.2.1 Strength

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

1. Bearing resistance
2. Soil compressibility; and
3. Soil compactability.

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

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

2.6.2.2 Compaction

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

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

2.6.2.3 Collapse and Swelling

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

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

2.6.2.4 Particle size

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

Table 2.1: Classification of materials based on particle size distribution

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

Source: BS 1924: Part 1: 1990

Table 2.2: Soil classifications and properties

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

Source: Townsend, 1973

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

2.6.2.5 Cohesion and Plasticity

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

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

The transition points are fairly arbitrary, determined by index tests described in BS 1377-2:1990, but they do serve a valuable function in the classification of cohesive soils. With an increase in moisture content, granular soils pass rapidly from a solid to a fluid condition. In these circumstances the PL and LL cannot be identified and such soils are classified as non-plastic; Sherwood (1993). Figure 2.3 describes the soil plasticity.

Increasing Moisture Content ⟶

Powdered condition   Plastic condition   Liquid condition

P   L

L ⟵ Plasticity ⟶ L

Index (PI)

PI = LL – PL

Figure 2.3: Definitions of soil plasticity; Sherwood (1993)

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

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

2.7 Classification of Soils

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

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

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

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

l. Grain size

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

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

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

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

3. If cobbles and boulders (size larger than 75 mm) are encountered, they are excluded from the portion of the soil sample from which classification is made. However, the percentage of such material is recorded. Table 2.3 displays the classification system using AASHTO

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

General classification	**Granular materials (35% or less of total sample passing No. 200 sieve)**						
	A-1			**A-2**			
Group classification	**A-1-a**	**A-1-b**	**A-3**	**A-2-4**	**A-2-5**	**A-2-6**	**A-2-7**
Sieve analysis (% passing)							
No. 10 sieve	50 max						
No. 40 sieve	30 max	50 max	51 min				
No. 200 sieve	15 max	25 max	10 max	35 max	35 max	35 max	35 max
For fraction passing No. 40 sieve							
Liquid limit (LL)				40 max	41 min	40 max	41 min
Plasticity index (PI)	6 max		Nonplastic	10 max	10 max	11 min	11 min
Usual type of material	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
Subgrade rating	Excellent to good						

General classification	**Silt-clay materials (More than 35% of total sample passing No. 200 sieve)**			
Group classification	**A-4**	**A-5**	**A-6**	**A-7** A-7-5[a] A-7-6[b]
Sieve analysis (% passing)				
No. 10 sieve				
No. 40 sieve				
No. 200 sieve	36 min	36 min	36 min	36 min
For fraction passing No. 40 sieve				
Liquid limit (LL)	40 max	41 min	40 max	41 min
Plasticity index (PI)	10 max	10 max	11 min	11 min
Usual types of material	Mostly silty soils		Mostly clayey soils	
Subgrade rating	Fair to poor			

[a]If PI ≤ LL − 30, the classification is A-7-5.
[b]If PI > LL − 30, the classification is A-7-6.

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

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

There are three broad types under which the AASHTO groups and subgroups are divided. These are "granular" (A-1, A-3, and A-2), "silt-clay" (A-4 through A-7), and highly organic (A-8) materials. The transitional group, A-2, includes soils which exhibit the characteristics of both granular and silt-clay soils, making subdivision of the group necessary for adequate identification of material properties. Figure 2.4 explains the relationship between liquid limit and plasticity index for a silt-clay material.

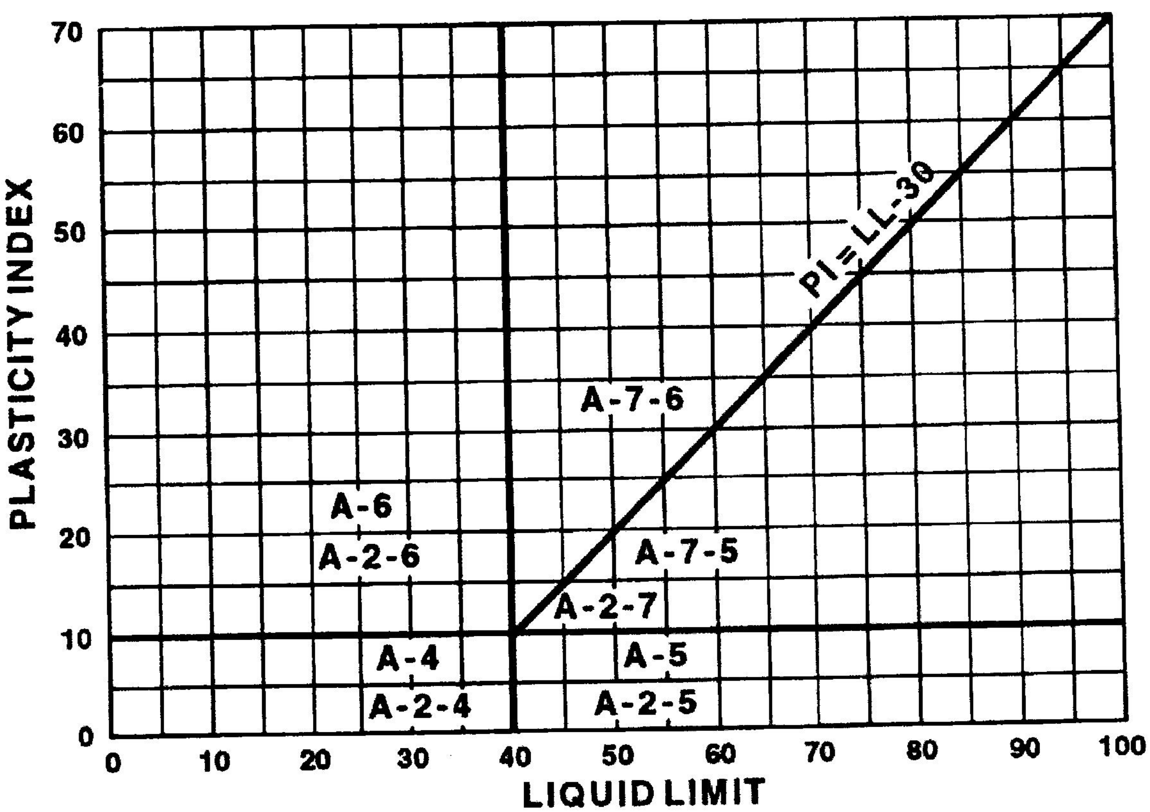

Figure 2.4: Relationship between liquid limit and plasticity index for silt-clay groups (AASHTO M 145-91).

2.7.2 Unified Soil Classification System

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

Table 2.4 shows the USCS classification system along with the criteria utilized for associating the group symbol, such as "CL," with the soil. In this chart, D_{60} refers to the diameter of the soil particles that 60 percent of the sample would pass on a sieve, as indicated on the gradation curve. Similarly, D_{10} relates to the maximum diameter of the smallest 10 percent, by weight. Figure 2.5 describes the Unified Soil Classification System

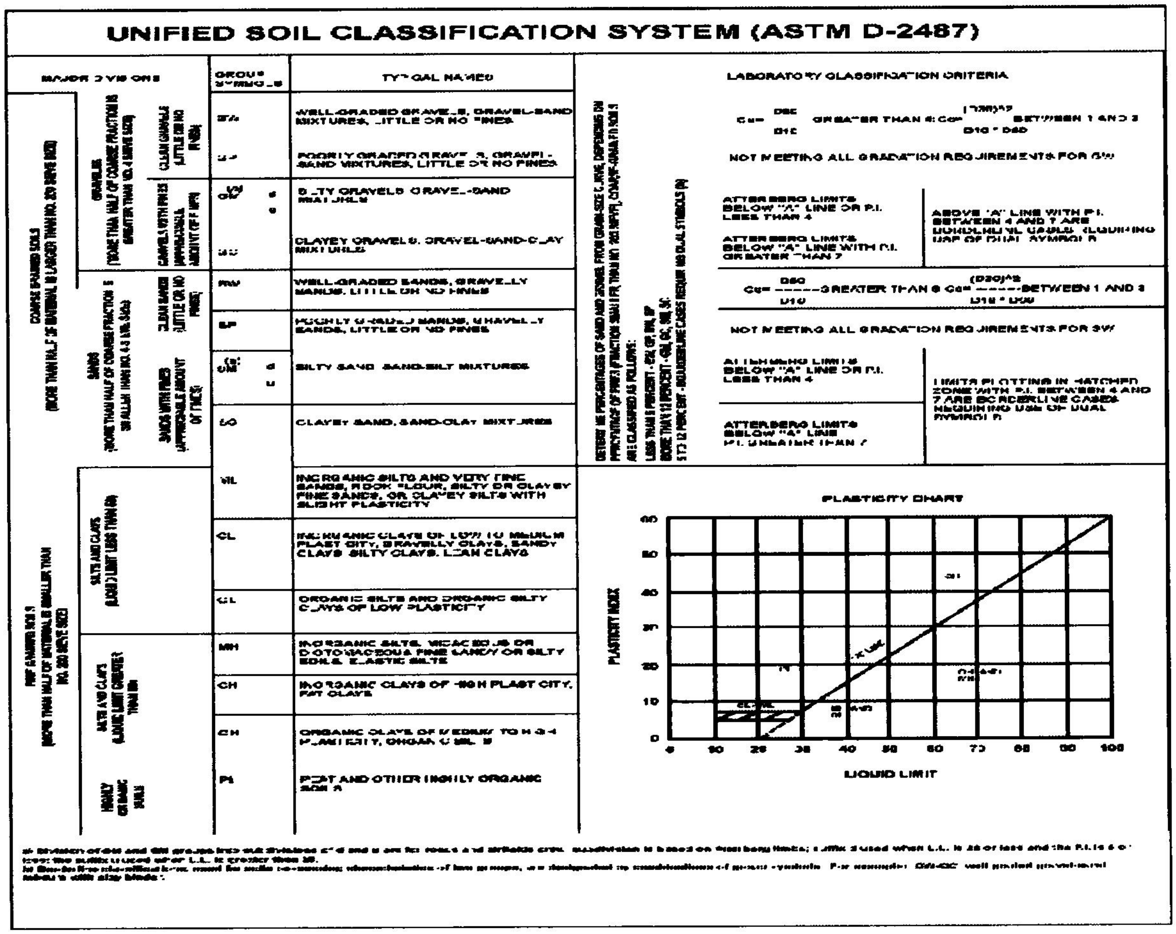

Figure 2.5: Unified Soil Classification System chart (after U.S. Army Corps of Engineers, Waterways Experiment Station, TM3-357, 1953).

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

2.7.3 Correlation of the Classification Systems

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

CHAPTER THREE

METHODOLOGY

3.1 Preamble

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

3.2 Field work

In order to carry out the geotechnical examination work, a trial pit was dug at the locations chosen for collection of soil samples. Basically the scope of field work involves; the exploration of five trial pits by using digger and shovel for digging technique. Disturbed soil samples were collected below the formation level of about 1.0 metre depth below the existing ground level and the overlying soil material as well as the top soil was discarded. The soil samples were contained in covered and labelled plastic bags and taken to the laboratory for tests

3.3 SAMPLING OF MATERIALS

In order to carry out the geotechnical examination work, a borehole was sunk at the locations chosen for collection of soil sample.

Disturbed soil samples were collected below the formation level of about 1.0 metre depth below the existing ground level and the overlying soil material as well as the top soil was discarded. The soil samples were contained in covered and labelled sacks and taken to the laboratory for tests. The coordinates of the location are given below as contained in Table 3.1

Table 3.1: Coordinates of Locations

Location	Coordinate in degrees		Coordinate in metrics(m)	
	Northings	Eastings	Northings	Eastings
TP 1	07^{O} 48.288'	005^{O} 29.766'	827792	753575
TP 2	07^{O} 48.317'	005^{O} 29.707'	827824	753510
TP 3	07^{O} 48.364'	005^{O} 29.698'	827876	753499
TP 4	07^{O} 48.427'	005^{O} 29.642'	827945	753437
TP 5	07^{O} 48.454'	005^{O} 29.685'	827975	753484

Sampling is one of the major operations in laboratory works. It is the initial beginning that could be regarded as the foundational work. If wrong method is used, it may drastically affect the laboratory analysis and results that may lead to erroneous conclusion hence, optimum consideration and attention was given to it.

3.3.1 Methods of Collecting Samples

Generally, there are two main methods of collecting samples namely:

i. Disturbed sampling

ii. Undisturbed sampling

But for the purpose of this research, we shall be discussing the method adopted which is the disturbed sampling method.

Disturbed Sampling

The vegetative layer and the top soil is first removed as it is generally regarded as unsuitable using spade, shovel and digger. Digging is done to the required or specified depth before samples are collected into polythene bag, properly tied to maintain its natural moisture content. This was well labeled and dated for the purpose of easy identification and to prevent mix up in the laboratory.

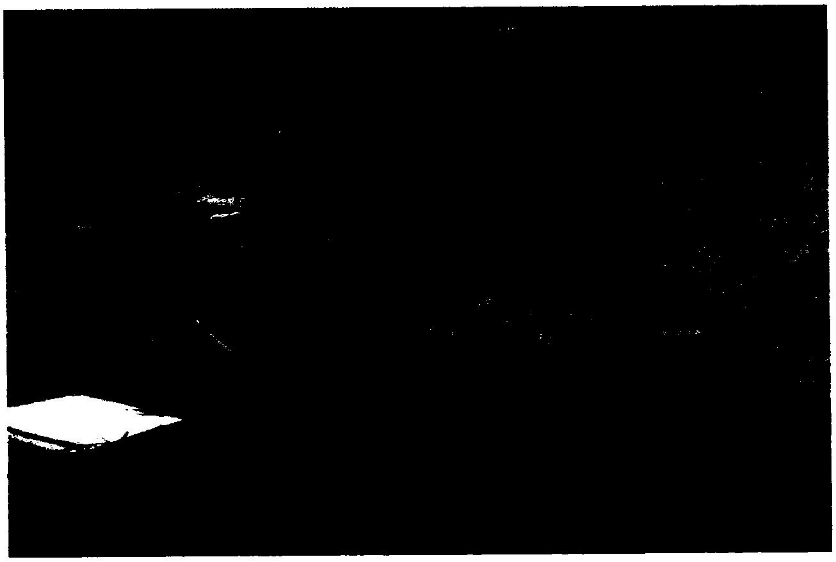

Plate 1: Samples being taken at one of the locations

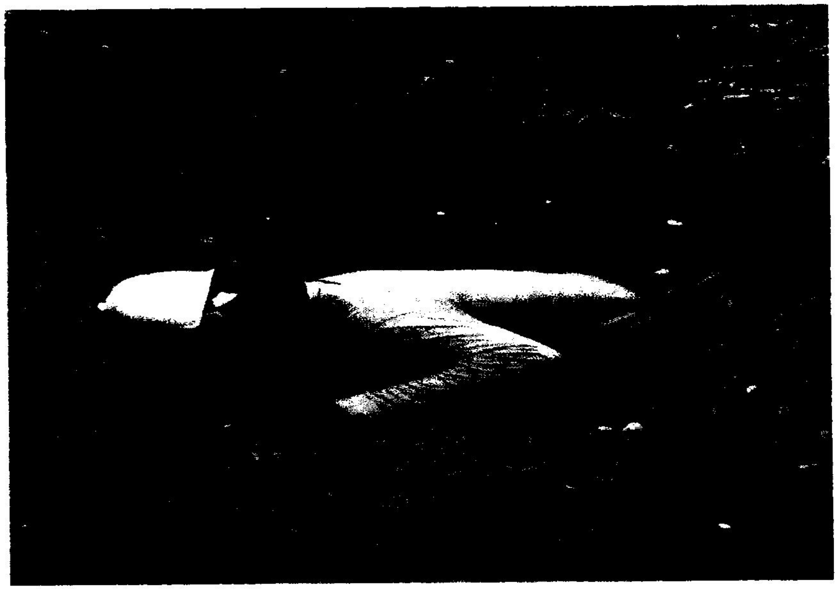

Plate 2: Measurement of the depth of the trial pit with a measuring tape

3.3.2 Sampling Technique

The type of technique that will be adopted for taking the sample is hand dug method while taking samples in the five locations.

3.4 Laboratory Testing

All the laboratory tests would be carried out at the civil engineering laboratory at the Federal Polytechnic Ado-Ekiti to help classify and determination of strength in the collected soil samples. The laboratory analysis will be performed according British standard methods of test for soil for civil engineering purposes; BS 1377: Part 1-9 (1990). Laboratory tests carried out are as follows:

A. Determination of Index properties of soil (classification):

i. Particle size analysis,

ii. Moisture content determination,

iii. Consistency limit test {Atterberg}

iv. Specific gravity test.

B. Determination of Strength properties of soils:

i. Compaction,

ii. Direct shear test.

iii.California Bearing Ratio test

3.5 Methods

3.5.1 Particle Size Distribution

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

Tools

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

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

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

iv. Oven

v. Mechanical shaker

vi. Mortar with rubber pestle

vii. Brushes

viii. Trays

Preparation of Sample

i. Soil sample, as received from the field, was dried in the sun. Tree roots and pieces of bark were removed from the sample.

ii. Care was taken not to break the individual soil particles.

iii. A representative soil sample of required quantity was taken and dried in the oven at 105 to 120°C.

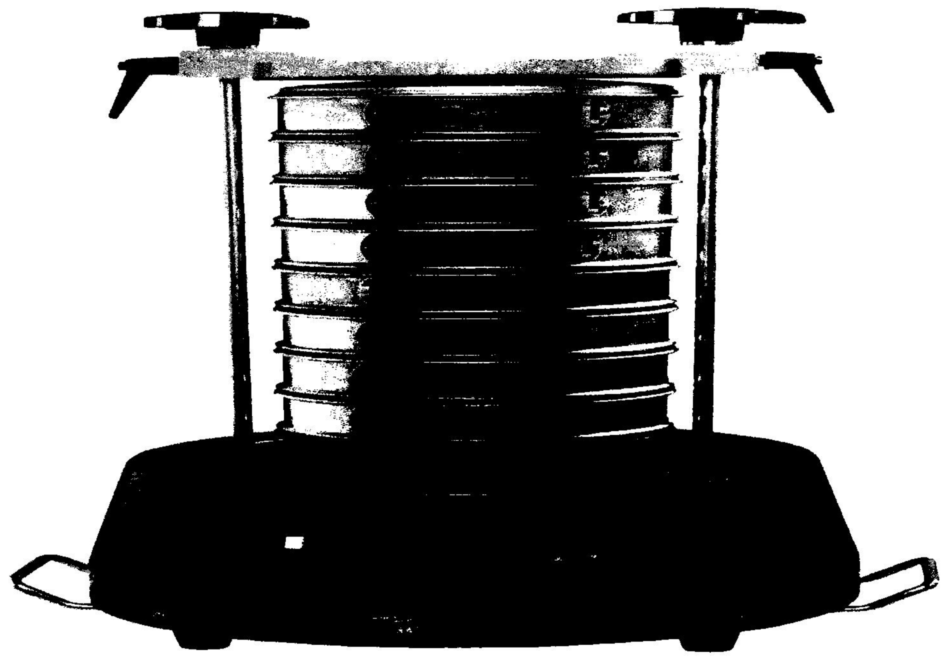

Plate 3 Mechanical Sieve Shaker

Procedure used to determine particle size distribution of soil

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

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

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

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

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

Hydrometer Analysis

i. Particles passed through 75μm IS Sieve along with water were collected and put into a 1000ml jar for hydrometer analysis. More water was added to make the soil water suspension just 1000ml. The suspension in the jar was vigorously shaken horizontally by keeping the jar in-between the palms of the two hands. The jar was then put on the table.

ii. A graduated hydrometer was carefully inserted into the suspension with minimum disturbance.

iii. At different time intervals, the density of the suspension at the centre of gravity of the hydrometer was noted by seeing the depth of sinking of the stem. The temperature of the suspension was noted for each recording of the hydrometer reading.

iv. Hydrometer readings were taken at a time interval of 0.5 minute, 1.0 minute, 2.0 minutes, 4.0 minutes, 15.0 minutes, 45.0 minutes, 90.0 minutes, 3hrs., 6hrs., 24hrs. and 48hrs.

v. By using the nomogram given in IS: 2720 (Part 4) – 1985, the diameter of the particles for different hydrometer readings was found out.

Reporting of Results

After completing mechanical analysis and hydrometer analysis, the results are plotted on a semi-log graph with particle size as abscissa (log scale) and the percentage smaller than the specified diameter as ordinate.

3.5.2 Specific Gravity

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

Tools

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

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

iii) Vacuum desiccator

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

v) Weighing balance, with an accuracy of 0.001g

vi) Spatula

Procedure to Determine the Specific Gravity of Fine-Grained Soil

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

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

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

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

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

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

vii) I repeated the process till no more air bubbles were observed in the soil-water mixture.

viii) I observed the constant temperature in the bottle and recorded.

ix) I inserted the stopper in the density bottle, wiped and weighed as (W_3).

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

3.5.3 Compaction Test

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

1. Standard Proctor test

2. Modified AASHTO method
3. West Africa method

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

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

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

iv) Steel straightedge – 30cm long

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

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

i) A 5kg sample of air-dried soil passing through the 19mm IS Sieve was taken. The sample was mixed thoroughly with a suitable amount of water based on the soil type. The soil sample was stored in a sealed container for a minimum period of 16hrs.

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

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

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

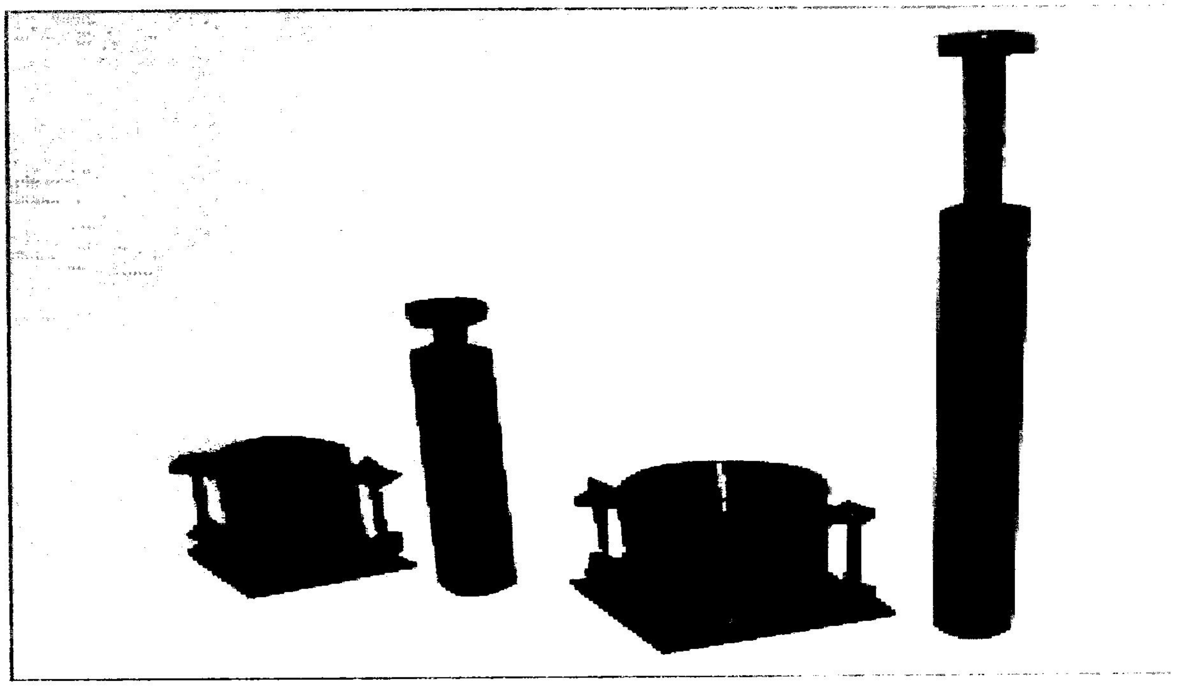

Plate 4: Moulds and Rammers

3.5.4 Plastic Limit Test

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

Tools

i) Porcelain evaporating dish about 120mm dia.

ii) Spatula

iii) Container to determine moisture content

iv) Balance, with an accuracy of 0.01g

v) Oven

vi) Ground glass plate – 20cm x 15cm

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

Procedure to determine the Plastic Limit of Soil

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

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

iii) Repeated the process of alternate rolling and kneading until the thread crumbled.

iv) Collected and kept the pieces of crumbled soil thread in the container used to determine the moisture content.

v) Repeated the process twice more with fresh samples of plastic soil each time.

3.5.5 California Bearing Ratio Test

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

Tools

i) Mould

ii) Steel Cutting collar

iii) Spacer Disc

iv) Surcharge weight

v) Dial gauges

vi) IS Sieves

vii) Penetration Plunger and loading machine

CBR Test Procedure

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

i) Weight of empty mould was recorded

ii) Added water to the first specimen (compacted it in five layer by giving 10 blows per layer)

iii) After compaction, I removed the collar and levelled the surface.

iv) Took samples for determination of moisture content.

v) Weight of mould + compacted specimen was recorded.

vi) Took other samples and applied different blows and repeated the whole process.

vii) After four days, I measured the swell reading and found %age swell.

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

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

x) I applied the load and noted the penetration load values.

3.5.6 Direct Shear Test

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

Tools

i) Direct shear box apparatus

ii) Loading frame (motor attached).

iii) Dial gauge.

iv) Proving ring.

v) Tamper.

vi) Straight edge.

vii) Balance to weigh up to 200 mg.

viii) Spatula.

Procedure

1. Checked the inner dimension of the soil container.
2. Put the parts of the soil container together.
3. Calculated the volume of the container. Weighed the container.
4. Placed the soil in smooth layers (approximately 10 mm thick).

5. Weighed the soil container, the difference of these two is the weight of the soil. I then calculated the density of the soil.

6. Made the surface of the soil plane.

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

8. Measured the thickness of soil specimen.

9. Applied the desired normal load.

10.Removed the shear pin.

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

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

13. Before proceeding to test, I checked all adjustments to see that there was no connection between two parts except sand/soil.

14. Started the motor. Took the reading of the shear force and recorded the reading.

15.Took volume change readings till failure.

16. Added 5 kg normal stress 0.5 kg/cm2 and continued the experiment till failure

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

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

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

This chapter presents the results of the tests carried out on the geotechnical characterization and classification of soils in FUOYE, Ikole campus.

4.1 Atterberg Limit Test

The summary of the atterberg limit for the studied soils are shown on Table 4.1 and 4.2 below.

Table 4.1: Table showing result of Atterberg limits, soil types and index groupings.

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Plasticity type	Soil Type by (USCS)
PIT 1	56.0	19.3	24.7	Medium	Inorganic
PIT 2	51.0	21.9	19.1	Medium	Inorganic
PIT 3	53.2	22.7	20.5	Medium	Inorganic
PIT 4	28.5	17.7	10.8	Low	Inorganic silty clay
PIT 5	55.0	21.1	23.8	Medium	Inorganic

From the above table, the locations have (LL) ranging from 28.5% to 56.0% with plasticity index ranging from 10.8% to 24.7% respectively.

The plasticity was less than 25% which is the maximum recommended value for subgrade tropical soils (Madedor, 1963). The Cassagrande chart indicates low to medium plasticity. Table 4.2 shows the result of percentage of soil passing through sieve 200.

Table 4.2: Table showing result of percentage of soil passing through the sieve 200 with (LL) and (PI) classification using USCS.

Sample Location	% passing sieve 200	Liquid Limit (%)	Plasticity Index (%)	USCS major division	Rating
PIT 1	46.0	44.0	24.7	Fine grained silty clayey soil	Fair to poor
PIT 2	49.7	51.0	19.1	Fine grained silty clayey soil	Fair to poor
PIT 3	69.6	53.2	20.5	Fine grained silty clayey soil	Fair to poor
PIT 4	20.1	28.5	10.8	Silty or Clayey Gravel	Good
PIT 5	59.8	55.0	23.8	Fine grained silty clayey soil	Fair to poor

From the summary results of the table 4.2, the % passing sieve 200 ranges from 20.1% to 69.6% and (LL) liquid limit ranges from 28.5% to 55.0%. All liquid limit values in this study are less than 100% hence, classifying the clay as inorganic and organic soils. Plasticity index ranging from 10.8% to 24.7%, this indicates that the soils are mostly of medium except for

PIT 4 which possesses a low plasticity. This observation was further supported using Cassagrande plasticity chart (1948).

To further support the assertion, also the shrinkage limit falls within 7.1% to 11.6%. According to Brink et al, 1982; Jegede (2004), soils with linear shrinkage value above 8% should be inactive and inexpansible, such will not present field compaction difficulties. Therefore, it shows that the earlier results did not conform with the standard specification earlier mentioned above, only PIT 4 met the specifications.

4.2 Compaction Test

From the laboratory, results of the compaction test shows variation on the Optimum Moisture Content and Maximum Dry Density from the different locations, prior to soil California bearing ratio (CBR) determination, compaction was performed on the soil sample using standard Proctor density test (AASHTO T-99) so as to obtain its maximum dry density and the corresponding optimum moisture content. the results of the test are shown on table 4.3. The optimum moisture content was used for CBR purposes; details of this are provided in table 4.3. The maximum dry density of the soil is logically a justification of the specific gravity of the soil sample, its organic nature and poor gradation. Specific gravity of organic soils is usually less than 2.00 according to ASTM D 854. Poor graded soils cannot be easily and fully compacted. The graphical presentations of the obtained results are presented in the appendix A2. Table 4.3 shows the maximum density and optimum moisture content.

Table 4.3: Table showing maximum density and optimum moisture content

Sample Location	OMC (%)	MDD (kg/m^2)	2.5 (mm)	5.0 (mm)
PIT 1	20.0	1.59	6.8	10.0
PIT 2	17.0	1.72	63.4	74.6
PIT 3	20.8	1.62	6.8	15.5
PIT 4	17.9	1.70	72.5	81.7
PIT 5	22.5	1.63	3.8	14.0

Table 4.3 shows values of compaction test and the C.B.R values to stabilize the soil as at 1.0m depth. The MDD of the soils varied falling within the range of 1.59kg/m^3 and OMC ranging from 17.0% and 22.5%.

4.3 California Bearing Ratio Test

This was performed to BS 1377(2010). The CBR with respect to the moisture content that corresponds to the maximum dry density in the compaction test was determined from the graph shown in appendix B5; to get the mechanical strength of the soil sample, the CBR curve was superimposed on the Proctor curve both using the same horizontal axis (moisture content axis). Table 4.4 shows the result of the California Bearing Ratio.

Table 4.4 Result of California Bearing Ratio

S/N	CBR Value	PIT 1 (%)	PIT 2 (%)	PIT 3 (%)	PIT 4 (%)	PIT 5 (%)
1	2.5	6.8	63.4	6.8	72.5	3.8
2	5.0	10.0	74.6	15.5	81.7	14.0

4.4 Direct Shear Strength

The direct shear test was conducted on the samples to determine the Cohension, C, and Angle of Internal Friction, Phi. It was observed that there are variations in the properties of the soils at different locations. Table 4.5 shows the direct shear test results.

Table 4.5: Results of Direct Shear Test

S/N	Location	PIT 1	PIT 2	PIT 3	PIT 4	PIT 5
1	Slope	0.499	0.675	0.608	0.789	0.681
2	Intercept	56.633	19.467	20.833	55.20	24.30
3	Phi ¢	5^0	6^0	5^0	10^0	7^0
4	Cohesion	50	26	20	50	25

From the shear strength parameters, C and ¢ which is the cohension and the angle of internal friction, ranges within 20 to 50 for "C" and 5^0 to 10^0 for ¢ and 0.499 to 0.789 as the range of slope while the intercept falls within 24.3 to 56.633.

4.5 The Specific Gravity

The specific gravity of the soil samples ranges from 2.35 to 2.50. PIT 1 has the highest specific gravity value. According to specification, a good lateritic material should have specific gravity ranging from 2.6 to 2.9. Since PITS 1, 2, 3 and 5 are outside the range it suggests that soil consists of mostly Kaolinite clay mineral or potassium feldspar, according to, Braja M. Das (2010). Table 4.6 describes the specific gravity of common minerals while Table 4.7 describes the average specific gravity of samples.

Table 4.6 Specific Gravity of Common Minerals, Braja M. Das (2010)

Mineral	Specific gravity, G_s
Quartz	2.65
Kaolinite	2.6
Illite	2.8
Montmorillonite	2.65–2.80
Halloysite	2.0–2.55
Potassium feldspar	2.57
Sodium and calcium feldspar	2.62–2.76
Chlorite	2.6–2.9
Biotite	2.8–3.2
Muscovite	2.76–3.1
Hornblende	3.0–3.47
Limonite	3.6–4.0
Olivine	3.27–3.7

Table 4.7: Average Specific Gravity of Samples

S/N	Location of Sample	Average Specific Gravity
1	PIT 1	2.50
2	PIT 2	2.39
3	PIT3	2.26
4	PIT 4	2.35
5	PIT 5	2.42

4.6 Natural Moisture Content Test

The result of natural moisture content is shown in Appendix A4, some of the results were fairly high consistency, the time of the test, indicating the soil potential for the water retention except for one sample having 10.2% as the lowest moisture content. Table 4.8 describes the average moisture content for the samples.

Table 4.8: Average Moisture Content for the samples

S/N	Sample Location	Average Moisture Content
1	School Main Gate	10.3%
2	Agric Faculty	14.7%
3	Engineering Faculty	17.4%
4	School Hostel	10.2%
5	Health Centre	13.8%

4.7 Particle Size Distribution

For the area studied soils, the result show that some of has high percentage finer than 0.0075 fraction, that is, > 35% while soil from PIT 4 has finer < 35%, the materials are constituents of clay, sandy, and silt in which the percentage passing number 200 sieve is higher than 35%.

Table 4.9 Results of Particle Size Distribution

S/N	Sieve size (mm)	PIT1 %passing	PIT2 %passing	PIT3 %passing	PIT4 %passing	PIT5 %passing
1	9.50	100	106	100	100	100
2	4.75	97.8	96.4	99.8	98.5	99.4
3	2.36	88.7	89.4	98.6	89.8	96.9
4	1.18	75.3	81.3	95.4	78.9	92.3
5	600	67.9	72.8	89.4	65.7	84.5
6	0.30	59.4	61.9	80.7	37.6	74.5
7	0.15	51.1	53.6	74.7	25.1	65.4
8	0.0075	46.0	49.7	69.7	20.1	59.8

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION

The following conclusions were deduced from the summary of laboratory tests results. Results of the soil classification tests conducted on the samples shows that the soils tested classifies soils from pit 1 (School main gate) as a clayey soil material with a group classification of **A-7-6** according to AASHTO soil classification system because more than 35% passed through the sieve 200 with good percentage of sand and finer fractions. Equally, its plasticity index is greater than its liquid limit (PI > LL-30) and are fair to poor in general subgrade rating while samples from pits 2, 3, and 5 (Agric faculty, Engineering faculty, and Health centre) can generally be classified as silty-clay soil material with a group classification of **A-7-5** according to AASHTO soil classification system as in table 2.3 because more than 35% passed through the sieve 200 with good percentage of sand and finer fractions. Equally, its plasticity index is less than or equal to its liquid limit (PI < LL-30) and are fair to poor in subgrade rating and finally, samples from Pit 4 (School hostel) can be classified as a silty or clayey gravel material with a group classification of **A-2-6** because less than 35% passed through the sieve 200 and has its liquid limit less than 40%.

Using the USCS classification system, samples from Pits 1, 2, 3 and 5 can be classified as a CH soil because of its liquid limit greater than 50% and soil from Pit 4 classified as a CL soil because of its liquid limit less than 50%, as included in figure 2.5. All the soils in the study location have low potential of water retention with their natural moisture content not exceeding 22% and most of the sample soils are of clayey materials because greater than 35% of the soil passed through the 0.0075mm sieve (USCS Classification).

For the compaction test using the maximum dry density and optimum moisture content values, it shows that all the soil samples of the studied area are not suitable as subgrade materials for construction and may require soil stabilization for satisfactory performance.

5.2 RECOMMENDATIONS

These valuable data obtained from the geotechnical analysis can be useful for civil engineers in the design and construction of roads in the university and environs for maximum durability and efficiency. Based on the findings and confirmations from the geotechnical analysis of the soil samples, the following actions are recommended;

1. Soil stabilization or treatment of the soil if a construction need arises in the future.
2. Classification of other areas on the campus so as to have a concrete characterization of the community at large.
3. More engineering tests should be carried out before embarking on a construction in future.

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APPENDICES

Appendix A1: Sieve Analysis Result

Table A1.1: Sieve Analysis Result of PIT 1

Sample A			
sieve size	wt. ret.	% ret.	% passing
9.50	0	0	100
4.75	10.90	2.2	97.8
2.36	45.70	9.1	88.7
1.18	66.90	13.4	75.3
600	37.0	7.4	67.9
300	42.3	8.5	59.4
150	41.4	8.3	51.1
75	25.5	5.1	46.0

Table A1.2: Sieve Analysis Result of PIT 2

Sample B			
sieve size	wt. ret.	% ret.	% passing
9.50	0	0	100
4.75	18.1	3.6	96.4
2.36	35.2	7.0	89.4
1.18	40.6	8.1	81.3
600	42.7	8.5	72.8
300	54.6	10.9	61.9
150	41.7	8.3	53.6
75	19.3	3.9	49.7

Table A1.3: Sieve Analysis Result of PIT 3

Sample C			
sieve size	wt. ret.	% ret.	% passing
9.50	0	0	100
4.75	3.0	0.6	99.4
2.36	12.7	2.5	96.9
1.18	23.0	4.6	92.3
600	36.8	7.4	84.5
300	50.1	10.0	74.5
150	47.6	9.5	65.4
75	28.0	5.6	59.8

Table A1.4: Sieve Analysis Result of PIT 4

Sample D			
sieve size	wt. ret.	% ret.	% passing
9.50	0	0	100
4.75	7.5	1.5	98.5
2.36	43.5	8.7	89.8
1.18	54.7	10.9	78.9
600	65.9	13.2	65.7
300	140.7	28.1	37.6
150	62.3	12.5	25.1
75	25.1	5.0	20.1

Table A1.5: Sieve Analysis Result of PIT 5

Sample E			
sieve size	wt. ret.	% ret.	% passing
9.50	0	0	100
4.75	0.9	0.2	98.5
2.36	6.1	1.2	89.8
1.18	15.8	3.2	78.9
600	29.9	6.0	65.7
300	43.6	8.7	37.6
150	30.0	6.0	25.1
75	25.1	5.0	20.1

Appendix A2: West African Compaction Result and Calculation

Appendix A2: Compaction Analysis Result

Table A2.1: Compaction Result of PIT 1

Sample A				
Trial No.	1	2	3	4
Weight of mould + soil (g)	4700	4900	5100	4950
weight of empty mould (g)	3200	3200	3200	3200
weight of wet soil	1500	1700	1900	1750
wet density of soil (Kg/m^3)	1.5	1.7	1.9	1.75
container identification no.	A1	B1	C1	D1
weight of container (g)	19.8	26.5	16.7	27.6
weight of wet soil + container (g)	89.8	82.9	70.6	80.5
weight of dry soil + container (g)	83.9	75.7	61.7	70.4
weight of water (g)	5.9	7.2	8.9	10.1
weight of dry soil (g)	64.1	49.2	45.0	42.8
moisture content (%)	9.2	14.6	19.8	23.6

dry density (kN/m³)	1.37	1.48	1.59	1.42

Table A2.2: Compaction Result of PIT 2

Sample B				
Trial No.	1	2	3	4
Weight of mould + soil (g)	5100	5250	5300	5200
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1950	2100	2150	2050
wet density of soil (Kg/m³)	1.95	2.10	2.15	2.05
container identification no.	A2	B2	C2	D2
weight of container (g)	26.60	10.70	26.70	26.90
weight of wet soil + container (g)	85.40	72.40	81.40	88.50
weight of dry soil + container (g)	73.40	65.70	74.10	79.30
weight of water (g)	5.10	6.70	7.30	9.20
weight of dry soil (g)	51.70	55.00	47.40	52.40
moisture content (%)	9.90	12.20	15.40	17.80
dry density (kN/m³)	1.77	1.83	1.86	1.74

Table A2.3: Compaction Result of PIT 3

Sample C				
Trial No.	1	2	3	4
Weight of mould + soil (g)	5100	5300	5350	5250
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1950	2150	2200	2100
wet density of soil (Kg/m³)	1.95	2.15	2.20	2.10
container identification no.	A3	B3	C3	D3

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weight of container (g)	20.10	26.70	26.60	18.00
weight of wet soil + container (g)	83.00	83.50	81.60	76.20
weight of dry soil + container (g)	79.20	78.60	76.10	68.30
weight of water (g)	3.80	4.90	5.50	7.90
weight of dry soil (g)	59.10	51.90	49.50	50.30
moisture content (%)	6.40	9.40	11.10	15.70
dry density (kN/m³)	1.83	1.97	1.98	1.82

Table A2.4: Compaction Result of PIT 4

Sample D				
Trial No.	1	2	3	4
Weight of mould + soil (g)	4950	5150	5300	5200
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1800	2000	2150	2050
wet density of soil (Kg/m³)	1.80	2.00	2.15	2.05
container identification no.	A4	B4	C4	D4
weight of container (g)	17.80	26.60	18.50	18.30
weight of wet soil + container (g)	83.40	82.10	87.60	85.90
weight of dry soil + container (g)	77.80	76.30	79.30	75.10
weight of water (g)	5.00	5.80	8.30	10.80
weight of dry soil (g)	60.00	49.70	60.80	56.80
moisture content (%)	8.30	11.70	13.70	19.10
dry density (kN/m³)	1.66	1.79	1.89	1.72

Table A2.5: Compaction Result of PIT 5

Sample E				
Trial No.	1	2	3	4

Weight of mould + soil (g)	4950	5150	5300	5200
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1800	2000	2150	2050
wet density of soil (Kg/m³)	1.80	2.00	2.15	2.05
container identification no.	A5	B5	C5	D5
weight of container (g)	17.80	26.60	18.50	18.30
weight of wet soil + container (g)	83.40	82.10	87.60	85.90
weight of dry soil + container (g)	77.80	76.30	79.30	75.10
weight of water (g)	5.00	5.80	8.30	10.80
weight of dry soil (g)	60.00	49.70	60.80	56.80
moisture content (%)	8.30	11.70	13.70	19.10
dry density (kN/m³)	1.66	1.79	1.89	1.72

Appendix A3: Specific Gravity Result and Calculation

Table A3.1: Specific Gravity Result of PIT 1

Sample A		
Trial no.	1	2
weight of empty density bottle (g)	23.8	25.8
weight of density bottle + dry soil (g)	49.6	53.7
weight of density bottle + soil + water (g)	93.9	95.6
weight of density bottle + water (g)	78.3	79.0
Specific Gravity, S.G	2.53	2.47

W1 = weight of empty density bottle

W2 = weight of density bottle + dry soil

W3 = weight of density bottle + soil + water

W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.50$$

Table A3.2: Specific Gravity Result of PIT 2

Sample B		
Trial no.	1	2
weight of empty density bottle (g)	25.8	25.8
weight of density bottle + dry soil (g)	51.7	53.5
weight of density bottle + soil + water (g)	93.3	94.0
weight of density bottle + water (g)	78.1	78.0
Specific Gravity, S.G	2.42	2.37

W1 = weight of empty density bottle

W2 = weight of density bottle + dry soil

W3 = weight of density bottle + soil + water

W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.39$$

Table A3.3: Specific Gravity Result of PIT 3

Sample C		
Trial no.	1	2
weight of empty density bottle (g)	26.4	26.4
weight of density bottle + dry soil (g)	50.9	53.4
weight of density bottle + soil + water (g)	91.4	94.8
weight of density bottle + water (g)	77.6	79.9
Specific Gravity, S.G	2.29	2.23

W1 = weight of empty density bottle

W2 = weight of density bottle + dry soil

W3 = weight of density bottle + soil + water

W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.26$$

Table A3.4: Specific Gravity Result of PIT 4

Sample D		
Trial no.	1	2
weight of empty density bottle (g)	23.8	26.9
weight of density bottle + dry soil (g)	49.9	49.4
weight of density bottle + soil + water (g)	93.1	92.4
weight of density bottle + water (g)	78.2	79.4

Specific Gravity, S.G	2.33	2.37

W1 = weight of empty density bottle

W2 = weight of density bottle + dry soil

W3 = weight of density bottle + soil + water

W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.35$$

Table A3.5: Specific Gravity Result of PIT 5

Sample E		
Trial no.	1	2
weight of empty density bottle (g)	23.8	26.9
weight of density bottle + dry soil (g)	49.9	49.4
weight of density bottle + soil + water (g)	93.1	92.4
weight of density bottle + water (g)	78.2	79.4
Specific Gravity, S.G	2.33	2.37

W1 = weight of empty density bottle

W2 = weight of density bottle + dry soil

W3 = weight of density bottle + soil + water

W4 = weight of density bottle + water

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.42$$

Appendix A4: Natural Moisture Content Result and Calculation

Table A4.1: Natural Moisture Content Result of PIT 1

Sample A		
Trial no.	1	2
weight of container (g)	26.6	19.7
weight of container + soil+ water (g)	88.8	79.3
weight of container + dry soil (g)	81.7	72.3
Moisture content (%)	12.9	13.3

A = weight of container

B = weight of container + water + soil

C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 13.1$$

Table A4.2: Natural Moisture Content Result of PIT 2

Sample B		
Trial no.	1	2
weight of container (g)	10.0	9.9
weight of container + soil+ water (g)	70.2	76.7
weight of container + dry soil (g)	60.7	67.9
Moisture content (%)	18.7	15.2

A = weight of container

B = weight of container + water + soil

C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 17.0$$

Table A4.3: Natural Moisture Content Result of PIT 3

Sample C		
Trial no.	1	2
weight of container (g)	26.7	26.9
weight of container + soil+ water (g)	95.3	92.8
weight of container + dry soil (g)	78.2	85.4
Moisture content (%)	33.2	12.6

A = weight of container

B = weight of container + water + soil

C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 22.9$$

Table A4.3: Natural Moisture Content Result of PIT 4

Sample D		
Trial no.	1	2
weight of container (g)	20.1	26.6
weight of container + soil+ water (g)	63.9	93.4
weight of container + dry soil (g)	57.9	84.7
Moisture content, M.C	15.9	15.0

A = weight of container

B = weight of container + water + soil

C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 15.4$$

Table A4.5: Natural Moisture Content Result of PIT 5

Sample E		
Trial no.	1	2
weight of container (g)	20.1	26.6
weight of container + soil+ water (g)	63.9	93.4
weight of container + dry soil (g)	57.9	84.7
Moisture content, M.C	15.9	15.0

A = weight of container

B = weight of container + water + soil

C = weight of container + dry soil

M.C = B - C x 100 = 18.4

Appendix A5: Direct Shear Test Result

Table A5.1: Direct Shear Test Result of PIT 1

Test No	Normal stress (kg/cm^2)	Normal Force (KN)	Normal Strength (KN/m^2)	Max DR Div	Shear Force (KN)	Shear Stress (KN/m^2)
1	5	0.491	1.36	47.1	0.283	78.6
2	10	0.980	2.72	67.5	0.405	112.5
3	15	0.147	4.80	77.1	0.463	128.5

Table A5.2: Direct Shear Test Result of PIT 2

Test No	Normal stress (kg/cm^2)	Normal Force (KN)	Normal Strength (KN/m^2)	Max DR Div	Shear Force (KN)	Shear Stress (KN/m^2)
1	5	0.491	1.36	32.3	0.195	54.2
2	10	0.980	2.72	48.0	0.288	80.0
3	15	0.147	4.80	73.0	0.438	121.7

Table A5.3: Direct Shear Test Result of PIT 3

Test No	Normal stress (kg/cm^2)	Normal Force (KN)	Normal Strength (KN/m^2)	Max DR Div	Shear Force (KN)	Shear Stress (KN/m^2)
1	5	0.491	1.36	30.7	0.184	51.1
2	10	0.980	2.72	49.1	0.295	81.9
3	15	0.147	4.80	67.1	0.403	111.9

Table A5.4: Direct Shear Test Result of PIT 4

Test No	Normal stress (kg/cm^2)	Normal Force (KN)	Normal Strength (KN/m^2)	Max DR Div	Shear Force (KN)	Shear Stress (KN/m^2)
1	5	0.491	1.36	55.0	0.330	91.7
2	10	0.980	2.72	84.0	0.504	140.0
3	15	0.147	4.80	102.4	0.614	170.6

Table A5.5: Direct Shear Test Result of PIT 5

Test No	Normal stress (kg/cm^2)	Normal Force (KN)	Normal Strength (KN/m^2)	Max DR Div	Shear Force (KN)	Shear Stress (KN/m^2)
1	5	0.491	1.36	35.2	0.211	58.6
2	10	0.980	2.72	49.0	0.294	73.3
3	15	0.147	4.80	76.0	0.456	126.7

Appendix A6: California Bearing Ratio Test Result

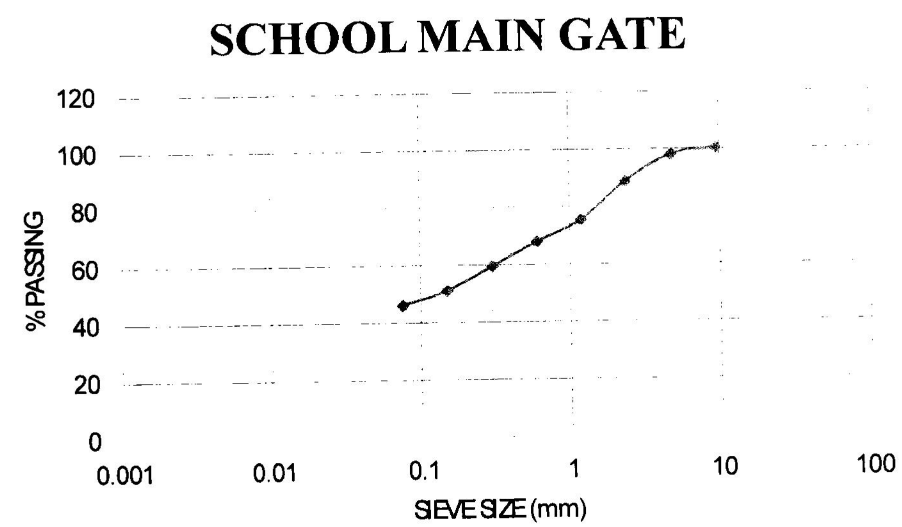
Table A6.1: California Bearing Ratio Test Result of PITS 1-4

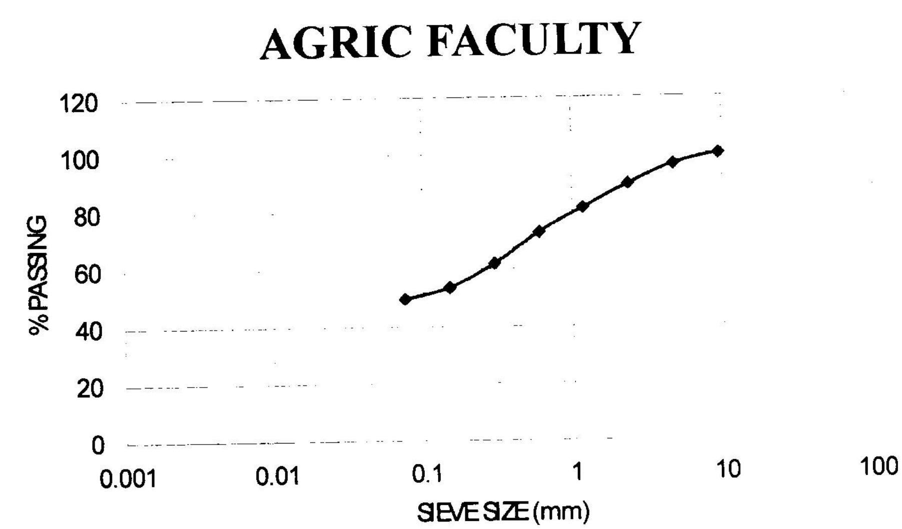
CARLIFORNIA BEARING RATIO								
Plunger Penetration (mm)	Sample A		Sample B		Sample C		Sample D	
	Dial Gauge Reading	Load (kN)	Dial Gauge Reading	Load (kN)	Dial Gauge Reading	Load (kN)	Dial Gauge Reading	Load (kN)
0.5	109	2.73	76	1.90	201	5.00	201	5.00
1.0	132	3.30	91	2.28	289	7.20	289	7.20
1.5	176	4.40	127	3.18	330	8.30	330	8.30
2.0	205	5.13	168	4.20	378	9.50	378	9.50
2.5	244	6.10	203	5.08	426	10.70	426	10.70
3.0	310	7.75	251	6.28	478	12.00	478	12.00
3.5	348	8.70	302	7.55	538	13.50	538	13.50
4.0	483	12.08	465	11.63	587	14.70	587	14.70
4.5	595	14.88	536	13.40	628	15.70	628	15.70
5.0	685	17.13	638	15.95	675	16.90	675	16.90
5.5	725	18.13	662	16.55	710	17.80	710	17.80
6.0	769	19.23	695	17.38	762	19.10	762	19.10
6.5	802	20.05	719	17.98	801	20.00	801	20.00
7.0	838	14.90	741	18.53	820	20.50	820	20.50
7.5	876	21.90	768	19.20	838	21.00	838	21.00

APPENDIX B: GEOTECHNICAL TESTS RESULT GRAPHS

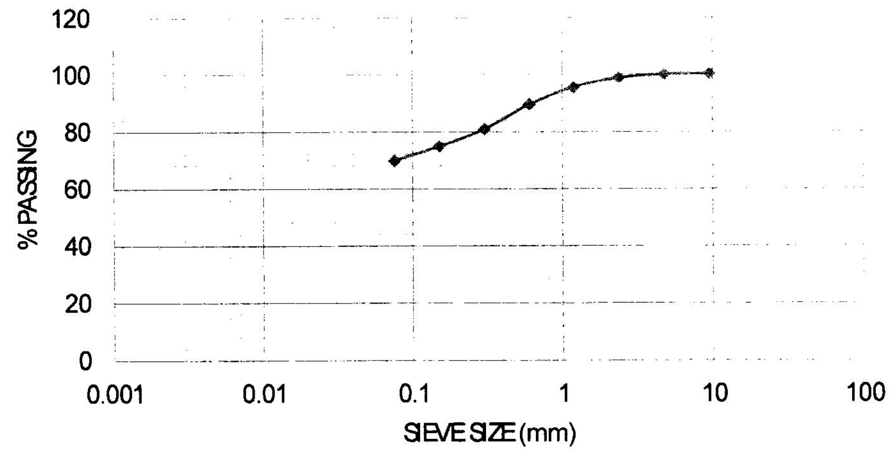
Appendix B1: Sieve Analysis Graphs

GRAPH OF SIEVE SIZE AGAINST %PASSING FOR ALL TRIAL PITS

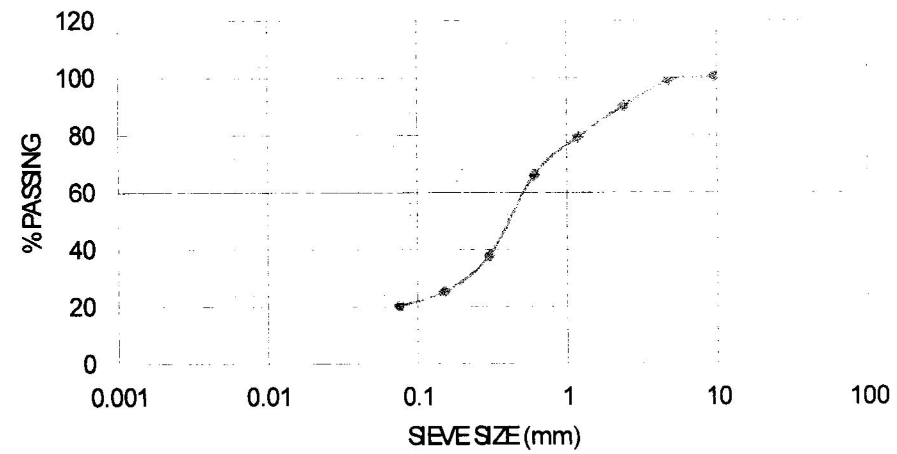




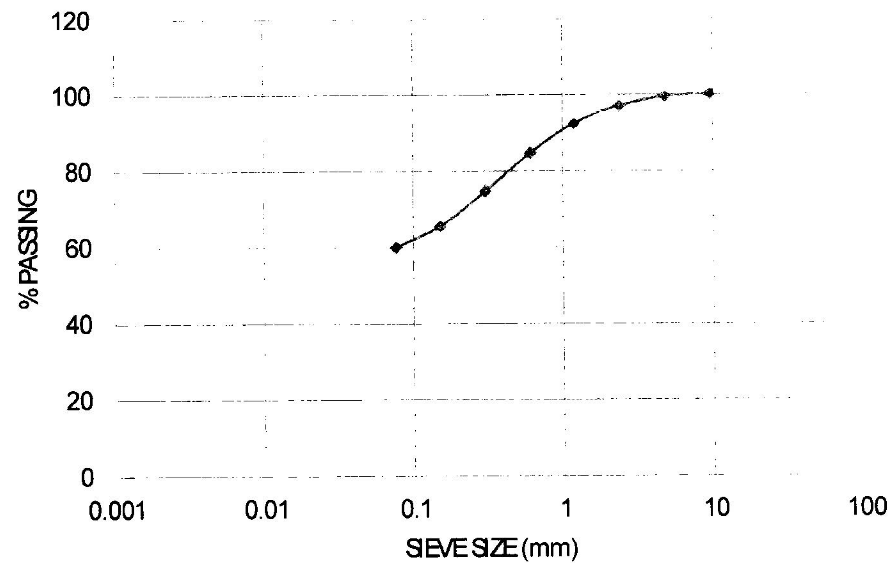
ENGINEERING FACULTY



SCHOOL HOSTEL



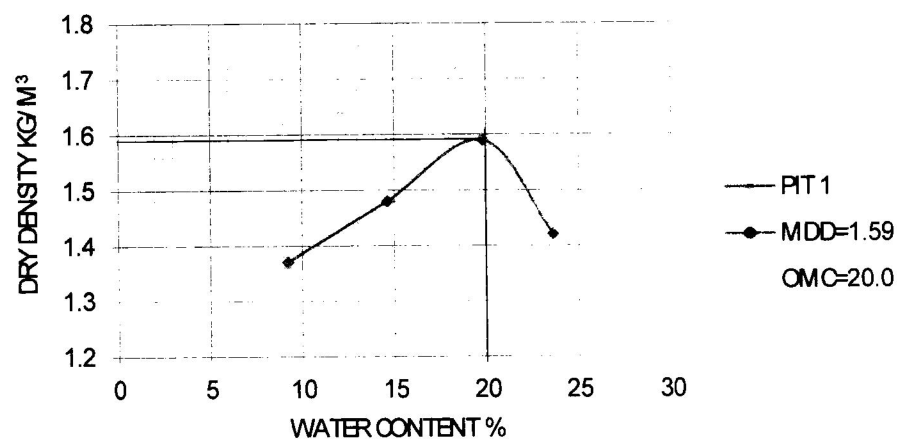
HEALTH CENTRE



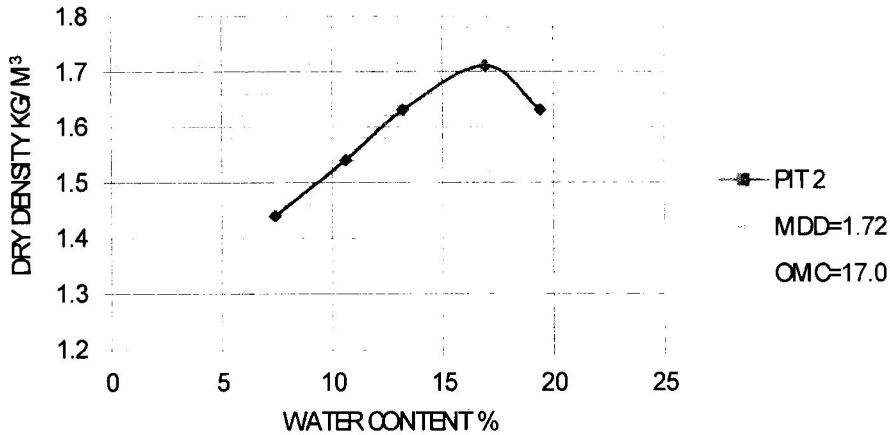
Appendix B2: West African Compaction Graphs

GRAPHS OF WATER CONTENT AGAINST DRY DENSITY FOR ALL TRIAL PITS

SCHOOL MAIN GATE



AGRIC FACULTY



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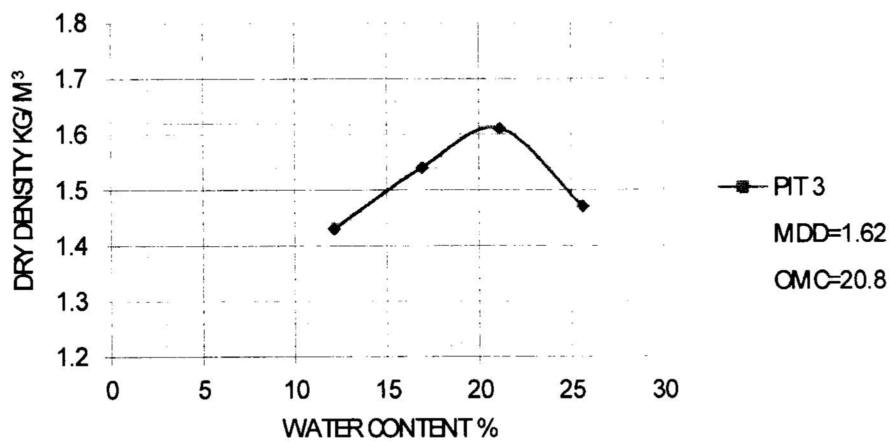

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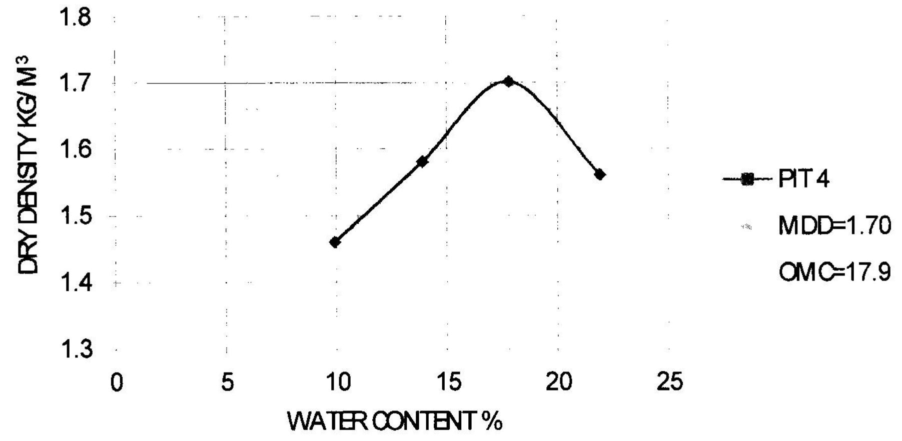

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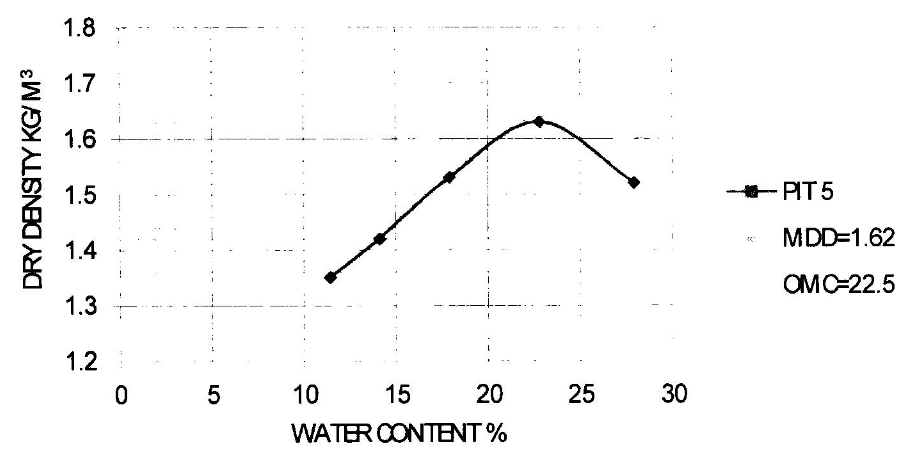
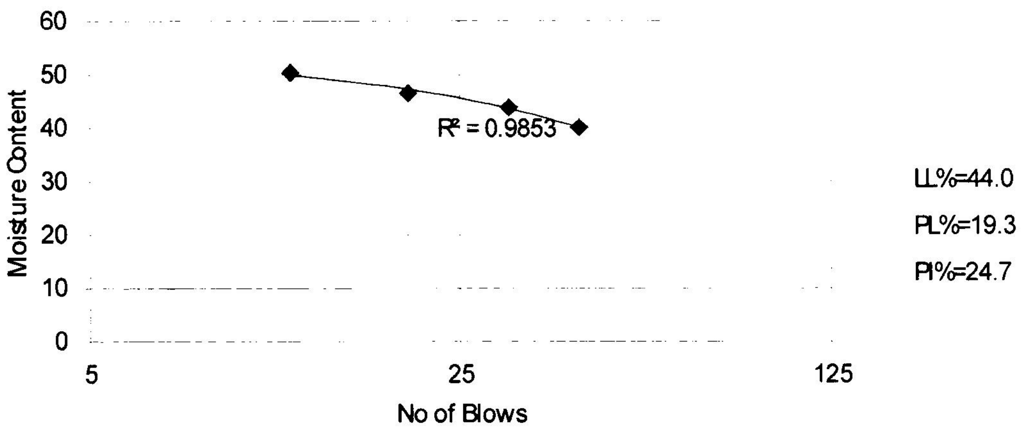


Appendix B3: Atterberg Limit Graphs

GRAPHS OF LIQUID LIMIT FOR ALL TRIAL PITS

SCHOOL MAIN GATE



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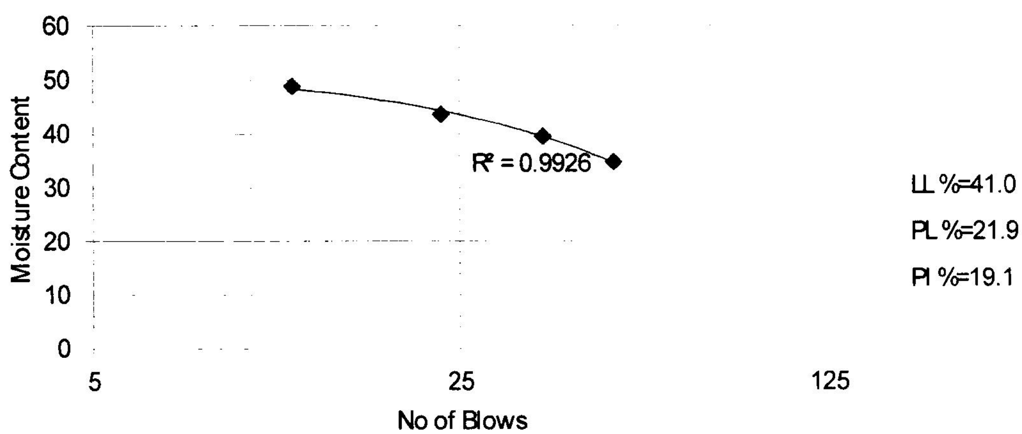

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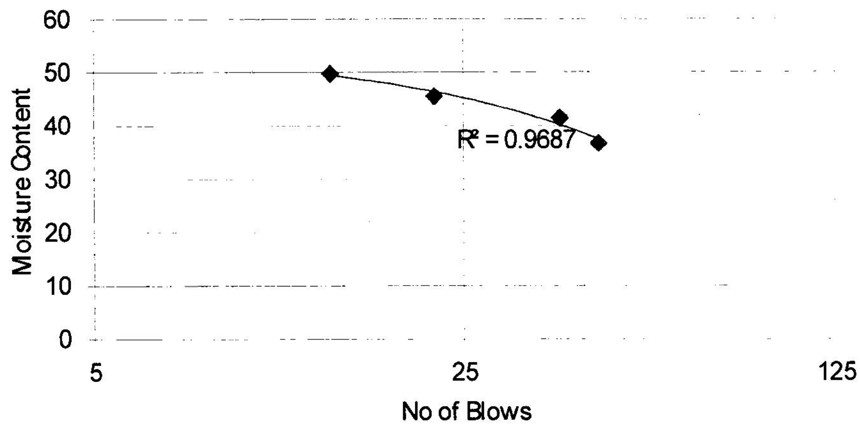

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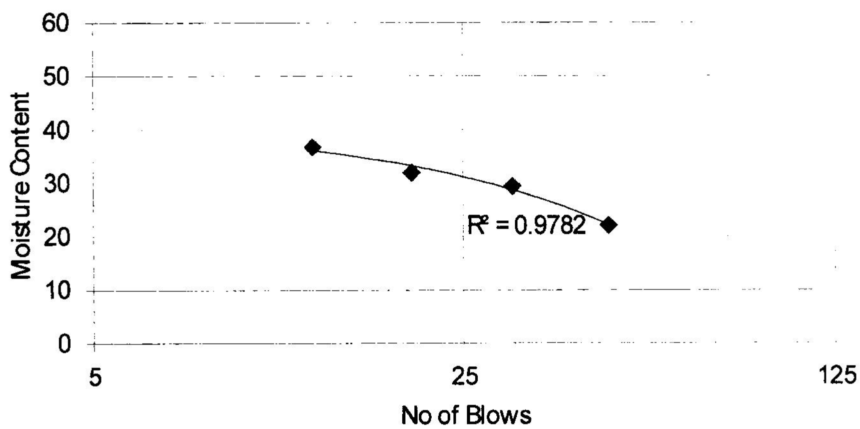
SCHOOL HOSTEL



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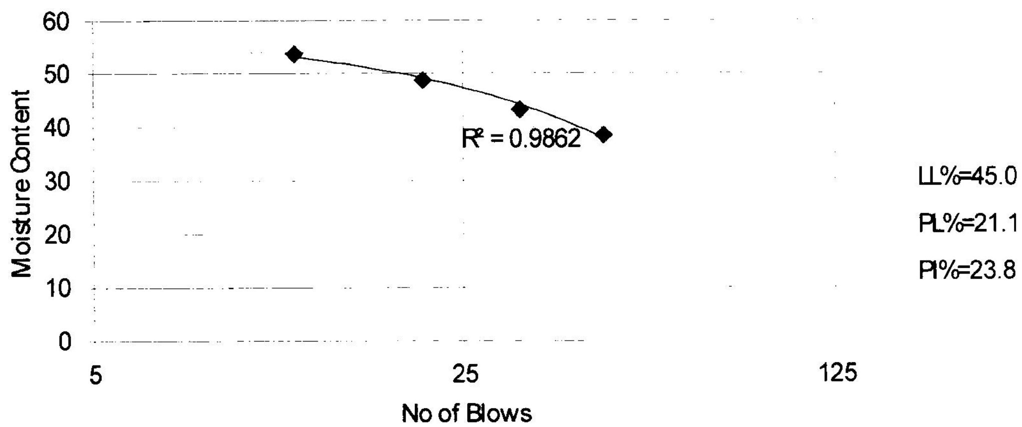
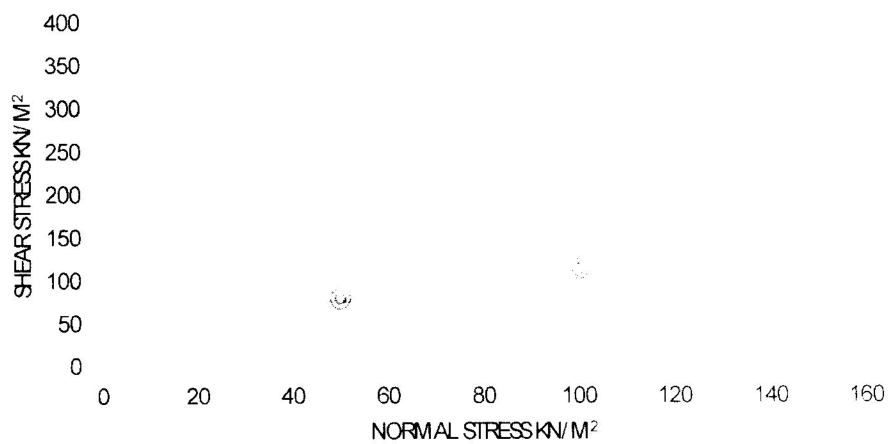
HEALTH CENTRE



Appendix B4: Direct Shear Test Graphs

GRAPHS OF NORMAL STRESS AGAINST SHEAR STRENGTH FOR ALL PITS

SCHOOL MAIN GATE



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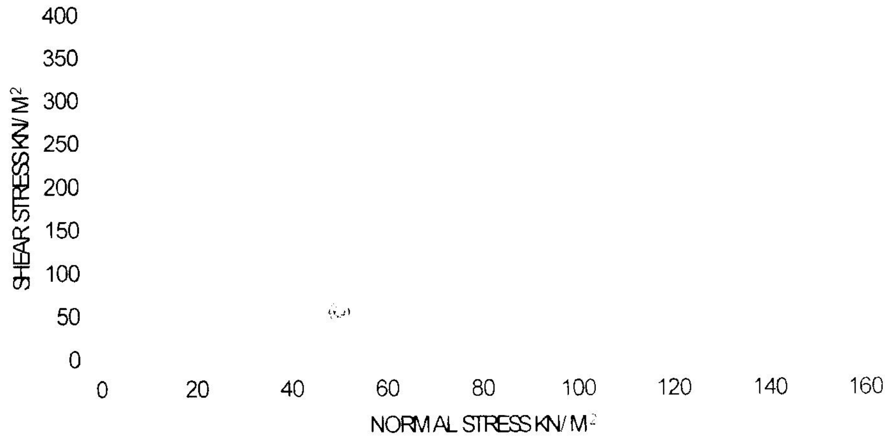

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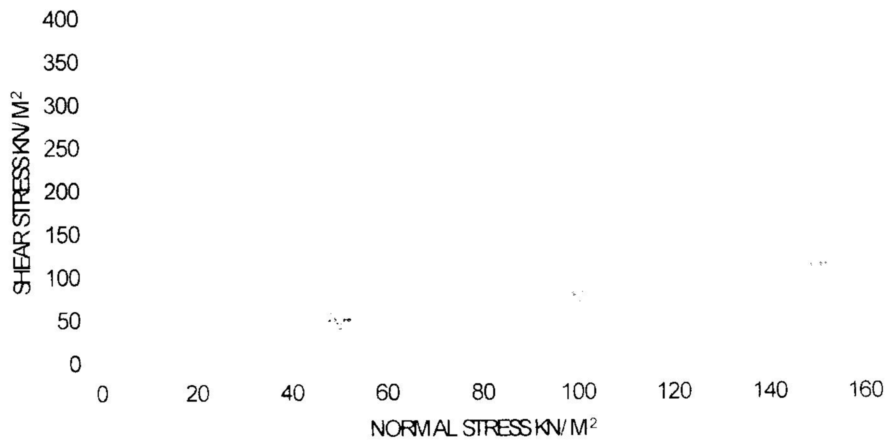
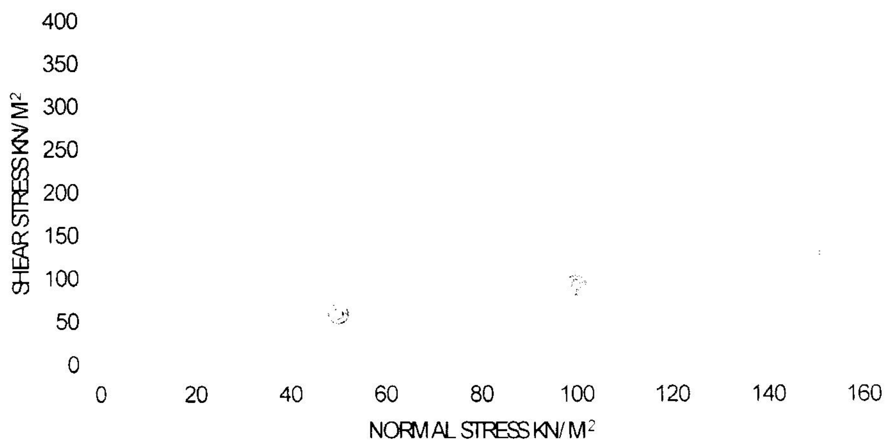


HEALTH CENTRE



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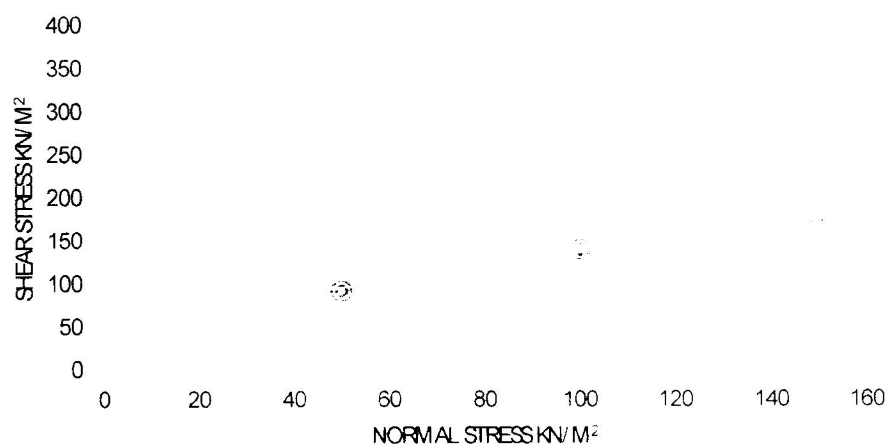
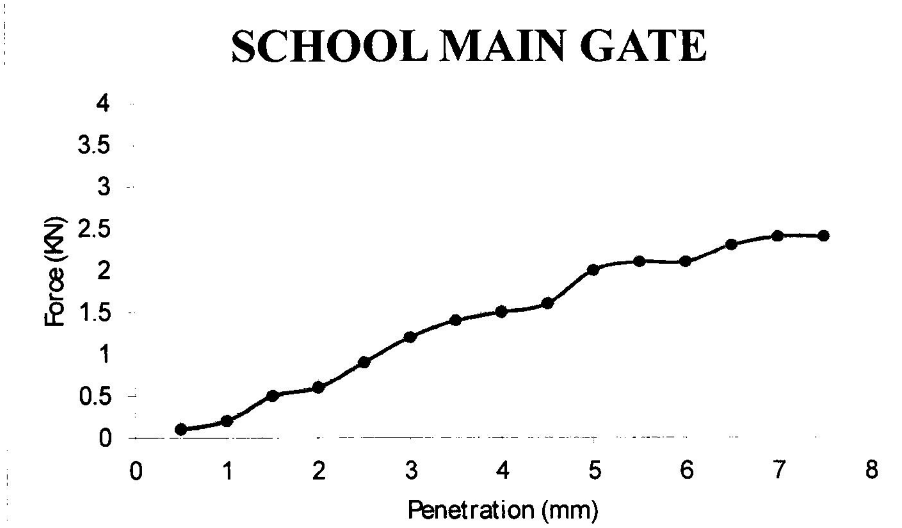
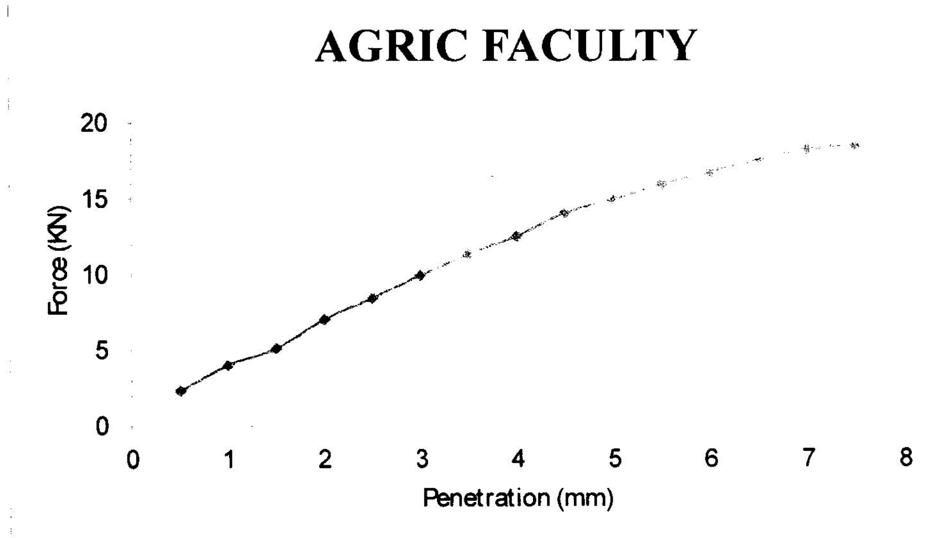


Appendix B5: California Bearing Ratio Graphs

GRAPHS OF PENETRATION AGAINST FORCE FOR ALL TRIAL PITS





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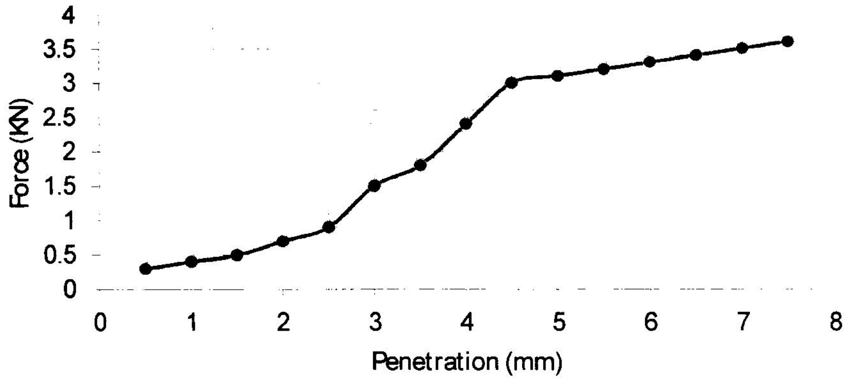

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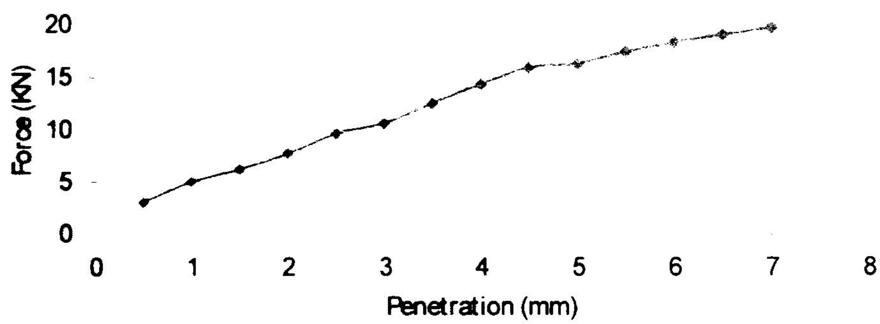

HEALTH CENTRE