STRENGTH CHARACTERISTICS OF SOILS IN IKOLE CAMPUS OF FEDERAL UNIVERSITY OYE EKITI, EKITI STATE, NIGERIA

By

JOSU, Gigonu Michael

CVE/11/0369

A project report submitted to the Department of Civil Engineering, Federal University Oye Ekiti in partial fulfillment of the requirement for the award of the B. Eng. (Hons) in Civil Engineering.

Department of Civil Engineering

Faculty of Engineering

2016

ABSTRACT

The development of soil and rock properties for geotechnical design purposes begins with developing/defining the geologic strata present at the site in question. Therefore, the focus of geotechnical design property assessment and final selection shall be on the individual geologic strata identified at the project site.

This research work was been carried out to determine the shear strength characteristics of soils in Ikole Campus of Federal University Oye Ekiti, Ekiti State. Classification, compaction and strength characteristics were examined for the soil samples in the study area. The study area falls within coordinates 7.7983°N, 5.5145°E and covers a land area of 538.550 hectares in Ikole Ekiti, Ekiti state.

Undisturbed and disturbed samples were obtained at depths of 1.5m and 3.0m from five different trial pits, (TP_1, TP_2, TP_3, TP_4 and TP_5). The coordinates of the trial pits across the campus are TP_1 (866971.98N, 610838.61E), TP_2 (867676.65N, 611093.21E), TP_3 (867224.80N, 610566.90E), TP_4 (867759.99N, 610610.02E) and TP_5 (867382.93N, 610810.38E). The tests carried out in this research include natural moisture content, particle size distribution, specific gravity, Atterberg limit, compaction, California bearing ratio and quick triaxial test.

The results show that the study location have low potential of water retention with their natural moisture content not exceeding 25% and all the soils have clay content from the specific gravity result. TP1, TP2, TP4 and TP5 can generally be classified as Silty-Clay soil material with fair to poor in general subgrade rating while only TP3 is generally classified as a granular soil and is rated as excellent to good in general subgrade rating. The Atterberg test results shows that the soil samples from TP1, TP2, TP4 and TP5 can be grouped as A-7-5 or A-7-6 class i.e. clayey soil while TP3 is classified as A-2-6 i.e. lateritic soil according to AASHTO classification system (1978). The triaxial results show that the study location has a value of cohesion ranging between 9 – 190 kN/m^2. These valuable data obtained from this geotechnical analysis can be useful for civil engineers in the design and construction of structures and roads in Ikole campus and environs for maximum durability and efficiency.

Keyword: Soil, Shear strength and Ikole Ekiti.

CERTIFICATION

I hereby certify that this research work was carried out by **JOSU GIGONU MICHAEL** with matriculation number **CVE/11/0369** of the department of **CIVIL ENGINEERING** under the faculty of **ENGINEERING, FEDERAL UNIVERSITY OF OYE EKITI, EKITI STATE** as part of the partial fulfillment requirements of the award of Bachelor of Engineering, (B.Eng) in Civil Engineering. Therefore, the information provided in this report is honest and has been meticulously checked by me.

Josu G. M. ....

Student Sign & Date

Engr. Bolarinwa A. A. ....

Supervisor Sign & Date

Prof. Adeyeri J. B. ....

Head of Department (H.O.D) Sign & Date

DEDICATION

"For He is worthy of our praise" I dedicate this research project to God Almighty; the all knowing, the all sufficient and the giver of wisdom, knowledge and understanding who spared my life from birth till date, may His only name be highly exulted.

ACKNOWLEDGEMENT

My gratitude goes to Almighty God who gave me the spirit of wisdom and understanding in my academics. I will also like to show my appreciation to the Head of Department, Prof. J. B. Adeyeri. I will never forget to thank every of my lecturers who has impacted great knowledge in me in one way or the other, I say thank you. I will be an ingrate if heart of gratitude is not shown to my Supervisor, Engr. A. Bolarinwa for his all-round support and encouragement, God bless you Sir. Finally, I will like to give a big thank you to my parent Mr. N.J Josu and Mrs E. B Josu and to my friends, course mates, families and loved ones for their relentless support financially, morally, academically, socially and in all aspect of life.

Table of Content

List of Figures

List of Tables

CHAPTER ONE

1.0 INTRODUCTION

Naturally occurring deposits of the earth's crust are classified by engineers into "soils" and "rock" with an arbitrary division based on strength, related physical properties and use, Alam (2008). Soil, in an engineering sense, is the relatively loose agglomerate of mineral and organic materials and sediments found above the bedrock, Robert and William (1981).

Structures of all types (buildings, bridges, highway, etc.) rest directly on, in, or against soil; hence, proper analysis of soils and design of foundations are necessary to ensure that these structures remain safe and free of undue settling and collapse; Cheng and Jack (2009). It has been observed that problem-soils poses a serious threat to civil engineering projects which results in defect or collapse of infrastructures such as roads, buildings, dams among others.

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, Holtz and Kovacs (1981).

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

1.1 Soil Description and Classification

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, Craig (1992). The principal material characteristics are particle size distribution (or grading) and plasticity, from which the soil name can be deduced Bowels (1996). 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)

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 Ramamurthy and Sitharam (2010). 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 Irving *et al.* (1980). However, classification is particularly useful if the soil in question is to be used as a construction material, for example in an embankment. Engineers can also draw on past experience of the behaviour of soils of similar classification. (Townsend, 1973)

1.1.1 Details of Soil Description

A detailed guide to soil description is given in BS 5930. According to this standard the basic soil types are boulders, cobbles, gravel, sand, silt and clay, defined in terms of the particle size ranges, added to these are organic clay, silt or sand, and peat. A soil is of basic type sand or gravel (these being termed coarse soils) if, after the removal of any cobbles or boulders, over 65% of the material is of sand and gravel sizes. A soil is of basic type silt or clay (termed fine soils) if, after the removal of any cobbles or boulders, over 35% of the material is of silt and clay sizes. However, these percentages should be considered as approximate guidelines, not forming a rigid boundary. Sand and gravel may each be subdivided into coarse, medium and fine fractions. The state of sand and gravel can be described as well graded, poorly graded, uniform or gap graded. In the case of gravels, particle shape (angular, sub-angular, sub-rounded, rounded, flat, elongated) and surface texture (rough, smooth, polished) can be described if necessary. Particle composition can also be stated. Gravel particles are usually rock fragments (e.g. sandstone, schist). Sand particles usually consist of individual mineral grains (e.g. quartz, feldspar). Fine soils should be described as either silt or clay: terms such as silty clay should not be used. Composite types of coarse soil are named as shown in Table 2.1, the predominant component being written in capital letters. Fine soils containing 35–65% coarse material are described as sandy and/or gravelly SILT (or CLAY). Deposits containing over 50% of boulders and cobbles are referred to as very coarse and normally can be described only in excavations and exposures. Mixes of very coarse material with finer soils can be described by combining the descriptions of the two components, e.g. COBBLES with some FINER MATERIAL (sand); gravelly SAND with occasional BOULDERS.

Table 1.1 Composite types of coarse soil

Slightly sandy GRAVEL	Up to 5% sand
Sandy GRAVEL	5–20% sand
Very sandy GRAVEL	Over 20% sand
SAND and GRAVEL	About equal proportions
Very gravelly SAND	Over 20% gravel
Gravelly SAND	5–20% gravel
Slightly gravelly SAND	Up to 5% gravel
Slightly silty SAND (and/or GRAVEL)	Up to 5% silt
Silty SAND (and/or GRAVEL)	5–20% silt
Very silty SAND (and/or GRAVEL)	Over 20% silt
Slightly clayey SAND (and/or GRAVEL)	Up to 5% clay
Clayey SAND (and/or GRAVEL)	5–20% clay
Very clayey SAND (and/or GRAVEL)	Over 20% clay

Source: BS 5930

Notes: Terms such as 'Slightly clayey gravelly SAND' (having less than 5% clay and gravel) and 'Silty sandy GRAVEL' (having 5–20% silt and sand) can be used, based on the above proportions of secondary constituents.

1.1.2 Strength Properties of Soils

Soils are weathered under conditions of high temperatures and humidity with well-defined alternating wet and dry seasons and continually leached by rainwater causing a tendency for deterioration of its strength characteristics, Oberg and Salfours (1997). Shear strength characteristics of these soils have been found to depend significantly on the parent materials and the degree of weathering (i.e. degree of decomposition, laterization and dessication) which is a function of the position of the sample in the soil profile and the compositional factors, Bjerrum (1974). The higher the degrees of laterization, the more favourable are the shear strength parameters, Bowels (1978). Furthermore, the structural elements in the soil are often a less stable coarse-grained aggregation of variable strength which may break down in performance, Kezdi and Rethati (1988) in addition to their varying silt and clay content which often render them moisture sensitive, Bowels (1996). The aforementioned properties give an indication of their engineering limitations that restricts its uses and of such sites to minor engineering projects.

1.2 Geology of the Study Area

Ekiti State is underlain entirely by crystalline Basement Complex rocks of the Gneiss – Schist Complex, the meta-sediments and meta-volcanic series and the Pan African granitoids (older granites) which are composed of gneisses, schists,quartzite migmatite, charnockite, diorites, granites, granodiorites and pegmatites all of which granites, granodiorites and pegmatites all of which are Precambrian in Malomo (2011.)

A group of granites called younger granites, which are made up of Granites, Granite porphyry, Syenites, Gabbro, Rhyolite and others are regarded as Jurassic in age and a similar trace of these types of rock are found around Ado-Ekiti, Ikole, Ikere, Aramoko, all in Ekiti state and are referred to as charnockite series, Oladapo (2013). The geology of

Ekiti State has been well researched by Kayode and Adelusi (2010), Omotoyinbo (1994), Shittu and Fasina (2004), Omotoyinbo and Olusoji (2008), Olusiji (1013), Bayowa *et al.*, (2007).

1.3 Study Location

The study area falls within coordinates 7.7983°N, 5.5145°E of and covers a land area of 538.550 hectares in Ikole Ekiti, Ekiti state. Stated in Table 1.1 below are the coordinates of the trial pits. Figure 1.1 shows the coordinates of the trial pits on the survey map of the study location.

Table 1.2: Trial Pits and Coordinates.

S/N	Trial pits	Coordinate in degree		Coordinate in metric(m)		Elevation (m)
		Northing	Easting	Northing	Easting	
1	TP 1	7.801562°	5.496712°	866971.98	610838.61	553
2	TP 2	7.808083°	5.499003°	867676.65	611093.21	539
3	TP 3	7.803837°	5.494267°	867224.80	610566.90	561
4	TP 4	7.808653°	5.494655°	867759.99	610610.02	551
5	TP 5	7.805260°	5.496458°	867382.93	610810.38	568

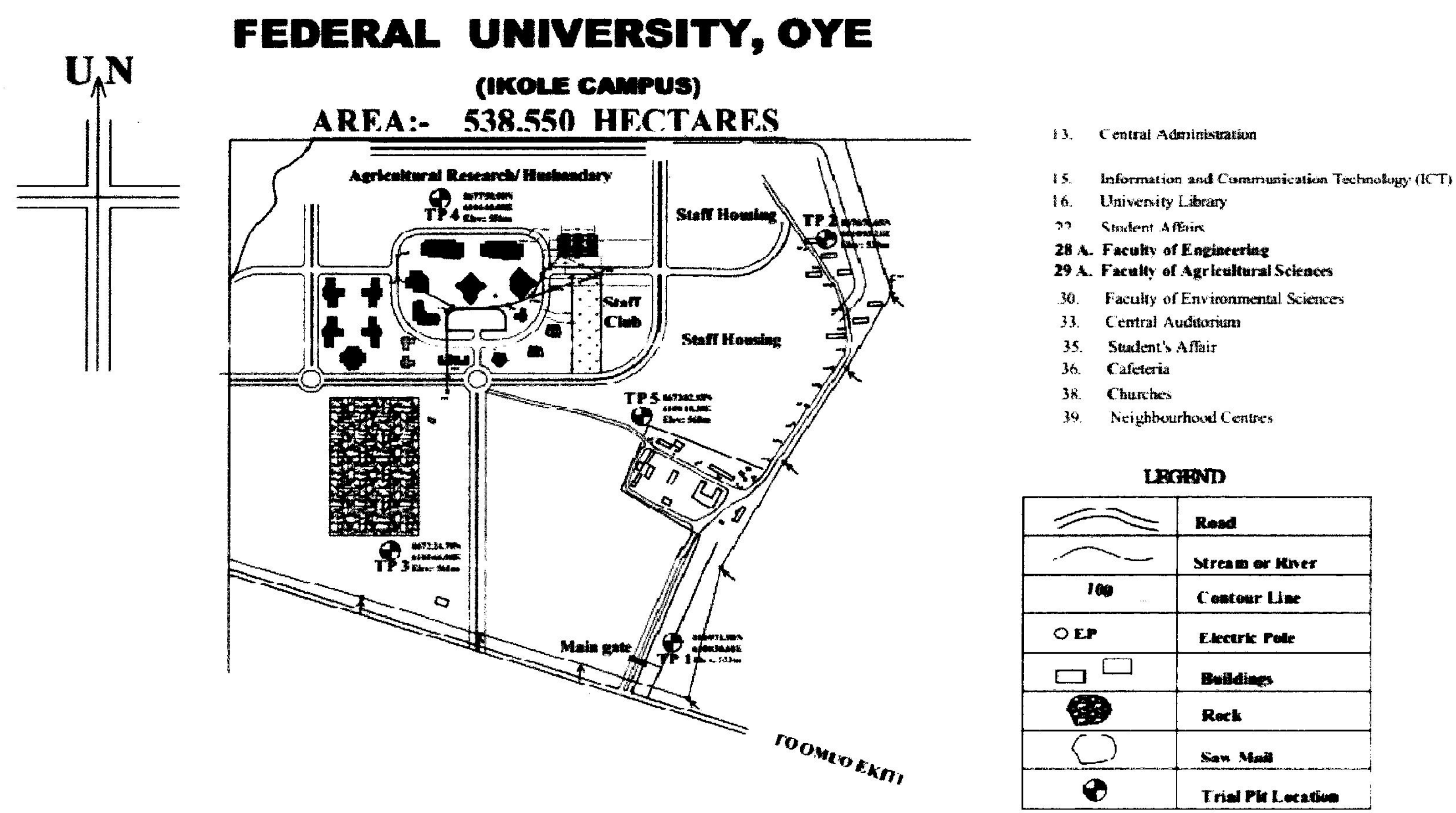


Fig. 1.1: Survey map showing study location and trial pit with coordinate

1.4 Aim and objectives

The aim of this project research is to determine the strength characteristics of soils in Ikole Campus of Federal University Oye Ekiti, Ekiti State, Nigeria. Soil samples were obtained from five spatial locations across the campus so as to detect variation in values of soil properties as shown in map in Fig. 1.1.

The objectives of this research project are;

i. to determine the strength properties of soils such as California Bearing Ratio (CBR) (soaked & unsoaked), drained & undrained cohesion, angle of internal friction.
ii. to obtain range of bearing capacities for soil samples..
iii. to establish the economic value of the soils.
iv. to make necessary recommendations.

1.5 Statement of Problem

Federal University Oye Ekiti, Ikole Campus is prone to development on which structural loads are anticipated to be imposed on its soil in nearest future. The study town, Ikole Ekiti has no current available literature on its soil geotechnical properties hence, the justification for this study.

1.6 Justification of Research

The study is considered to be very important as it will investigate geotechnical properties of soils in the study area. Recommendation will be provided as a guide on possible bearing loads at a described depth. The study will also provide information on the soil properties of the area for future reference.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Geologic Setting of the Study Area

The study area falls within the southwestern part of Nigeria and lots of work have been carried out on the geology of this area. The southwestern Nigeria falls between latitude 7°N and 10° N and longitude 2° E and 7° E which is made up of rocks that are of mainly Precambrian age. Ekiti state lies within Latitudes 7° 15' 00" and 8° 10' 00" North of the Equator and Longitude 4° 45' 00" and 5° 50' 00" East of the Greenwich Meridian. It is underlain by the precambrian rocks of the Basement Complex of Southwestern Nigeria which covers about 50% of the land surface of Nigeria (Figure 1). The Basement Complex forms part of the mobile-belt east of the West African craton and it is polycyclic.

The major lithologic units in Ekiti State are the migmatite-gneiss complex; the older granites; the charnockitic rocks; the slightly migmatised to unmigmatised paraschists and meta-igneous rocks and the un-metamorphosed granitic rocks. The migmatite-gneiss complex is composed mainly of early Gneiss, mafic and ultramafic bands and the granitic or felsic components. Figure 2 show that the rock type is the most widespread rock type, covering about half of the study area. The older granites comprises the porphyritic-biotite granite and the medium-coarse grained granite gneiss. The charnockitic rocks are composed of quartz, alkali feldspars, plagioclase, orthopyroxene, clinopyroxene, hornblende, biotite and accessory amount of opaque ore apatite, zircon and allanite. The slightly migmatised to unmigmatised paraschists and metaigneous rocks consist of pelitic schists, quartzites, amphibolites, talcose rocks, metaconglomerates, marbles and calc-silicate rocks. The umetamorphosed granitic rocks manifest as dolerite dykes, pegmatites and quartz veins, more detail can be seen in Rahaman (1976 & 1988), Olusiji (2013), Bayowa et al. (2007), Omotoyinbo *et. al.* (2008), Jegede (2000), Rahaman et al. (1983).

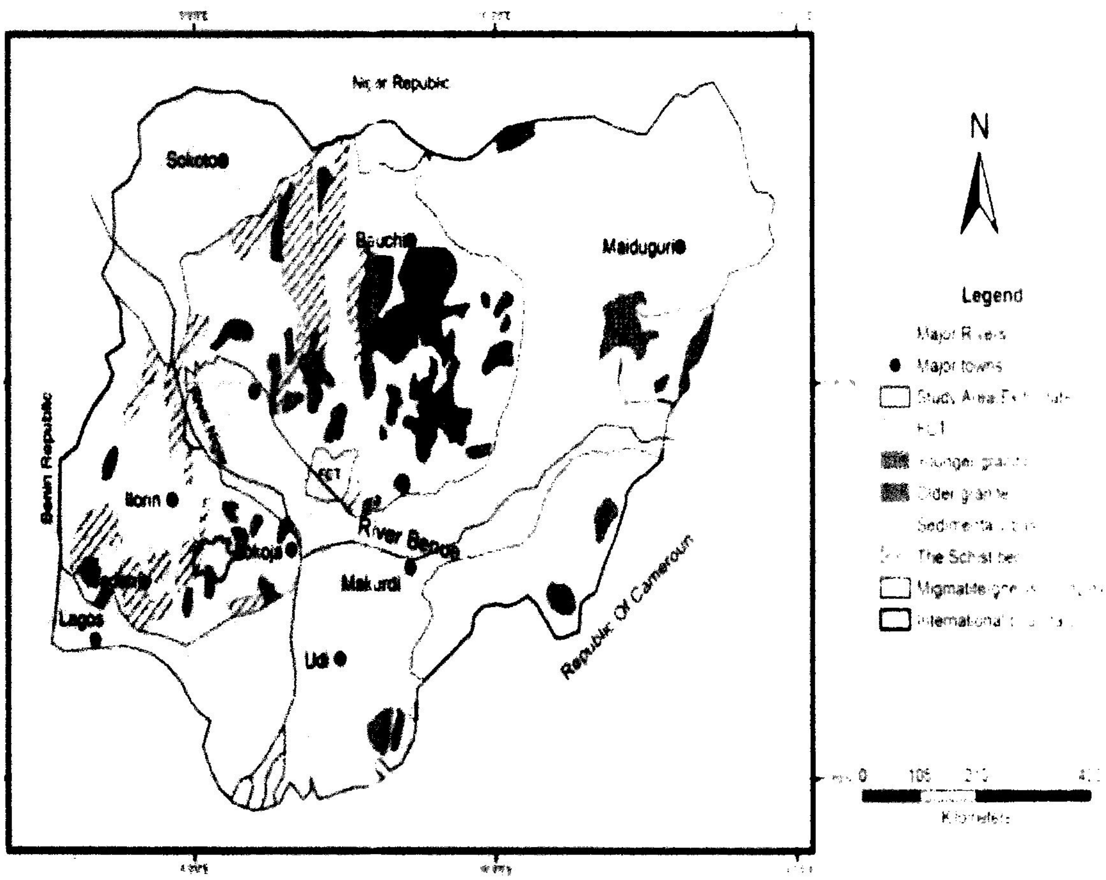


Fig. 2.1: Geological Map of Nigeria (Digitized from Ajibade and Umeji (1989).

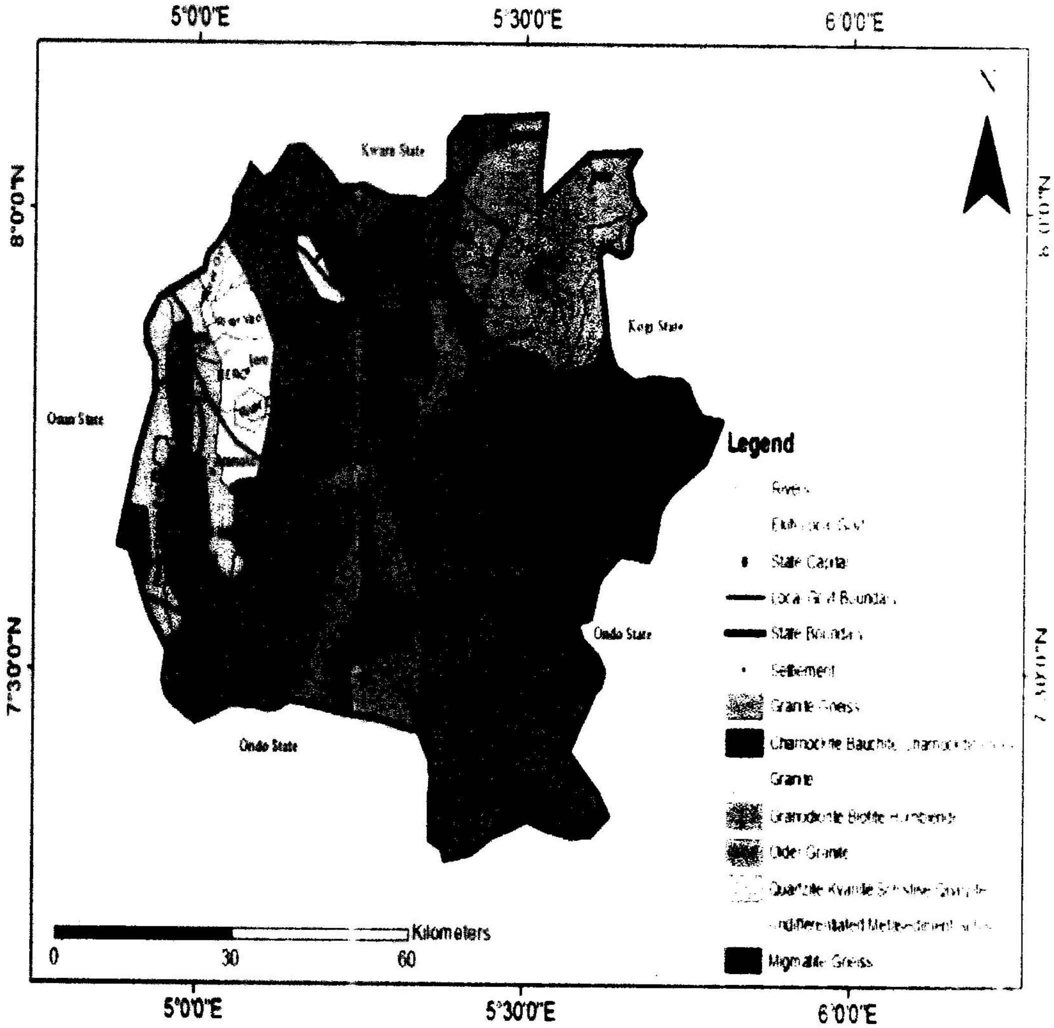


Fig. 2.2: Geological Map of Ekiti-State (Digitized from Ademilua 2014)

2.2 Previous Works on Geotechnical Properties of Soils around Ekiti State

Many attempts have been made by researchers in Ekiti State on improving soils properties (stabilization) using locally available materials or agricultural wastes. The materials used for the improvement of soils are lime-bamboo leaf ash, palm kernel shell ash, sawdust ash, tyre ash, groundnut shell ash, etc. the results show that these materials are good stabilizing agent for improving soils properties especially in road construction, for instance, Dada and Faluyi (2015), Adetoro and Oladapo (2015), Ojo and Omoleye (2015), Adetoro and Adekanmi (2015), Adetoro and Adeyemi (2015), Adetoro and Adams (2015), Afolagboye and Talabi (2013), Adeyemi and Joseph (2015).

The moisture density relationship of clay soils in Ekiti State was examined in which it was observed that the increase in compactive energy (Modified Proctor) causes increase in Maximum Dry Density and decrease in Optimum Moisture Content, Adekanmi and Adebayo (2016). Also, research was made on pavement indices of road that influences its failure in Ado-Ajebandele and analysis showed that failure occurred due to poor base course material, poor sub grade underlying the pavement and effect of ground water due to high water table, Adams *et al.* (2015).

Geotechnical properties of subsoil along some sections of highway in Ekiti State were examined in which the results show that some of the highway subsoil have good soil geotechnical properties (say Igbaraodo-Ikogosi highway) while results also show that the subsoil in some highway are not suitable for construction due to inadequate compaction, high clay content, rise in level of water table, etc. (say Ado-Afao road, Ado-Akure road) gotten from, Abe and Olulope (2014), Jegede and Olaleye (2013), Adams and Adetoro (2014).

From findings, it is obvious that wide range of researches on the geotechnical investigation of soil has been done within Ekiti State and its environs, but none has been carried out in Ikole-Ekiti.

Hence, the need to access the geotechnical properties of soil in Ikole L.G.A., moreso the Federal University Oye-Ekiti has its second campus in Ikole-Ekiti where more constructional activities are anticipated over time. This project research aims at assessing the soil geotechnical properties at Ikole Ekiti as it will provide relevant data and information for subsequent construction of civil work especially within the campus where the samples were obtained.

2.3 Shear Strength Characteristics of Soils

A soil may be considered to have failed to support a built structure if the soil compresses or settles (or swells) to an extent which causes damage to the structure; when reference is made to *failure of a soil*, its failure in shear is usually meant. That the state of stress in the soil is such that the shearing resistance of the soil is overcome and a relative and significant displacement occurs between two parts of the soil mass, Cheng and Jack (2009).

Shear strength is a term used in soil mechanics to describe the magnitude of the shear stress that a soil can sustain, Joseph (2012). The shear strength of a soil in any direction is the maximum shear stress that can be applied to the soil in that direction; when this maximum has been reached, the soil us regarded as having failed, Alam (2003). The bearing capacity of shallow or deep foundations, slope stability, retaining wall design and, indirectly, pavement design are all affected by the shearing strength of the soil in a slope, behind a retaining wall, or supporting a foundation or pavement, Robert and William (1981).

Shearing strength of a soil is the most difficult to comprehend in view of the multitude of factors known to affect it, Venkatramaiah (2006). Basically speaking, a soil derives its shearing strength from the following:

i. Resistance due to the interlocking of particles.
ii. Frictional resistance between the individual soil grains, which may be sliding friction, rolling friction or both.
iii. Adhesion between soil particles of 'cohesion'.

The volume change behavior of soils and inter-particle friction depend on the density of the particles, the inter-granular contact forces, and to a somewhat lesser extent, other factors such as the rate of shearing and the direction of the shear stress, Poulos (1981)

The shear strength of soil depends on the effective stress, drainage conditions, density of the particles, rate of strain, and direction of the strain; Henkel *et al* (1966). Stress-strain relationship of soils, and the shearing strength are factors controlling shear strength of soils and they are affected by the following factors; Poulos (1989):

Soil composition (basic soil material): mineralogy, grain size and grain size distribution, shape of particles, pore fluid type and content, ions on grain and in pore fluid.

Soil State (initial): Defined by the initial void ratio, effective normal stress and shear stress (stress history). State can be described by terms such as: loose, dense, overconsolidated, normally consolidated, stiff, soft, contractive, dilative, etc.

Soil Structure: Refers to the arrangement of particles within the soil mass; the manner the particles are packed or distributed. Features such as layers, joints, fissures, slickensides, voids, pockets, cementation, etc., are part of the structure. Structure of soils is described by terms such as: undisturbed, disturbed, remolded, compacted, cemented; flocculent, honey-combed, single-grained; flocculated, deflocculated; stratified, layered, laminated; isotropic and anisotropic.

Loading conditions of soil: Effective stress path, i.e., drained, and undrained; and type of loading, i.e., magnitude, rate (static, dynamic), and time history (monotonic, cyclic).

According to Craig (2004), shear strength of soil is a function of the normal stress applied, the angle of internal friction, and the cohesion in which the angle of internal friction describes the interparticle friction and the degree of the particle interlocking. This property depends on soil mineral type, soil particle texture/shape/gradation, void ratio, and normal stress. The frictional component of the soil shear strength cannot exist without any normal stress acting on the soil mass. The cohesion describes soil particle bonding caused by electrostatic attractions, covalent link, and/or chemical cementation. So, with normal stress, the angle of internal friction, and cohesion, the following equation, known as the Mohr-Coulomb theory, can be used to find the shear strength of soil under a certain condition:

$$\tau_f = c + \sigma_n \tan \varphi$$

where τ_f = shear strength; c = cohesion; σ= normal stress applied; and φ= angle of internal friction.

This equation can be plotted on an x-y graph with shear stress on the ordinate and normal stress on the abscissa. This is known as a shear failure envelope and is shown in Figure 2.3. Here, the cohesion and the friction angle are represented by the intercept and the slop of the linear curve, respectively. In reality, the shear failure envelope may not be perfectly linear. The degree of electrostatic attraction and cementation of cohesive particles in the soil can cause a slight concave downward curve to form instead.

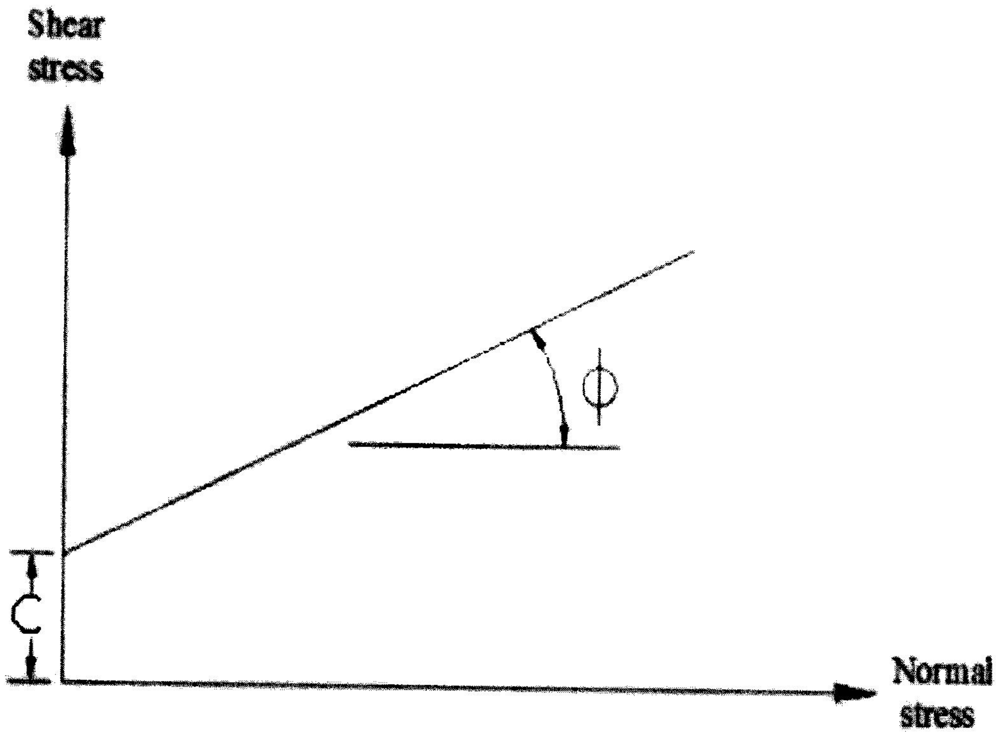


Fig. 2.3: Shear Failure Envelope for Soil (*source: Craig, 2004*)

2.3.1 Concept of Frictional and Cohesive Strengths

The concept of shear strength is that it comprises of two component, friction (φ) and cohesion (c), Das (2006). In terms of classification of soil the component of shear strength can be generalized as follows;

i. Coarse-grained soils, such as gravel and sand, and fine-grained silt, derive strength primarily from friction between particles. Therefore they are considered to be "**cohesionless**" or "**frictional**" soils and are often denoted as "c-soils."

ii. Fine-grained soils, composed mainly of clay, derive strength primarily from the electrochemical attraction, or bond, between particles. Therefore they are considered to be **"cohesive"** soils and are often denoted as "c-soils".

iii. Mixtures of cohesionless and cohesive soils derive strength from both interparticle friction and bonding. Such soils are commonly denoted as "**c- φ soils**."

2.3.2 Strength due to friction

Lane *et al.,* (2001) discussed that the strength due to friction between soil particles is dependent on the stress state of the soil (e.g., overburden pressure) and the angle of internal friction (φ) between the particles. The frictional resistance of soil is equal to the normal stress, times the tangent of friction angle. The tangent of is equal to the coefficient of friction (μ) between the soil particles. The equation for frictional resistance, τ, is written in terms of normal stress, σ_n, as follows:

$\tau = \sigma_n \tan \varphi$; for cohesionless soils where $c = 0$

$\tau = c + \sigma_n \tan \varphi$; for cohesive soils.

The coefficient of friction, φ, between individual particles depends on both their mineral hardness and surface roughness. However, the measured friction angle of a soil sample or deposit also depends on the density of the mass caused by interlocking of particles

2.3.3 Strength due to cohesion

There are two types of cohesion in soils: **true cohesion** and **apparent cohesion**. These are briefly discussed as follows (after Gbenga *et al*, 2009):

a) **True cohesion** may result from chemical cementation (just like in rocks) and/or forces of attraction (e.g., electrostatic and electromagnetic attractions) between colloidal (10-3 mm to 10-6 mm) clay particles. True cohesion is stress-independent unlike frictional resistance that is a function of normal stress.

b) **Apparent cohesion** may develop because of capillary stresses and mechanical interlocking as follows:

i. **Capillary stresses** develop between particles in a partially saturated soil due to surface tension in the water. The surface tension (negative pressure) in the water produces an equal and opposite effective stress between the soil particles, which results in an apparent cohesion since it too is stress-independent. The magnitude of this type of apparent cohesion can be extremely large, especially in fine grained soils. Such capillary stresses can be overcome by an increase in the degree of saturation.

ii. **Apparent mechanical forces** are often exhibited by the interlocking of rough (angular) soil particles. The interlock between the soil particles can offer some resistance to shear stresses even in the absence of a normal stress. This type of apparent cohesion is often the cause of cohesion measured in compacted soils. However, such apparent mechanical forces are susceptible to significant reduction by vibrations and other types of mechanical disturbance.

Figure 2.4 presents a graphical representation of the potential contribution of various mechanisms of cohesion. It can be seen that true cohesion in soils exists only when the particle size is colloidal. Unless the complete soil sample is composed of colloidal particles, true cohesion due to interparticle attraction cannot be relied on. Cementation by deposition is often observed in arid environments (e.g., desert southwest), but it is difficult to quantify. As indicated above, capillary stresses can provide a large apparent cohesion, but such cohesion can be overcome by saturation. Since cohesion cannot be defined with confidence, its contribution to long-term shear strength in c-φ soils is often disregarded or greatly minimized by using only a small value such as 5 to 25 kPa.

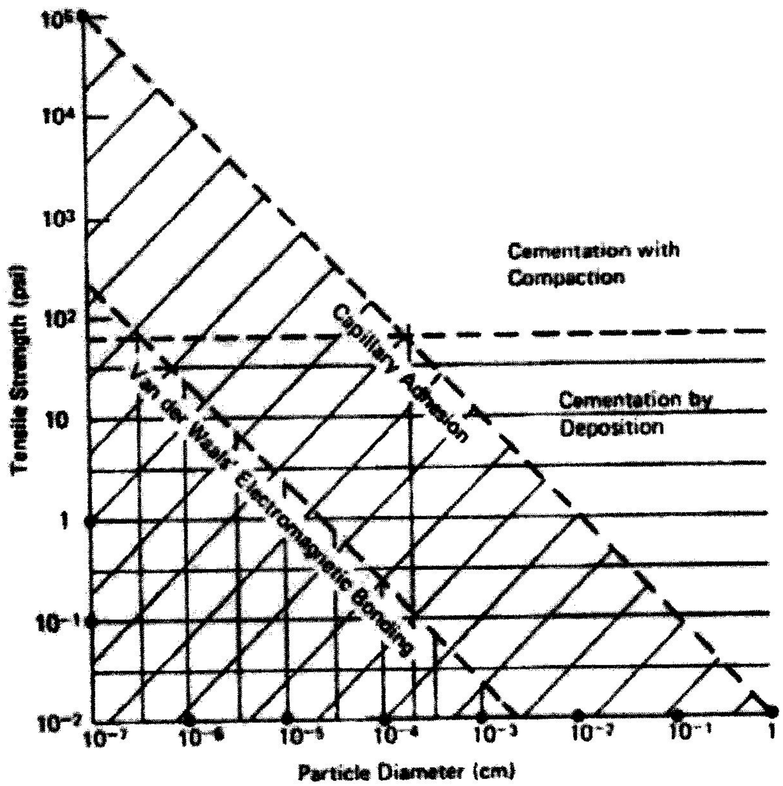


Fig. 2.4: Potential contributions of various bonding mechanisms to cohesive strength (after Ingles, 1964)

It is therefore important that the shearing strength of soil is determined using appropriate approach so that one of the purposes of construction is achieved, i.e. *"safety"*.

CHAPTER THREE

METHODOLOGY

3.1 Preamble

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

3.2 Site Investigation

The site investigation was carried out purposely for the determination of soil strength characteristics which is useful for foundation design. The investigation ranges in scope from a simple examination of the study location, to sample collection and execution of the laboratory tests on soil samples, analysis and discussion of test results.

3.3 Location of Soil Samples

The locations for the collection of samples were chosen with reference to the master plan of Ikole campus of the University. The plan indicates the developing area in which five spatial test points were chosen by picking four edges and center of the proposed developing area.

Trial pits were numbered such that Trial Pit 1 is TP1, Trial Pit 2 as TP2, Trial Pit 3 as TP3, Trial Pit 4 as TP4 and Trial Pit 5 as TP5.

TP 1 was around the campus school gate, TP 2 around the campus school hostel, TP 3 around former FADAMA, TP 4 around Engineering faculty while TP 5 was around the mini-mart. Detail can be got from figure 1.3.

3.4 Collection of samples

The method used for the collection of the samples was the trial pit / hand anger method because it is the cheapest method of soil exploration to shallow depth. The trial pits were excavated using local labourers.

Both disturbed and undisturbed soil samples were taken from each location at depth of 1.5m and 3.0m respectively below the ground surface which sums up into four soil samples per location.

Equipment Used

Shovel, digger, small digger, sacks, and polythene bags

3.4.1 Disturbed Samples

These are soil samples that have their natural state altered or disturbed as the word implies to the soil structure due to change in their physical appearance. They are the type of samples used for soil classification.

The disturbed soil samples were collected from the trial pits at 1.5m and 3m depths with the use of hand, packed into the sack and labeled to avoid misinterpretation of results. Adequate quantities were taken to ensure it will be sufficient for all laboratory tests.

While taking the disturbed samples, little quantity of the disturbed sample is also taken immediately from the trial pit and put into a small sealed polythene bag in order to avoid moisture loss. This soil sample is used in the laboratory for determining the moisture natural content of the soil.

3.4.2 Undisturbed samples

These are soil samples that have their natural state unaltered or undisturbed as the name implies to the soil structure. They are usually obtained as a single unit still in natural compact form. Undisturbed samples are usually obtained in cohesive soils.

The undisturbed soil samples were collected from trial pits at depths 1.5m and 3.0m with the use of a shovel and small digger. This was achievable by using the digger to dig around the chunk of the soil until a satisfied depth of about 175mm is reach and then the shovel was used to form a chamfer between the soil sample and the surrounding in-situ soil all round and then gently lifting the sample until it cut off from the parent soil.

These undisturbed samples were gently carried and lifted out of the pit to avoid breaking and were put immediately into airtight polythene bags so as to maintain natural moisture content.

All the samples collected, both disturbed and undisturbed were taken and transported to the laboratory immediately after obtaining them from each location (trial pit).

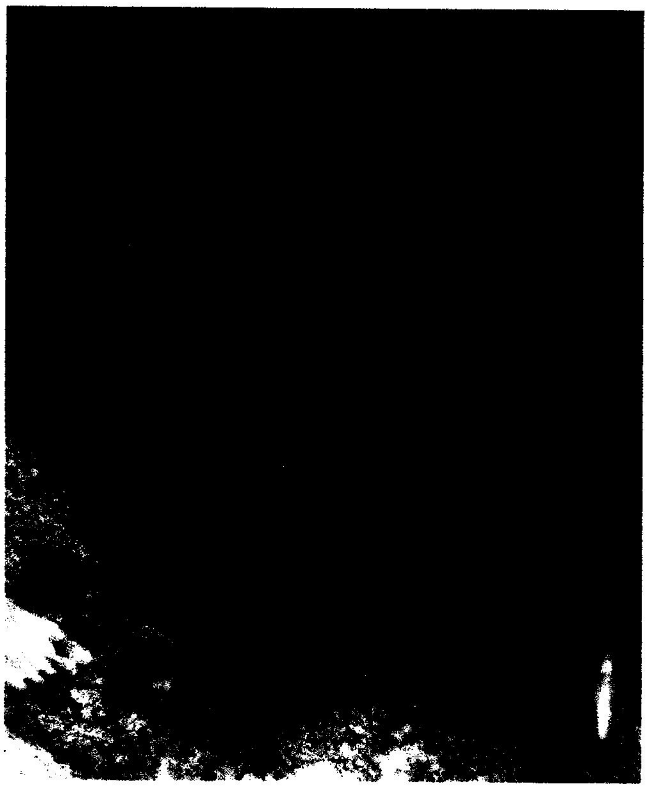


Fig. 3.1: Showing trial pit while trying to cut out undisturbed soil sample

Fig. 3.2: Showing trial pit after bringing out the undisturbed soil sample

Fig. 3.3: Showing student with the undisturbed soil sample from the pit

3.5 Laboratory Tests

In other to assess the soil geotechnical properties, strength characteristics and the physical characteristics of the soils, important laboratory tests which are of significant importance to the engineering objectives of sub soil investigations were carried out in the soil laboratory.

However two categories of tests were carried out for the purpose of this research namely; classification and strength related tests.

3.5.1 Classification Tests

3.5.2 Particle Size Distribution

This test is done to determine the particle size distribution of a soil sample. An oven-dried sample of the soil was weighed and passed through a set of BS sieves and shook thoroughly by using the mechanical sieve shaker as shown in plate 3.4.

The weight of each sieve was recorded and the percentage of sample retained and passing through each sieve was also calculated. The percentage passing was plotted against the sieve sizes on a semi-logarithmic scaled graph.

3.5.3 Specific Gravity

This is measured using standard density bottle. A known weight of oven-dried soil sample Ws is put into a density bottle and is topped up with distilled water and ensured that all air or void from the sample is removed. The bottle is brought to a constant temperature, carefully wiped dry and weighed.

$$\text{Specific gravity (Gs)} = \frac{weight\ of\ soil\ sample}{weight\ of\ equal\ volume\ of\ water}$$

3.5.4 Atterberg / Consistency limits test

These are a basic measure of the critical water contents of a fine-grained soil namely; liquid limit (LL), plastic limit (PL) and shrinkage limit (SL). As a dry, clayey soil takes on increasing amount of water, it undergoes distinct change in behaviour and consistency.

Liquid Limit

Liquid limit (LL) is defined as the minimum moisture content at which the soil will flow under its own weight. It is determined by the standard Casagrande device apparatus, Sowers (1979).

A simple of oven-dried soil all passing the 0.425mm sieve, is mixed with distilled water to a stiff consistency, a portion of it placed in the cup and leveled off parallel to the based. A groove is made through the center of this portion using the grooving tool.

By turning the handle at two revolutions per second the cup is lifted 10mm and dropped on to the rubber base until the bottom of the groove has closed over a length of 10mm. The number of blows at which the groove has closed 10mm is recoded. This is repeated until two consecutive runs give the same number of blows for closure. At this stage the moisture contents of the soil in the cup is determined.

Plastic Limit

Plastic limit (PL) is defined as the minimum moisture content at which the soil can be rolled into a thread 3mm diameter without breaking up, Sowers (1979).

About 20g of the dried soil, all passing the 0.425mm sieve, is mixed with distilled water and molded into a ball. The ball is rolled by hand on a glass plate with sufficient pressure to form a thread. When the diameter of the resulting thread becomes 3mm the soil is kneaded together and then rolled out again. The process is continued until the thread crumbles when it is 3mm diameter, and at this stage the moisture content of the soil is determined. This whole procedure is carried out twice and the average value of moisture content taken as the plastic limit of the soil.

Shrinkage Limit

Shrinkage limit (SL) is defined as the maximum moisture content at which further loss of moisture does not cause a decrease in the volume of the soil, Sowers (1979).

Mix a dried soil passing 0.425mm sieve to a consistency slightly above the expected liquid limit of the soil. Lightly coat the linear shrinkage mold with oil to prevent the soil sticking to the mould. The soils filled into the mould, ensured that no air is trapped and the whole sample is later dried.

The soil bar is measured and recorded as $L_{f,}$ as the original length of the mold is measured and recorded as $L_{0.}$

$$\text{Linear shrinkage L.S} = \frac{L_0 - L_f}{L_f} \times 100$$

3.5.5 Soil Strength Tests

These tests carried out here include essentially the soils' mechanical behaviour related tests namely; the compaction test, quick undrained triaxial compression test and California Bearing Ratio (CBR) test.

1. **Compaction Test (Proctor Test)**

The standard proctor test is a method used finding the optimum moisture content for maximum dry density (MDD) compaction of soil. A cylindrical mold $0.001m^3$ in volume is filled with a soil sample in three layers, each layer being compacted by 25 blows of a standard hammer, (weighing 2.5kg, height of drop 300mm each blow)

The mould is then trimmed and weighed, hence giving the bulk density of the soil. The moisture content of the soil is then determined, and hence the dry density. The experiment is carried out with soil at different moisture contents and a graph of dry density against moisture content is plotted, Cheng and Jack (2009).

2. **Quick Undrained Triaxial Compression Test**

The prepared undisturbed sample is placed in position in the triaxial compression apparatus and the transparent cylinder filled with water. A measured pressure head is applied to the water and the soil is in similar condition to the site condition when the soil has not been touched, where this lateral pressure would be due to the surrounding soil. This lateral pressure or cell pressure is the minimum principal stress.

A vertical load is then applied to the sample at a constant rate of strain until the sample fails. The vertical applied pressure at failure is measured on a proving ring, and when added to the cell pressure gives the maximum principal stress. With several samples of soil with stress at failure with different pressure, a series of Mohr circles are drawn which helped in determining the of undrained angle of friction and apparent cohesion, Cheng and Jack (2009).

3. **California Bearing Ratio**

Approximately 18kg of soil passing 19mm sieve and retained on sieve no. 4 is taken. Moisture and dry density curve is obtained using the standard AASHTO T 99 or T 180. Optimum Moisture Content (OPC) is obtained from the graph between moisture content and dry density. The sample is prepared by adding optimum moisture content and then compact the soil in five layers by applying 10, 30 and 65 blows respectively in three CBR molds using 4.54 kg rammer having 300mm height of fall. The compacted densities of the three specimens range from 95 percent to 100 % of the maximum dry density already determined by the T 180 compaction test.

Soaking was done by placing the swell plate with adjustable stem on the soil sample in the mould and sufficient annular weights was applied to produce an intensity of loading equal to the mass of sub-base and base courses and surfacing above the tested material, but not less than 4.54 kg. The tripod was later placed with dial indicator on top of the mould and initial dial reading is taken.

The mould immersed in water to allow free access of water and the sample placed in water for 96 hours (4 days). Dial guage reading was made on soaked specimen and swell calculated as a percentage of initial sample height. The sample was remove from the tank and allowed to drain for 15 minutes.

Penetration Test: The mould was placed on the loading frame and its potion adjusted until the piston is centered on the specimen. Seat the penetration piston with a 4.54kg load, and set both the load dial and the strain dial to zero. This initial load is considered as the zero load when determining the stress-penetration relationship. Surcharge weights was placed on the specimens equal to that used during soaking and load applied at a rate of 1.3 mm / min and the loads for penetration of 0.625mm, 1.250mm, 1.875mm, 2.500mm and so on up to 12.500mm were recorded.

Stress strain curve: Curves of load versus penetration for each specimen is plotted and readings of load for 2.500mm and 5.000mm penetration is taken and CBR for both penetrations were determined. The greater of the values is the required CBR for that specimen. Also find the dry density for each specimen.

$$\text{CBR value} = \frac{\textit{test load value}}{\textit{standard load}} \times 100$$

Design CBR: it is calculated by plotting a graph between CBR values and dry densities of all the three specimens and then calculating the design CBR against value of 85 % maximum dry density, Cheng and Jack (2009)

CHAPTER FOUR

RESULT AND DISCUSSION

This chapter presents the results of various geotechnical test carried out on the five trial pits used. Tables, graphs and result were also discussed in this chapter.

The tests carried out include natural moisture content test, specific gravity, particle size analysis, Atterberg limits, compaction, California Bearings Ratio and tiraxial compression tests.

4.1 Natural Moisture Content Test

Results of the natural moisture content (MC) are shown in Table 4.1 below. From the result all the trial pits have low values of moisture content which show the soils have low potential of water retention. They all have values ranging between 15.0% and 25.0% respectively.

Table 4.1: Results of Moisture Content of Soils.

	TP 1		TP 2		TP 3		TP 4		TP 5	
Depth	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m
MC (%)	16.7	21.3	21.3	22.6	15.9	25.0	22.2	22.7	21.5	25.0

4.2 Specific Gravity

The summary results of specific gravity (S.G) are shown in Table 4.2 below. From the results, the S.G varies in depth by a slight difference. TP1, TP2, TP3, TP4 and TP5 have values ranging between 2.35 – 2.58. This value range shows that the soil samples have some clay content since 2.36 is the average value for clay content.

Table 4.2: Results of Specific Gravity of Soils.

	TP 1		TP 2		TP 3		TP 4		TP 5	
Depth	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m
Specific gravity	2.58	2.41	2.40	2.36	2.44	2.35	2.38	2.35	2.37	2.47

4.3 Particle Size Analysis

The particle size distributions of samples are shown in the Table 4.3 below while the raw results graphs are provided in appendix 1. The average distributions are:

Table 4.3: Results of Particle Sizes of Soils.

	TP 1		TP 2		TP 3		TP 4		TP 5	
Depth	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m	1.5m	3.0m
Gravel	99.7	100	99.8	100	88.9	83.4	99.8	99.5	98.8	100
Sand	67.5	66.4	74.8	75.0	39.3	30.5	74.5	77.7	66.2	70.8
Clay	57.2 > 35%	51.9 > 35%	64.6 > 35%	62.7 > 35%	32.3 < 35%	22.0 < 35%	64.7 > 35%	63.6 > 35%	59.0 > 35%	62.0 > 35%
Group Class - fication	A-7-5 or A-7-6	A-7-5 or A-7-6	A-7-5 or A-7-6	A-7-5 or A-7-6	A-2-5 or A-2-6	A-2-5 or A-2-6	A-7-5 or A-7-6	A-7-5 or A-7-6	A-7-5 or A-7-6	A-7-5 or A-7-6
General subgrad e rating	Fair to Poor				Excellent to Good		Fair to Poor			

From the table above of the result analysis, over 75% of the soils are fine material passing through 0.075mm sieve.

TP1, TP2, TP4 and TP5 at depth 1.5m and 3.0m has 60% fines, fraction ranges between 51.9% and 64.7%, i.e. > 35% except TP3 at depth 1.5m and 3.0m that has 22.0% and 32.3% as passing through 0.075mm sieve. It was deduced that the soil samples in TP1, TP2, TP4 and TP5 are generally classified as Silty-Clay soil material with good percentage of sand and finer fraction while only TP3 is generally classified as a granular soil because it has less than 35% of particles that passed through sieve 0.0075mm.

4.4 Atterberg Limit Test

Appendix 3 shows the results of the Atterberg limlit tests carried out on the soil samples from the trial pits. The liquid limit, plastic limit, shrinkage limit and plasticity index values are presented in this appendix. The values of the liquid limit were obtained from the graphs shown in appendix 3. It was observed from the results obtained from the liquid limit, plastic limit and plasticity index, the soil samples from TP1, TP2, TP4 and TP5 can be grouped as A-7-5 or A-7-6 class i.e. clayey soil because the liquid limit values range between 31.80 and 58.50% while TP3 is classified as A-2-6 i.e. lateritic soil because the liquid limit values range between 21.56 and 30.00% according to AASHTO classification system (1978).

4.5 Compaction and California Bearing Ratio Tests Result

The results and graphs are shown in appendix 4 and 5. From the results obtained, the analysis is as follows, the Optimum Moisture Content and Maximum Dry Density were derived from the graphs.

The results in appendix 4 show that TP1, TP2, TP3, TP4 and TP5 have Optimum Moisture Content ranging between 17.0 – 21.5% and Maximum Dry Density ranging between17.5 – 18.0 kg/m^3. From compaction classification using Maximum Dry Density, it shows that all the soil samples from the trial pits are subgrade material which fall within good and fair for construction. Appendix 5 shows the results and graphs of the California Bearing Ratio test. The CBR test is used to determine the suitability of soils as highway materials (subgrade, subbase and base course).

From the results in appendix 5, outcome of the CBR values at 2.5 and 5.0 penetrations show that soil samples from TP1 and TP3 at 5.0 penetration with values ranging between 81.9 – 89.5 met the standard specification of BS 1377 and FMW Nig. (1997) for sub-base > 80%. TP2 and TP4 have extremely low value ranging between 2.3 – 9.1 at both penetrations and TP5 has average value 40.0 – 56.3 at 5.0 penetration. This implies that TP2, TP4 and TP5 do not meet the standard specification of BS 1377 and FMW Nig. (1997) for sub-base < 80%.

4.6 Triaxial Test

Appendix 6 shows the results and graphs of the Quick Undrained Unconsolidated Triaxial test carried out on the soil samples. Method used was the unconsolidated undrained triaxial test according to BS 1377; part 6 and 7 1990, all the soil samples were subjected to confining pressure of 100kN, 200kN and 300kN respectively. Deviator stresses were obtained at the collapse load and the deformed surface areas of the samples at point of failure. The Mohr's circles were drawn for all the soil samples and the values of cohesion (C) and internal friction angle (ϕ) were obtained from the graphs. The highest C was at TP1 depth 1.5m which was $190kN/m^2$ while the lowest C was at TP4 depth 3.0m which was $9kN/m^2$. The highest ϕ was at TP3 depth 3.0m which was 41° while the lowest ϕ was at TP1 depth 1.5m which was 21°.

These results show that the TP2 @ 3.0m, TP3 @ 3.0m and TP4 @ 3.0m have very low cohesive value ranging between 9 – 12 kN/m^2 which implies very weak shear strength, TP2 @1.5m, TP1 @ 3.0m, TP3 @ 1.5m, TP4 @ 1.5m and TP5 @ 3.0m have high cohesive value ranging between 50 – 93 kN/m^2 which implies average shear strength but TP1 @ 1.5m has very high cohesive value of $190kN/m^2$ which implies good shear strength.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The following conclusions were drawn from this research;

All the soils in the study location have low potential of water retention with their natural moisture content not exceeding 25% and all the soils have clay content from the specific gravity result.

TP1, TP2, TP4 and TP5 can generally be classified as Silty-Clay soil material with good percentage of sand and finer fraction and are fair to poor in general subgrade rating while only TP3 is generally classified as a granular soil and is rated as excellent to good in general subgrade rating. It was also observed from the Atterberg test results that the soil samples from TP1, TP2, TP4 and TP5 can be grouped as A-7-5 or A-7-6 class i.e. clayey soil while TP3 is classified as A-2-6 i.e. lateritic soil according to AASHTO classification system (1978).

From compaction classification using Maximum Dry Density, it shows that all the soil samples from the trial pits are subgrade material which fall within good and fair for construction. Outcome of the CBR test shows that soil samples from TP1 and TP3 met the standard specification of BS 1377 and FMW Nig. (1997) as a material for sub-base while TP2, TP4 and TP5 do not meet the standard specification of BS 1377 and FMW Nig. (1997) for sub-base material.

It was observed from the quick undrained triaxial test that from the same trial pit in some locations, the cohesion value is higher at 1.5m depth than 3.0m depth i.e. that of the TP2 @ 3.0m, TP3 @ 3.0m and TP4 @ 3.0m have very low cohesive value ranging between 9 – 12 kN/m^2 which implies very weak shear strength, TP2 @1.5m, TP1 @ 3.0m, TP3 @ 1.5m, TP4 @ 1.5m and TP5 @ 3.0m have high cohesive value ranging between 50 – 93 kN/m^2 which implies average shear strength but TP1 @ 1.5m has very high cohesive value of 190kN/m^2 which implies good shear strength.

5.2 Recommendation

These valuable data obtained from this geotechnical analysis can be useful for civil engineers in the design and construction of structures and roads in Ikole campus and environs for maximum durability and efficiency.

Also, it is recommended that stabilization should be done for areas with clayey particles and low angle of internal friction. Classification result will assist in determining the type of stabilization applied.

It is also recommended that engineering confirmatory tests be carried out before embarking on any civil engineering construction and further research should be carried out on other locations in the campus in order to have concrete characterization of the entire campus.

REFERENCES

Abe O. E. and Olulop O. R. (2014), Assessment of the effects of surface and subsurface moisture on ADO-AFAO road pavement failure in Ekiti-State, Nigeria, *International Journal Of Engineering And Computer Science*, Vol. 3 Issue 6, pp. 6319 – 6324. Available at www.ijecs.in

Adams J. O., Aderinola O. S. and Akinwamide J. T. (2015), Geotechnical Study of Pavement Indices Influencing Failure Along Ado - Ajebandele - Ikere Road, South Western, Nigeria, *International Journal of Engineering Innovation & Research*, Vol. 4, Issue 4, pp. 567 – 572.

Adams J.O. & Adetoro A. E. (2014), Analysis of road pavement failure caused by soil properties along Adoekiti - Akure road, Nigeria, *International Journal of Novel Research in Engineering and Applied Sciences (IJNREAS)*, Vol. 1 Issue 1, pp. 1 – 7. Available at www.avidpublishers.ca

Ademilua, O.L. (2014), A Geoelectric and Geologic Evaluation of Groundwater potential of Ekiti and Ondo States, Southwestern, Nigeria, *International Journal of Science and Technology*, Vol. 4 Issue 3, pp. 1 - 67. Available at www.ejournalofsciences.org

Adekanmi J. S and Adebayo V. B. (2016), MoistureDensity Relationship of Selected Clay Soils in Ekiti State, Nigeria, *Electronic Journal of Geotechnical Engineering*, Vol. 21 Issue 16, pp. 5421- 5428. Available at www.ejge.com

Adetoro E. A. and Adam O. J. (2015), Analysis of Influences of Locally Available Additives on Geotechnical Properties of Ekiti State Soil, Southwestern, Nigeria, *International Journal of Innovative Research in Science, Engineering and Technology*, Vol. 4, Issue 8, pp. 7093 – 7099.

Adetoro E. A. and Adam J. O. (2015), Research article comparative analyses of Ekiti state soil stabilized with different additives, *Asian Journal of Science and Technology,* Vol. 06, Issue, 12, pp. 2054 – 2058. Available at http://www.journalajst.com

Adetoro A. E. and Adekanmi S. J. (2015), Evaluation of Presence of Sawdust and Palm Kernel Shell Ashes on Geotechnical Properties of Ekiti State Soil, *Journal of Multidisciplinary Engineering Science and Technology (JMEST)*, Vol. 2 Issue 11, pp. 3184 – 3188. Available at www.jmest.org

Adetoro A. E. & Adekanmi J. S. (2015), Potentials Of Palm Kernel Shell And Sawdust Ashes For Stabilization Of Gbonyin Local Government Area Soil, Nigeria, *Journal of Multidisciplinary Engineering Science and Technology (JMEST),* Vol. 2 Issue 8, pp. 2315 – 2319. Available at www.jmest.org

Adetoro A.E. and Oladapo. S.A. (2015), Effects of Sawdust and Palm Kernel Shell Ashes on Geotechnical Properties of Emure / Ise-orun Local Government Areas Soil, Nigeria, *Scientific Research Journal (SCIRJ),* Vol. 3 Issue 7, pp. 35 – 38. Available at www.scirj.org

Adetoro A. E. & Ayeni I. S. (2015), Effects Of Locally Available Additives On Geotechnical Properties Of Ijero Local Government Soil, Ekiti State, Nigeria, *Journal of Multidisciplinary Engineering Science and Technology (JMEST),* Vol. 2 Issue 11, pp. 2994 – 2997. Available at www.jmest.org

Afolagboye O. L. and Talabi A. O. (2013), Consolidation properties of compacted lateritic soil stabilized with tyre ash, *Journal for Engineering and Manufacturing Technology, JEMT,* Vol. 1, pp. 36 – 44. Available at www.bluepenjournals.org/jemt

Ajibade, A.C and Umeji, A.C. (1989), *Precambrian Geology of Nigeria Publication, Geology Survey of Nigeria Kaduna.* Elizabeth publishing Company, Nigeria. pp.11 – 41.

Alam Singh (2008), *Modern Geotechnical Engineering*, 3rd edition, CBS Publishers & Distributors, New Delhi – 110002.

Bayowa O. G., Olorunfemi O. M., Akinluyi O. F., and Ademilua O. L.(2007), A Preliminary Approach to the Groundwater Potential Appraissal of Ekiti State, Southwestern Nigeria, *Journal of Engineering and Applied Sciences*, Vol. 2 Issue 7, pp. 1169 – 1172.

Bjerrum, L. (1974). "*Problems of Soil Mechanics and Construction on Soft Clays*" Norwegian Geotechnical Institute, Publication No. 110, Oslo.

Bowles, J. E. (1978), *Engineering Properties of Soils and their Measurement*, 2nd edition, McGraw-Hill Book Company, New York, 1978.

Bowles, J. E. (1996), *Foundation Analysis and Design*, 5th edition, McGraw-Hill Book Company, New York.

British Standards Institution (1999) BS 5930, *Code of Practice for Site Investigations.*

Cernica, J.N. (1995), *Geotechnical Engineering Foundation Design*, Wiley, New York.

Cheng L. & Jack B. E, (2009), *Soil Properties, Testing, Measurement and Evaluation*, 6th edition, Pearson Education Inc., Upper Saddle River, New Jersey 07458.

Chesworth, Ward, ed. (2008), *Encyclopedia of soil science*, Dordrecht, Netherlands: Springer. xxiv. ISBN 1-4020-3994-8.

Craig R. F. (2004), *Craig's soil mechanics,* 7th edition, Spon Press, 29 West 35th Street, New York, NY 10001, pg. 91 – 92.

Craig, R.F. (1992), *Soil Mechanics*, Chapman and Hall, London.

Dada M. O. & Faluyi, S. O. (2015), Physical Properties of Lime – Bamboo Leaf Ash Treated Samples of Lateritic Soils in Ado – Ekiti, Nigeria, *Global Journal of Engineering Design & Technology (G.J. E.D.T.),* Vol. 4 Issue 4, pp. 4 – 8.

Das B. M. (2006), *Principles of Geotechnical Engineering*, 7th edition, Cengage Learning, 200 First Stamford Place, Suite 400 Stamford, CT 06902 USA.

Gbenga M. A., Oluwole A. A., and Shodolapo O. F. (2009), Influence of Calcium Sulphate on Subsoil \Cohesion and Angle of Internal Friction, *Journal of Applied Sciences Research,* Vol. 5 Issue3, pp. 297 – 304.

Gilluly, J., Waters, A. C. and Woodford, A. O. (1975), *Principles of Geology,* (4th ed.). W.H. Freeman. ISBN 978-0716702696.

Henkel, D. J.; Wade, N. H. (1966), Plane Strain Tests on a Saturated Remolded Clay, *Journal of the Soil Mechanics and Foundation Division,* ASCE 92 (SM 6), pp. 67–80.

Holtz, R. and Kovacs, W., (1981), *An Introduction to Geotechnical Engineering*, Prentice Hall, Inc. ISBN 0-13-484394-0.

Ingles O. G. (1968), Soil Chemistry relevant to the Engineering Behaviour of Soils, *Soil Mechanics – Selected Topics, (*Ed. Lee). Butterworths, London.

Irving S. D., Loren R. A. and Fred W. K. (1980), *Fundamentals of Geotechnical Analysis*, John Wiley & Sons, Inc., New York, 1980.

Jegede O. G. (2000), Effect of soil properties on pavement failures along the F209 highway at Ado-Ekiti, Southwestern Nigeria, *Journal for Construction and Building Materials,* Vol. 14, pp. 311-315.

Jegede, O. G. and Olaleye, B. M. (2013), Evaluation of Engineering Geological and Geotechnical Properties of Sub Grade Soils along the Re-Aligned Igbara-Odo Ikogosi Highway, South Western, Nigeria, *The International Journal Of Engineering And Science (IJES),* Vol. 2 Issue 5, pp. 18 – 28. Available at www.theijes.com

Joseph, P. G. (2012), Physical Basis and Validation of a Constitutive Model for Soil Shear Derived from Micro-Structural Changes, *International Journal of Geomechanics,* doi:10.1061/(ASCE)GM.1943-5622.0000209

Journal of Mechanical and Civil Engineering (IOSR-JMCE) 2012. Vol. 4 Issue 4, pp. 01 – 04. Available at www.iosrjournals.org

Kayode, J.S. and Adelusi, A.O. (2010), Ground Magnetic Data Interpretation of Ijebu-Jesa Area, Southwestern Nigeria, using total component, *Research Journal of Applied Sciences, Engineering and Technology* Vol. 2 Issue 8, pp. 703 – 709.

Kezdi, A. and Rethati, L. (1988), *Soil Mechanics of Earthworks, Foundations and Highway Engineering,* Handbook of Soil Mechanics, Volume 3. Elsevier.

Oberg, A and Salfours, G. (1997), Determination of shear strength parameters of unsaturated silts and sands based on the water retention curve. *Geotechnical Testing Journal, GTJODJ,* Vol. 20, pp. 40-48.

Ojo M. D. and Omoloye E. A. (2015), The Relationship between the OMC and MDD of Asphalt – Dissolved Salt Stabilized Soils in Ado-Ekiti, Nigeria, *Global Journal of Engineering Design & Technology (G.J. E.D.T.),* 2015. Vol.4 Issue 4, pp. 1-3.

Oladapo, M.I. (2013), Hydrogeophysical investigation in selected parts of Irepodun/Ifelodun Local Government Area of Ekiti State, Southwestern Nigeria, *Journal of Geology and Mining Research,* Vol. 5, Issue 7: pp. 200-207.

Olusiji S. A. (2013), Geology and structure of the precambrian rocks in Iworoko, Are and Afao area, Southwestern Nigeria, *International Research Journal of Natural Sciences,* Vol.1 Issue 1, pp.14 – 29. Available at www.ea-journals.org

Omotoyinbo, (1994), Geology of Ado-Ekiti, Unpublished thesis, Ondo State University Ado- Ekiti.

Omotoyinbo O. S, Olusoji S. E. & Francis C. O. (2008), Influence of Rock Mineralogy on Subsurface Water in Ado-Ekiti, Nigeria, *African Research Review,* Vol. 2 Issue 2, pp. 175 – 186.

Poulos, S. J. (1971), *The Stress-Strain Curve of Soils (PDF),* GEI Internal Report

Poulos, S. J. (1981), *The Steady State of Deformation*, Journal of Geotechnical Engineering 107 (GT5), pp. 553–562

Poulos, S. J. (1989), Jansen, R. B., ed., *Liquefaction Related Phenomena* (PDF), Advanced Dam Engineering for Design (Van Nostrand Reinhold), pp. 292–320

Rahaman A.A. and Malomo S. (1983)., *Sedimentary and Crystalline Rocks of Nigeria,* In: S. A. Ola (Ed.) Tropical Soils of Nigeria in Engineering Practice. A.A. Balkerma, Netherlands, pp. 17 – 38.

Rahaman, M.A. (1976), *Review of the Basement Geology of Southwestern Nigeria. In Geology of Nigeria,* Elizabeth publishing Company, Nigeria. pp. 23 – 33.

Rahaman, M.A. (1988), *Recent advances in the study of the Basement Complex of Nigeria. In Oluyide et.al. (eds) Precambrian Geology of Nigeria, Publication,* Geological Survey of Nigeria, Kaduna. Elizabeth publishing Company, Nigeria. pp. 157-163.

Ramamurthy T. N. and Sitharam T.G. (2010), *Geotechnical Engineering, Soil Mechanics*, 3rd edition, S. Chand & Company Ltd., Ram Nagar, New Delhi-110055.

Robert D. H. & William D. K. (1981), *An introduction to Geotechnical Engineering*, Prentice Hall Publishers, Englewood cliffs, New Jersey – 07632.

Shittu, O. S. and Fasina, A. S. (2004), Cassava Yield as affected by different Fertilizer Models at Ado- Ekiti, *Nigerian Journal of Soil Science,* Vol. 14, pp. 66 – 73.

Siyan Malomo (2011), *Framework for and opportunities for and opportunities for sustainable private sector participation in in solid minerals development in Ekiti state solid minerals development in Ekiti state* at Ekiti State Economic Development Summit, 15th of October, 2011.

Sowers (1979), *Introductory Soil Mechanics and Foundations: Geotechnical Engineering,* 4th Ed., Macmillan, New York. (as referenced in Coduto, 1999. Geotechnical Engineering: Principals and Practices. Prentice Hall. New Jersey.)

Townsend, W.N. (1973) An Introduction to the Scientific Study of Soil, Butler and Tanner, Frome and London.

Venkatramaiah. C. (2006), *Geotechnical Engineering,* 3rd edition. New Age International (P) Ltd. Publishers, New Delhi – 110002.

APPENDIX ONE

PARTICLE SIZE DISTRIBUTION

RESULTS OF PARTICLE SIZE DISTRIBUTION

PIT 1

	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	0	0	100
4.75	1.3	0.26	99.7	0	0	100
2.36	13.3	2.66	97.1	5.6	1.12	98.9
1.18	29.8	5.96	91.1	27.3	5.46	98.4
600	44.7	8.94	82.2	45.4	9.08	84.3
300	49.2	9.84	72.3	55.1	11.02	73.3
150	48.2	9.64	62.7	69.4	13.88	59.4
75	27.3	5.46	57.2	37.6	7.52	51.9

PIT 2

	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	0	0	100
4.75	1.2	0.24	99.8	0	0	100
2.36	8.2	1.64	98.1	1.4	0.28	99.7
1.18	17.8	3.56	94.6	10.6	2.12	97.6
600	35.6	7.12	87.4	32.2	6.44	91.2
300	43.0	8.60	78.8	55.9	11.18	80.0
150	40.9	8.18	70.7	49.7	9.94	70.0
75	29.7	5.94	64.7	36.7	7.34	62.7

PIT 3

	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	55.5	11.1	88.9	83.0	16.60	83.4
4.75	89.4	17.88	71.0	70.9	14.18	69.2
2.36	43.9	8.78	62.2	37.3	7.46	61.8
1.18	35.1	7.02	55.2	44.0	8.80	53.0
600	33.1	6.62	48.6	47.6	9.52	43.4
300	33.1	6.62	42.0	46.0	9.20	34.2
150	27.6	5.52	36.5	37.5	7.50	26.7
75	20.9	4.18	32.3	23.7	4.74	22.0

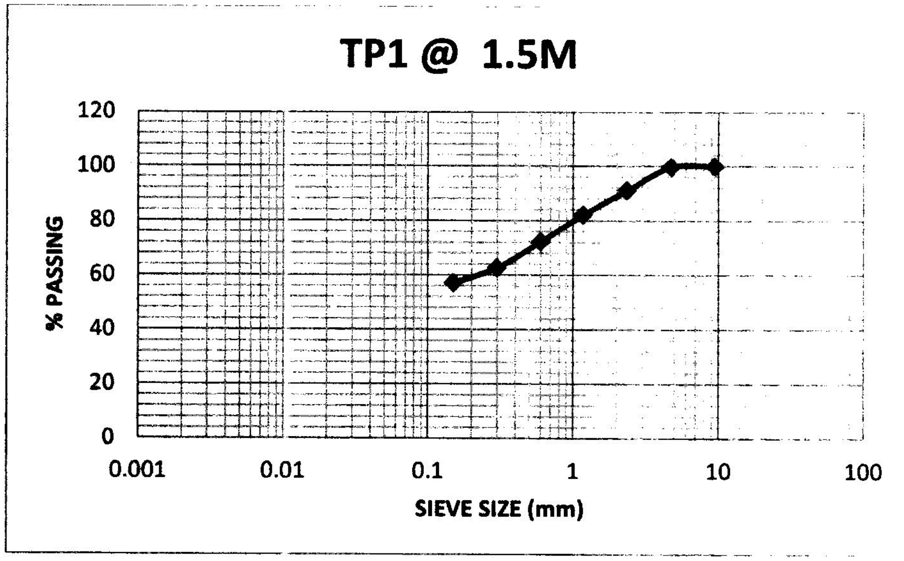
PIT 4

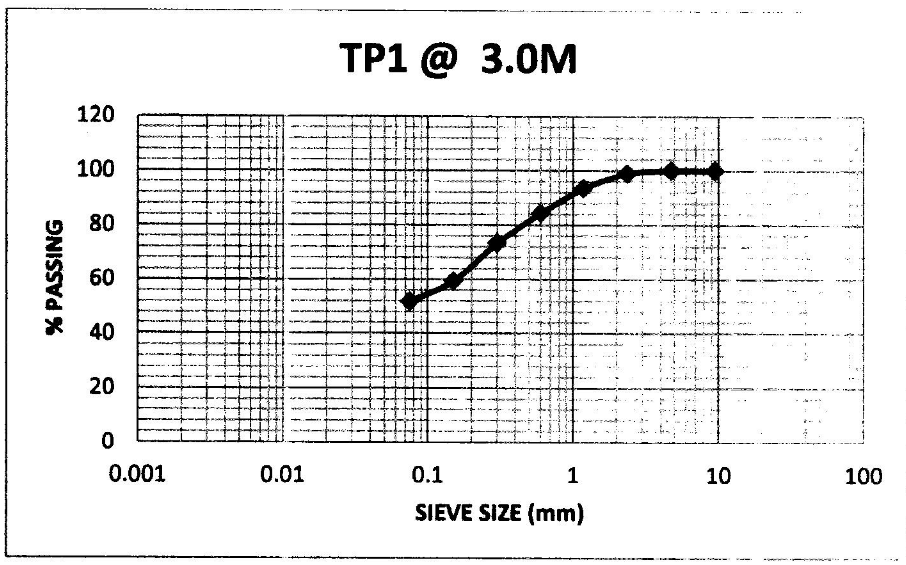
	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing(%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	2.6	0.52	99.5
4.75	1.1	0.22	99.8	4.8	0.96	98.5
2.36	8.9	1.78	98.0	8.1	1.62	96.9
1.18	19.1	3.82	94.2	16.4	3.28	93.6
600	35.5	7.1	87.1	32.6	6.52	87.1
300	42.7	8.54	78.5	40.0	8.00	79.1
150	40.2	8.04	70.5	44.6	8.92	70.2
75	29.1	5.82	64.7	33.0	6.6	63.6

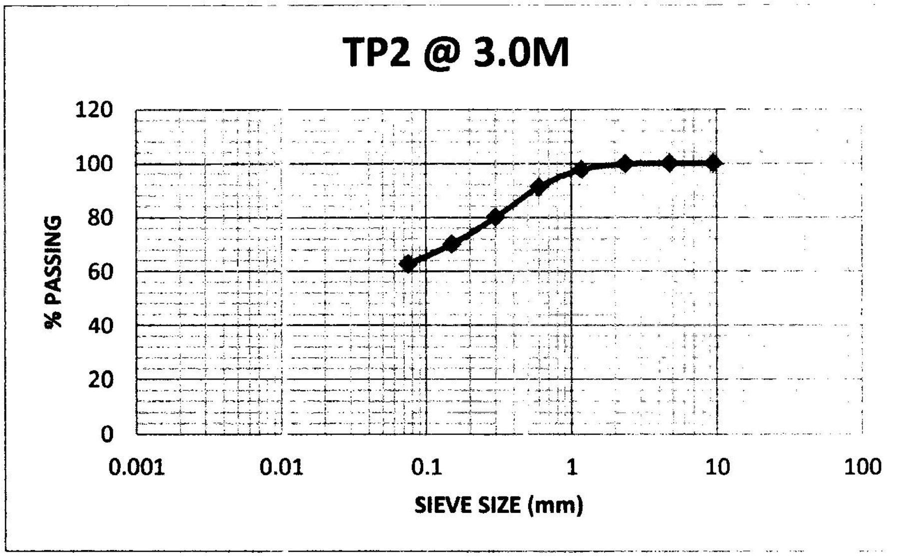
PIT 5

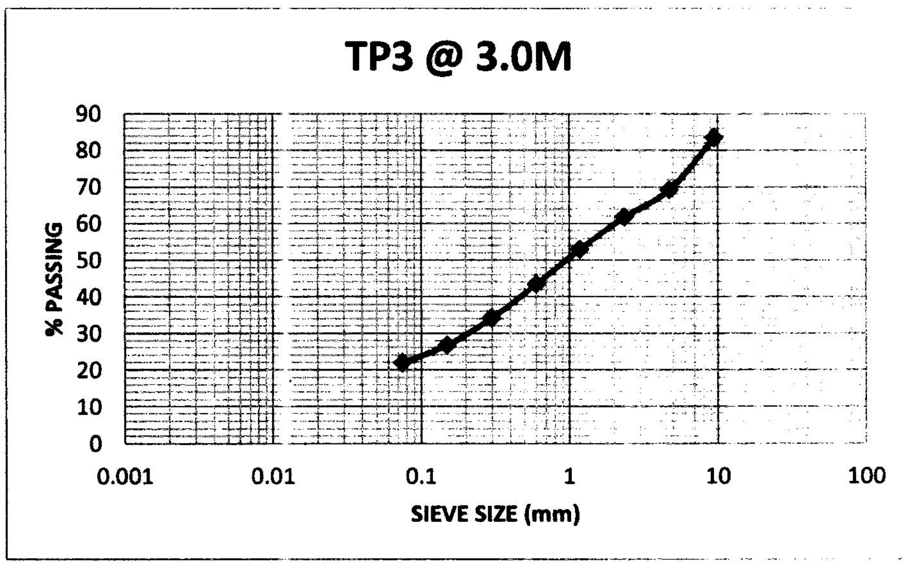
	1.5m depth			3.0m depth		
Sieve No	Weight Retain	Percentage Retain (%)	Percentage Passing (%)	Weight Retain	Percentage Retain (%)	Percentage Passing (%)
9.50	0	0	100	0	0	100
4.75	6.0	1.2	98.8	0	0	100
2.36	22.9	4.58	99.2	8.6	1.72	98.3
1.18	36.7	7.34	86.9	27.9	5.58	92.7
600	44.9	8.98	77.9	45.9	9.18	83.5
300	42.3	8.46	69.4	43.8	8.76	74.76
150	32.4	6.48	63.0	39.4	7.88	66.9
75	19.7	3.94	59.0	24.6	4.92	61.96

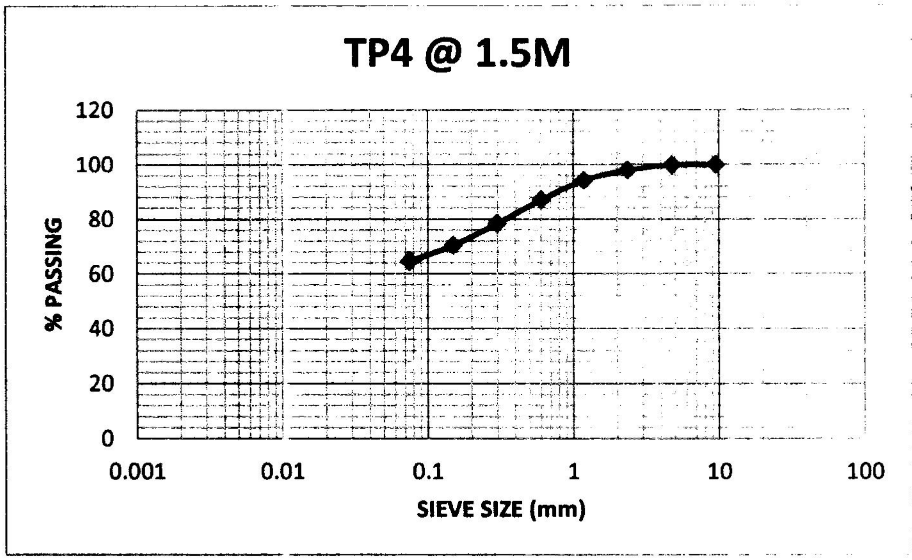
GRAPHS OF PARTICLE SIZE DISTRIBUTION



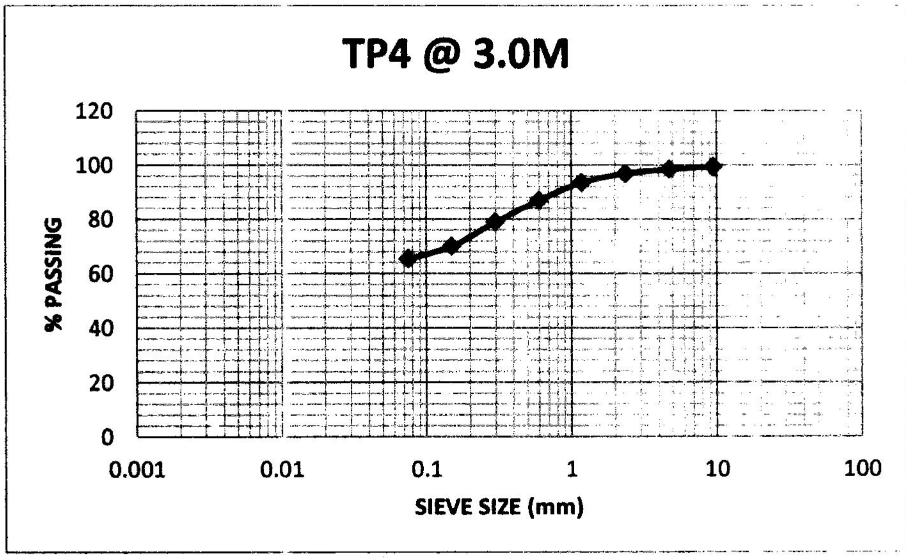



TP2 @ 3.0M
% PASSING
120
100
80
60
40
20
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)

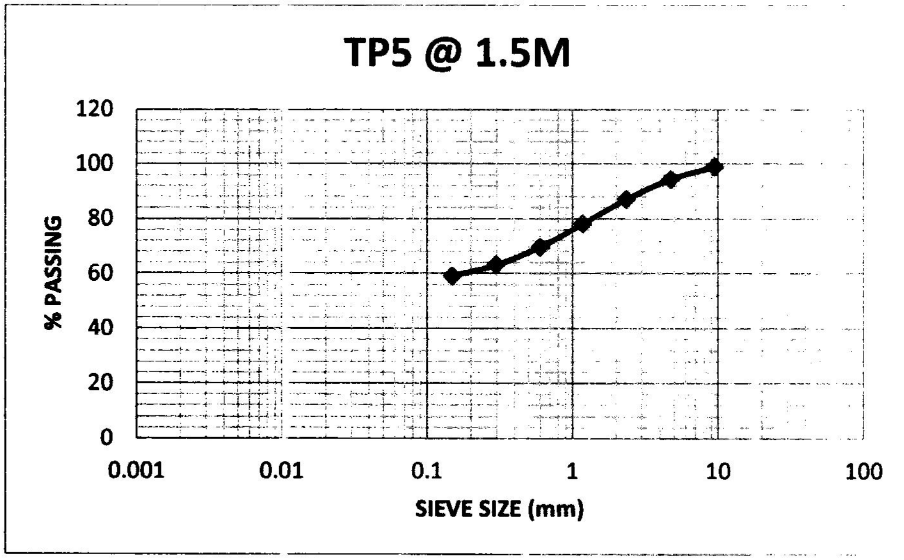

TP3 @ 3.0M
% PASSING
90
80
70
60
50
40
30
20
10
0
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)

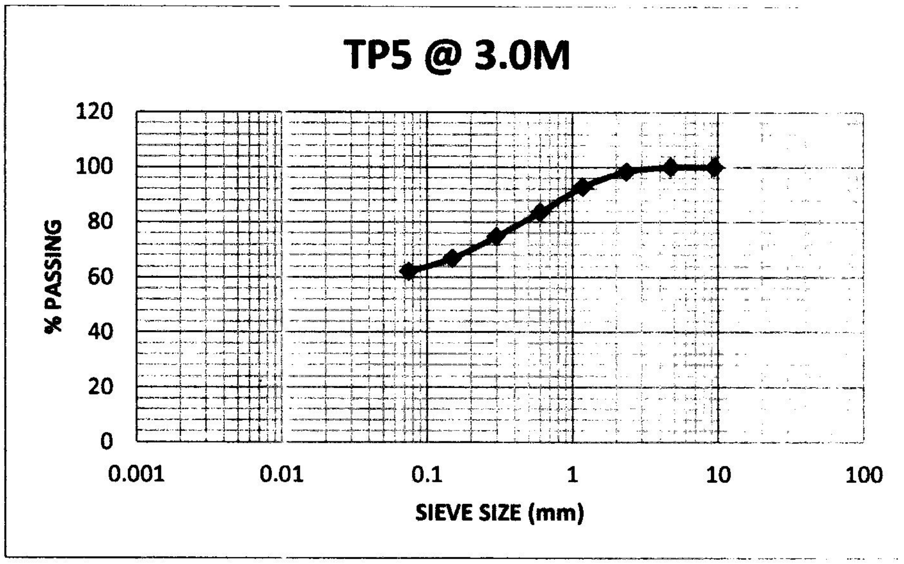


TP4 @ 1.5M
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)



TP4 @ 3.0M
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


TP5 @ 1.5M
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


TP5 @ 3.0M
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)

APPENDIX TWO

SPECIFIC GRAVITY

RESULTS OF SPECIFIC GRAVITY TEST

TP1 @ 3.0m

A. 45.0-18.8/(111.7-18.8)-(126.9-45.0) =26.2/11.0 =2.38

B. 47.1-23.9/(78.6-23.9)- (92.3-47.1)=23.2/9.5 = 2.44

Average= 2.41

TP2 @ 3.0m

A. 47.2-21.5/(72.8-21.5) –(87.4-47.2) = 25.7/11.1 = 2.32

B. 47.2-27.3/(88.7-27.3)- (102.4-51.2) = 23.9/10.0 =2.39

Average = 2.36

TP4 @ 1.5m

A. 47.0-20.8/(93.5-20.8)-(108.7-47.0) = 26.3/11 = 2.38

B.

TP4 @ 3.0m

A. 38.2-16.3/(93.6-20.9)-(106.1-47.2)=21.9/9.4 = 2.33

B. 47.2-20.9/(93.4-20.9)-(108.6-47.2) = 26.3/11.1 = 2.37

Average = 2.35

TP5 @ 1.5m

A. 41.1-15.9/(84.0-15.9)-(98.6-41.1) = 25.2/10.6 = 2.38

B 49.9-20.4/(80.8-20.4)-(97.8-49.9) = 29.5/12.3 = 2.36

Average = 2.37

TP5 @ 3.0m

A. 39.3-20.5/(72.8-20.5)-(84.0-39.5) = 18.8/7.6 = 2.47

B. 44.5-18.9/(75.5-18.9)-(90.8-44.5) = 25.7/10.4 = 2.47

Average =2.47

APPENDIX THREE

ATTERBERG LIMIT

TP 2 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
45	32	21	10		
E1	E2	E3	E4	E5	E6
19.3	16.3	22.1	10.3	10.5	9.2
42.3	42.6	53.8	42.6	25.4	27.2
35.4	33.8	42.8	30.9	21.9	23.4
6.9	8.8	11.0	11.7	4.0	4.3
16.1	17.5	20.7	20.6	11.4	14.2
42.9	50.3	53.1	56.8	35.1	30.3

Shrinkage Limit = 8.6%

TP 3 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
46	34	22	12		
A1	A2	A3	A4	A5	A6
10.4	14.9	10.7	13.0	14.2	22.3
33.0	37.2	39.2	43.8	30.1	41.2
26.3	30.1	29.9	33.2	26.6	36.8
6.7	7.1	9.3	10.6	3.5	4.4
15.9	15.2	19.2	20.2	12.4	14.5
42.1	46.7	48.4	52.3	28.2	30.3

Shrinkage Limit = 9.3%

TP 3 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
44	32	21	11		
C1	C2	C3	C4	C5	C6
10.0	9.0	13.1	12.7	9.2	14.9
31.6	35.6	43.4	48.2	24.7	32.7
25.1	27.2	33.7	36.3	21.2	28.7
6.5	8.4	9.7	11.9	3.5	4.0
15.1	18.2	20.6	23.6	12.0	13.8
43.0	46.1	47.1	50.4	29.2	29.0

Shrinkage Limit = 7.9%

TP 4 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
46	30	21	11		
J1	J2	J3	J4	J5	J6
7.2	9.1	8.8	18.6	19.6	18.6
32.7	31.9	36.5	49.9	38.8	34.1
24.7	24.0	26.5	38.2	34.2	30.3
8.5	7.7	10.0	11.7	4.6	3.8
17	14.7	17.7	19.6	14.6	11.7
				31.5	32.3

Shrinkage Limit = 10.0%

TP 4 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
47	34	22	12		
I1	I2	I3	I4	I5	I6
9.8	12.2	8.8	10.4	9.5	10.7
30.9	37.6	34.6	40.2	23.4	21.2
24.6	28.7	25.1	28.7	19.8	18.5
6.3	8.9	9.5	11.5	3.6	2.7
14.8	19.2	16.3	18.3	10.3	7.8
42.6	46.4	58.3	62.8	35.0	34.6

Shrinkage Limit = 11.4%

TP 5 (1.5 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
48	36	23	13		
D1	D2	D3	D4	D5	D6
7.3	7.4	11.4	9.1	13.7	13.0
32.3	33.5	37.5	45.6	30.3	26.2
24.5	24.8	28.5	32.6	26.5	23.2
7.0	8.7	9.0	13.0	3.8	3.0
17.2	17.4	17.1	23.5	12.8	10.2
40.7	51.0	52.6	55.3	29.7	29.4

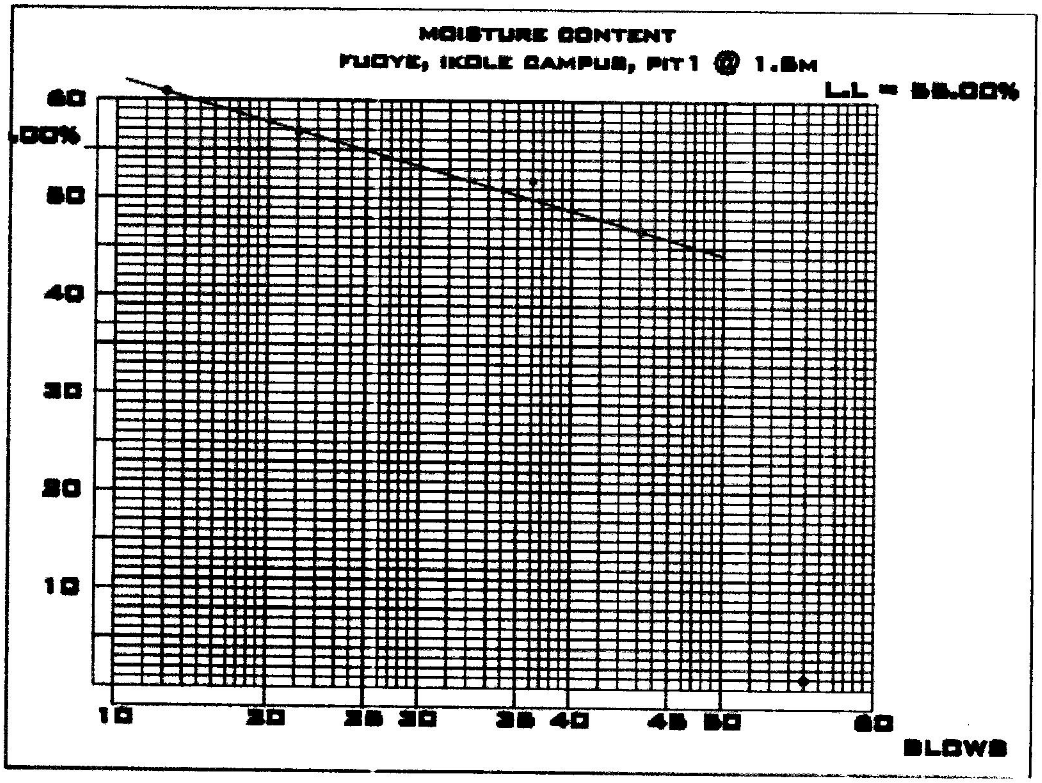
Shrinkage Limit = 9.3%

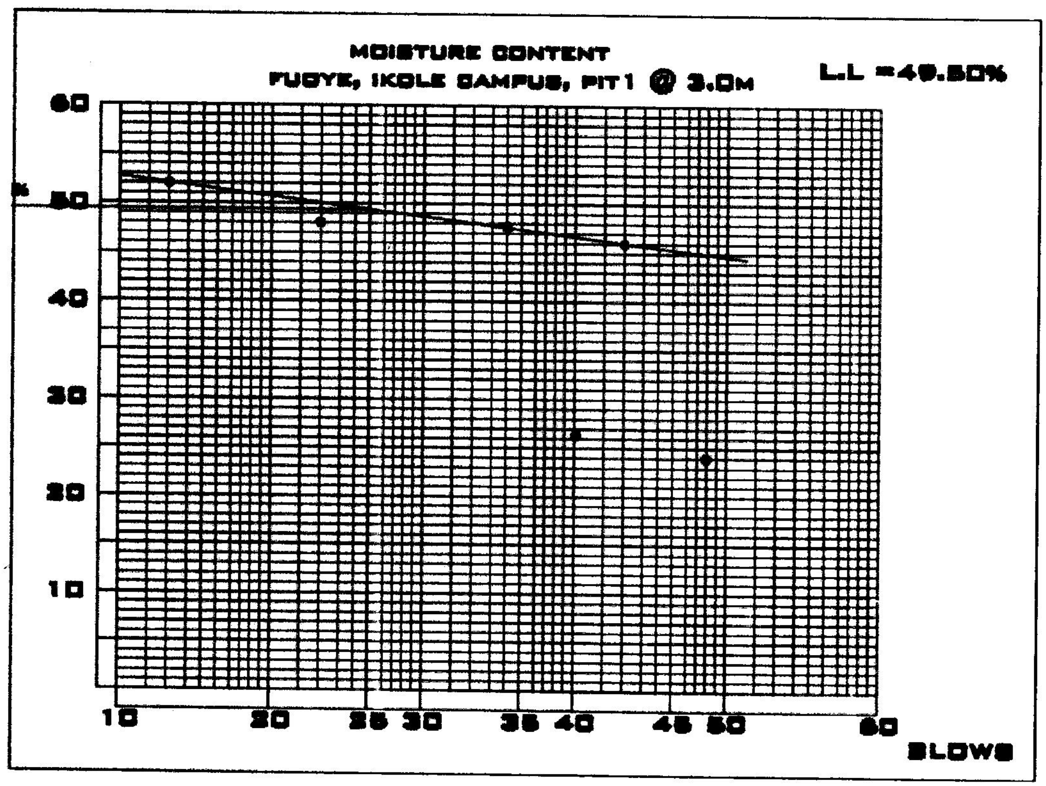
TP 5 (3.0 M)

Liquid limit				Plastic limit	
1	2	3	4	1	2
43	30	22	14		
B1	B2	B3	B4	B5	B5
18.0	11.6	9.0	9.5	10.4	8.2
5.6	32.3	35.1	39.5	24.7	21.1
5.8	7.2	9.4	11.3	3.3	3.0
11.8	13.5	16.7	18.3	11.0	9.9
49.2	53.3	56.3	61.7	30.0	30.0

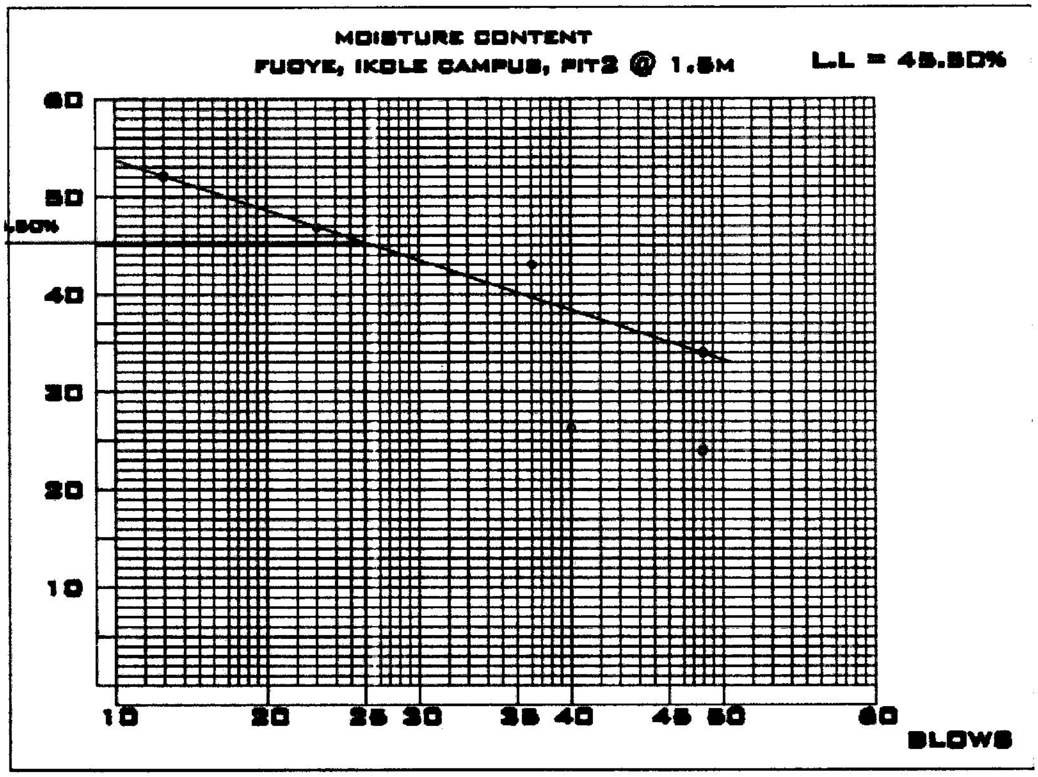
Shrinkage Limit = 10.7%

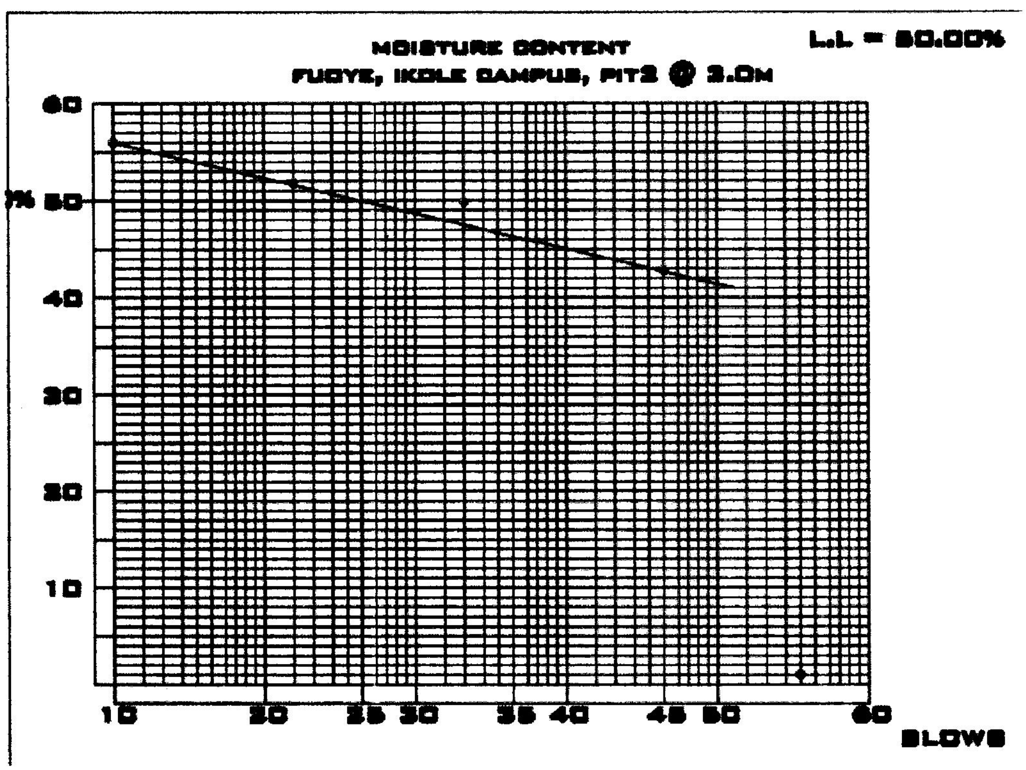
TP 1


MOISTURE CONTENT
FUOYE, IKOLE CAMPUS, PIT 1 @ 1.5M
L.L = 55.00%
60
50
40
30
20
10
10
20
25
30
35
40
45
50
60
BLOWS

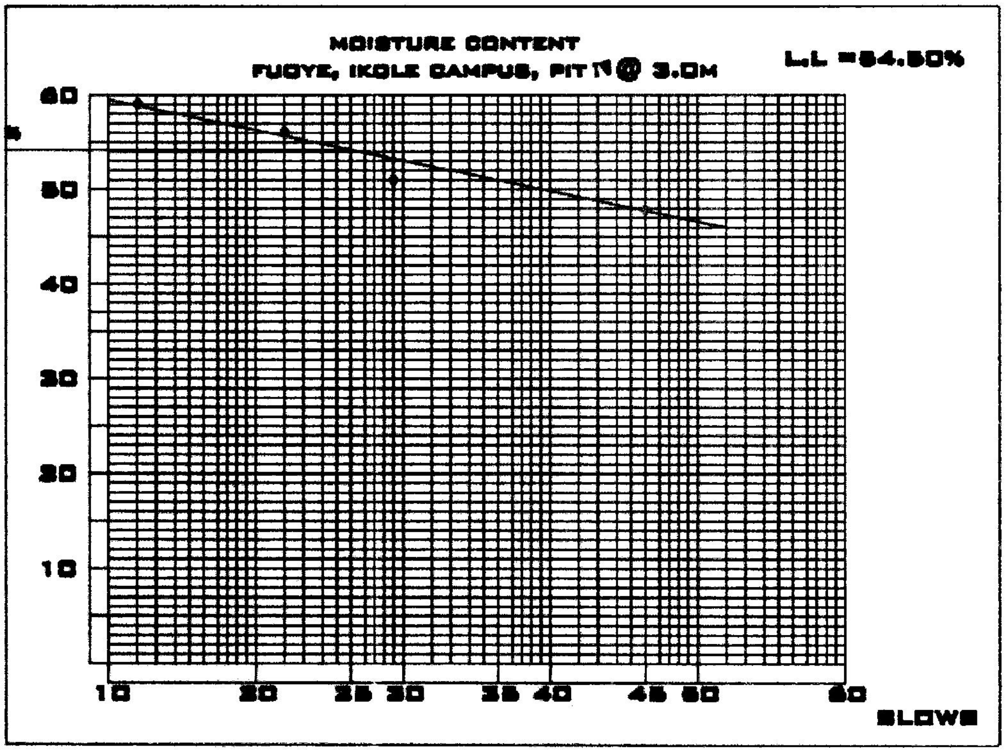

MOISTURE CONTENT
FUOYE, IKOLE CAMPUS, PIT 1 @ 3.0M
L.L =49.50%
60
50
40
30
20
10
10
20
25
30
35
40
45
50
60
BLOWS

TP 2

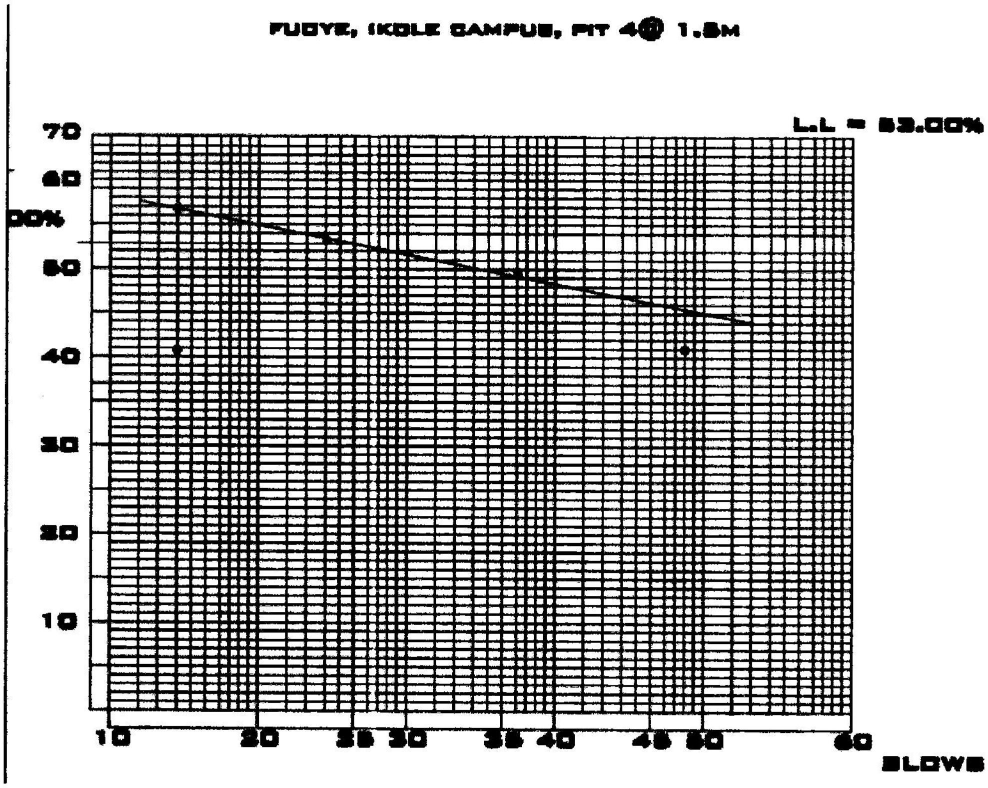

MOISTURE CONTENT
FUOYE, IKOLE CAMPUS, PIT2 @ 1.5M
L.L = 45.50%
60
50
40
30
20
10
10
20
25
30
35
40
45
50
60
BLOWS

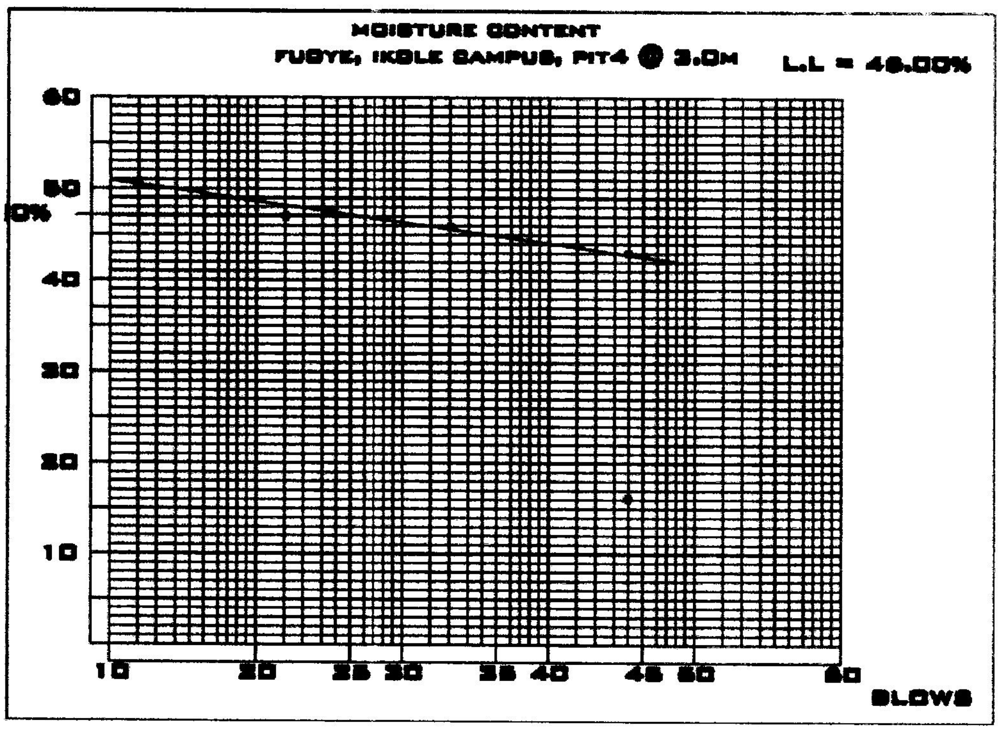

MOISTURE CONTENT
FUOYE, IKOLE CAMPUS, PIT2 @ 3.0M
L.L = 50.00%
60
50
40
30
20
10
10
20
25
30
35
40
45
50
60
BLOWS

TP 3

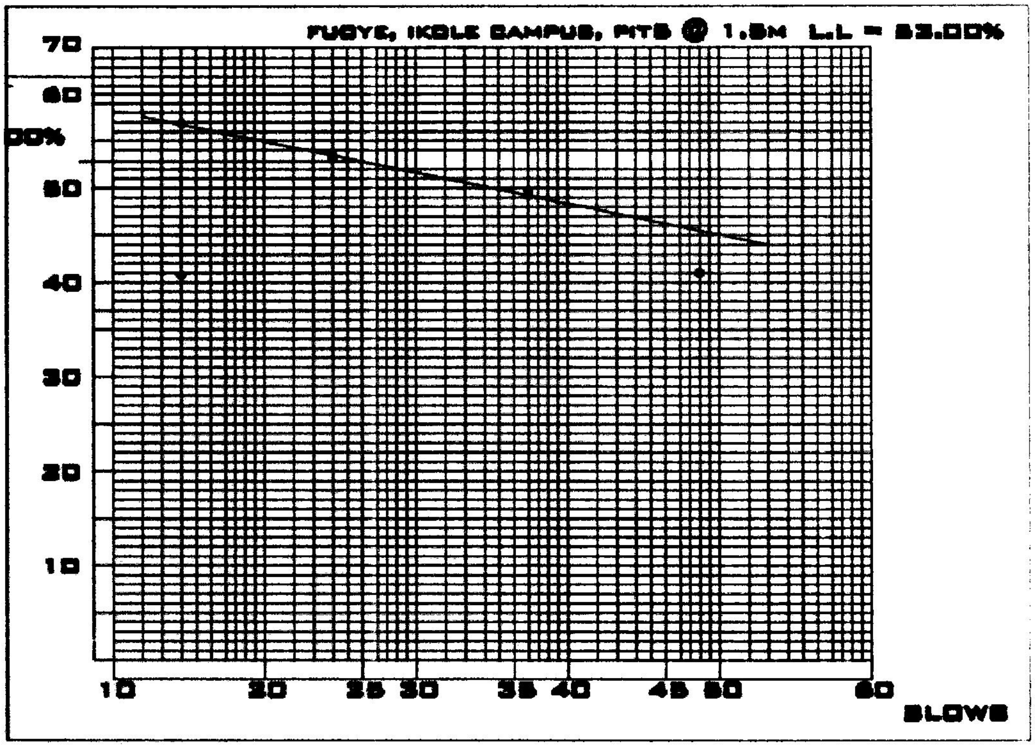

MOISTURE CONTENT
FUOYE, IKOLE CAMPUS, PIT 1 @ 3.0M
L.L =54.50%
60
50
40
30
20
10
10
20
25
30
35
40
45
50
60
BLOWS

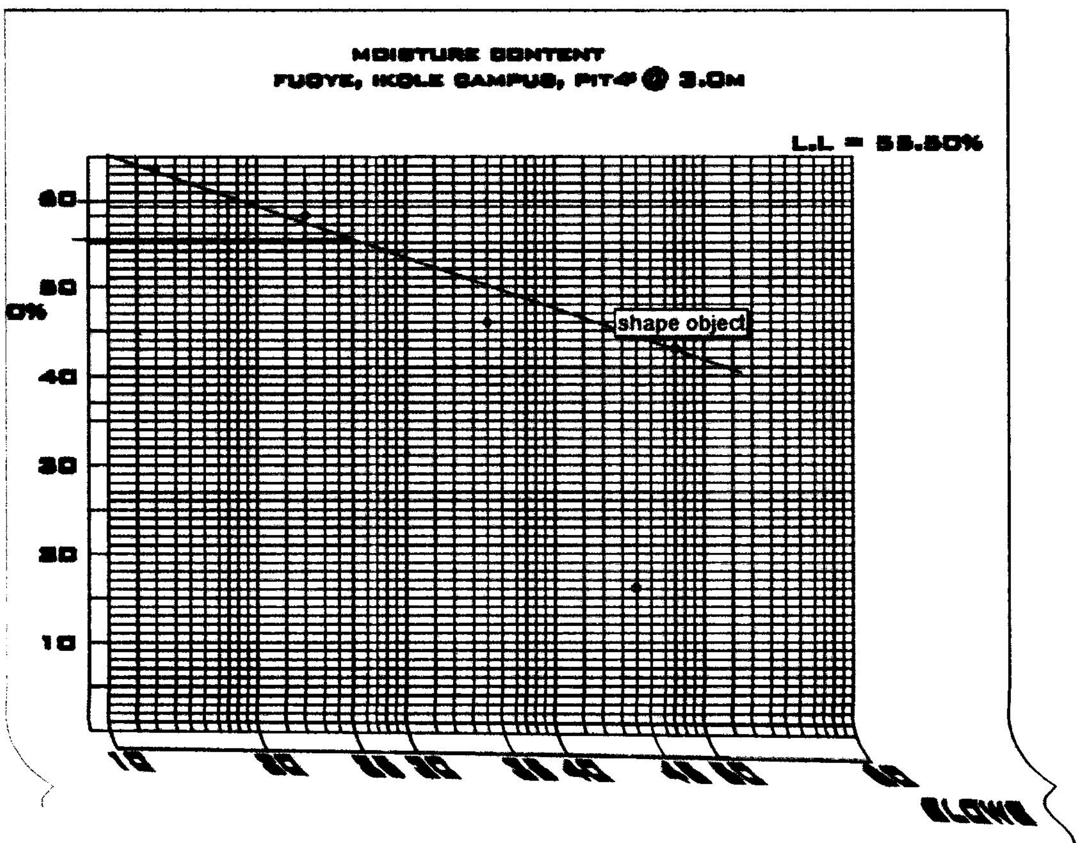
TP 4


FUOYE, IKOLE CAMPUS, PIT 4@ 1.5M
L.L = 53.00%
70
60
50
40
30
20
10
10
20
25 30
35 40
45 50
60
BLOWS


MOISTURE CONTENT
FUOYE, IKOLE CAMPUS, PIT4 @ 3.0M
L.L = 46.00%
60
50
40
30
20
10
10
20
25 30
35 40
45 50
60
BLOWS

TP 5


FUOYE, IKOLE CAMPUS, PIT5 @ 1.5M L.L = 53.00%
70
60
50
40
30
20
10
10
20
25
30
35
40
45
50
60
BLOWS


MOISTURE CONTENT
FUOYE, IKOLE CAMPUS, PIT4 @ 3.0M
60
50
40
30
20
10
shape object
BLOWS

APPENDIX FOUR

COMPACTION TEST

RESULTS OF COMPACTION TEST

TP 2 (3.0m)

Weight of mould + soil wet	5600	5800	5900	59750
Weight of empty mould	3800	3800	3800	3800
Weight of wet soil	1800	2000	2100	1950
Bulk density	1.80	2.00	2.10	1.95
Can no	AB	BC	EF	GF
Can weight	14.2	11.6	11.3	9.5
Weight of can + soil wet	48.5	47.5	53.9	50.1
Weight of can + soil dry	44.4	41.2	45.4	40.7
Weight of water	5.1	6.3	8.5	9.4
M.C	29.2	29.6	34.1	31.2
Dry density kg/m^3	17.5	21.3	25.0	30.1

TP 5 (3.0M)

Weight of mould + soil wet	5500	5650	5850	5900	5850
Weight of empty mould	3800	3800	3800	3800	3800
Weight of wet soil	1700	1800	2050	2100	2050
Bulk density	1.70	1.80	2.50	2.10	2.50
Can no	AB1	BC1	CE1	DF1	GH1
Can weight	12.8	15.2	11.4	11.2	9.8
Weight of can + soil wet	65.6	71`.0	64.8	70.6	63.7
Weight of can + soil dry	59.9	63.6	56.0	60.3	51.9
Weight of water	5.7	7.4	8.8	10.3	11.8
M.c	12.1	15.5	19.7	21.0	28.0
Dry density kg/m^3	1.52	1.60	1.75	1.74	1.60

TP 1 (3.0M)

Weight of mould + soil wet	5600	5750	5900	5800
Weight of empty mould	3800	3800	3800	3800
Weight of wet soil	1800	1950	2100	2000
Bulk density	1.80	1.95	2.10	2.00
Can no	K1	K2	K3	K4
Can weight	27.4	22.1	13.9	11.7
Weight of can + soil wet	82.7	73.0	68.2	58.4
Weight of can + soil dry	77.6	65.7	59.4	48.9
Weight of water	6.1	7.3	8.8	9.5
M.c	12.2	16.7	19.2	22.5
Dry density kg/m^3	1.6	1.67	1.76	1.59

TP 4 (3.0m)

Weight of mould + soil wet	5550	5700	5900	5800
Weight of empty mould	3800	3800	3800	3800
Weight of wet soil	1750	1900	2100	2000
Bulk density	1.75	1.90	2.10	2.00
Can no	G1	G2	K3	K4
Can weight	15.4	9.7	13.9	11.7
Weight of can + soil wet	62.8	56.8	68.2	58.4
Weight of can + soil dry	56.5	49.0	59.4	48.9
Weight of water	6.3	7.3	8.8	9.5
M.c	15.3	16.7	19.2	22.5
Dry density kg/m^3	1.07	1.59	1.66	1.52

TP 3 (1.5m)

Weight of mould + soil wet	4650	4850	5200	5150
Weight of empty mould	3100	3100	3100	3100
Weight of wet soil	1550	1700	2100	2050
Bulk density	1.55	1.70	2.10	2.05
Can no	H1	H2	H3	H4
Can weight	20.0	21.4	12.0	11.8
Weight of can + soil wet	93.7	70.4	58.2	56.6
Weight of can + soil dry	88.8	65.3	51.5	49.2
Weight of water	4.9	5.1	6.7	7.4
M.c	7.1	11.6	16.9	19.8
Dry density kg/m^3	1.45	1.52	1.79	1.71

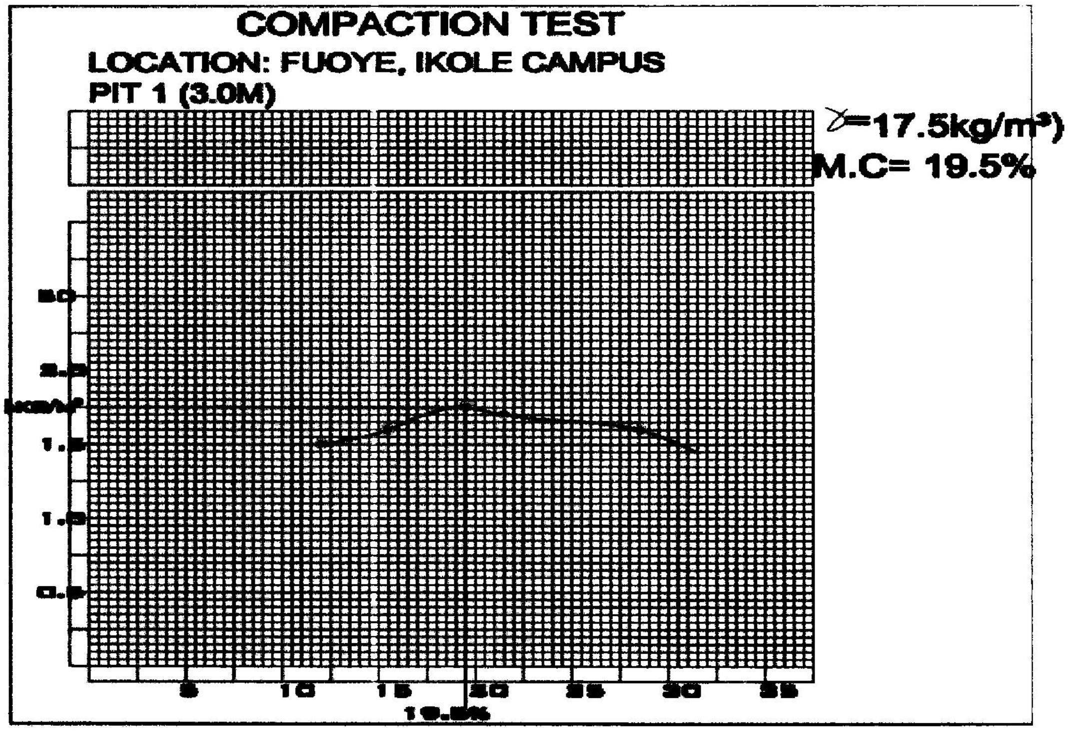
TP 3 (3.0m)

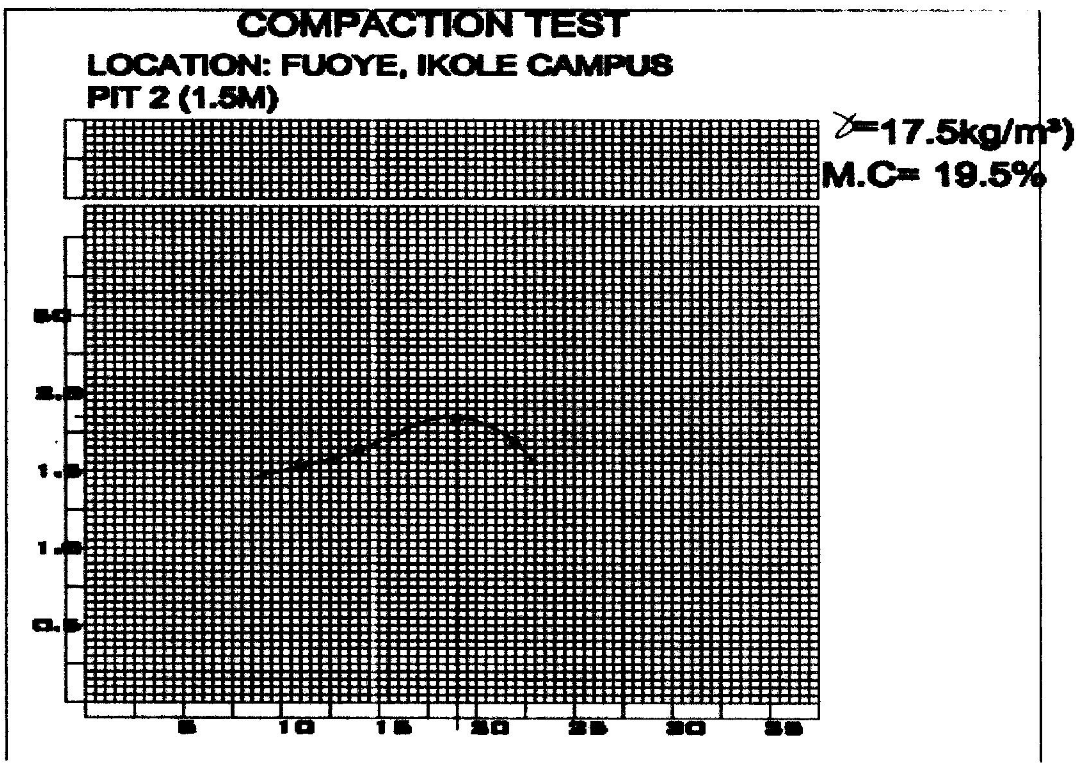
Weight of mould + soil wet	4800	5000	5250	5150
Weight of empty mould	3100	3100	3100	3100
Weight of wet soil	1700	1900	2150	2050
Bulk density	1.70	1.90	2.15	2.05
Can no	J1	J2	J3	J4
Can weight	11.2	10.3	11.7	9.5
Weight of can + soil wet	78.7	66.7	61.2	88.8
Weight of can + soil dry	71.9	59.6	52.9	74.2
Weight of water	6.8	7.1	8.3	14.6
M.C	11.2	14.4	19.9	22.6
Dry density kg/m^3	1.53	1.66	1.79	1.67

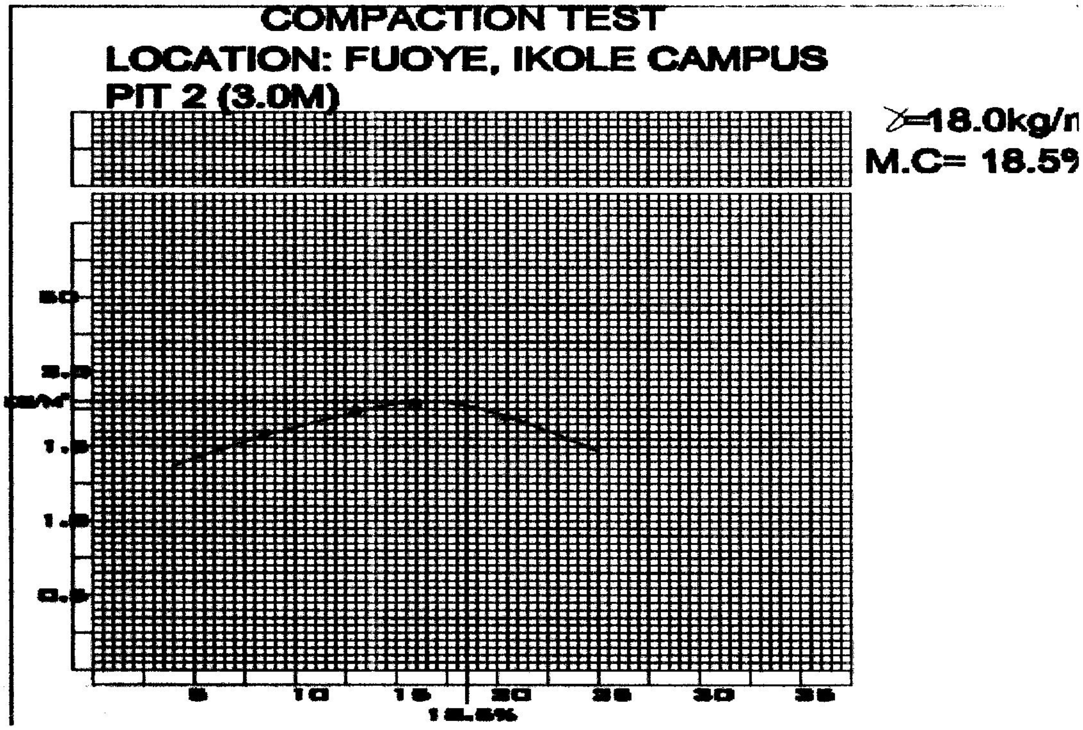
TP 5 (1.5m)

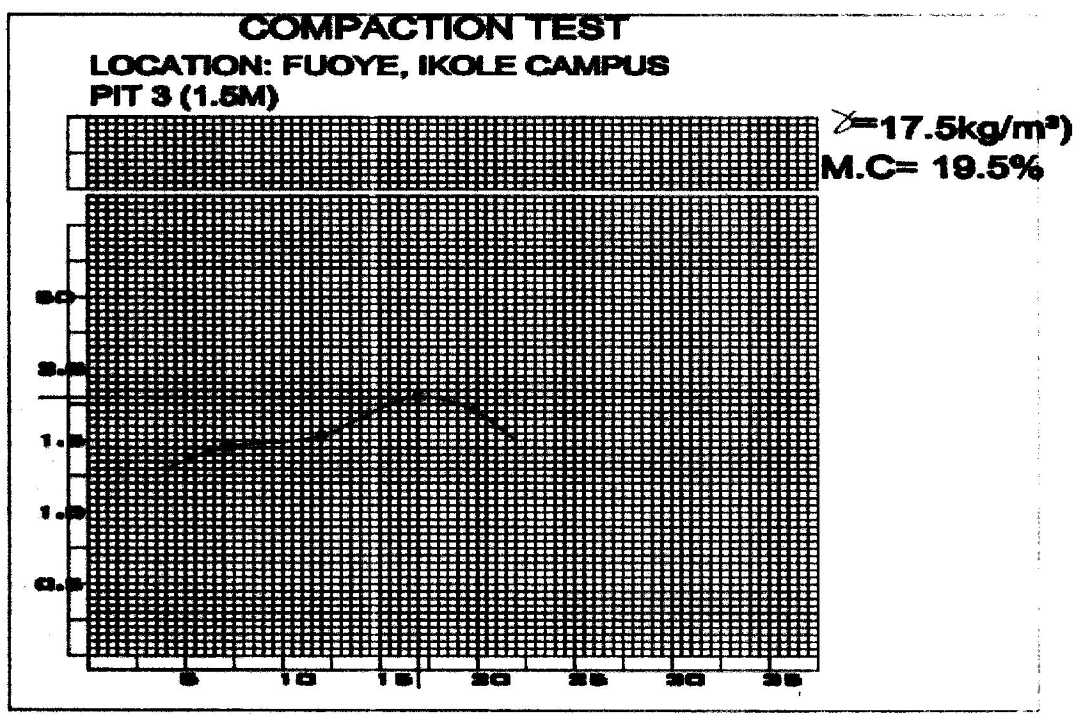
Weight of mould + soil wet	4850	5050	5200	5100
Weight of empty mould	3100	3100	3100	3100
Weight of wet soil	1750	1950	2100	2000
Bulk density	1.75	1.95	2.10	2.00
Can no	I1	I2	I3	I4
Can weight	10.2	8.2	12.1	14.3
Weight of can + soil wet	63.3	54.0	55.7	74.5
Weight of can + soil dry	58.4	48.7	48.9	63.6
Weight of water	5.2	6.0	6.8	10.9
M.c	11.3	14.8	18.5	22.1
Dry density kg/m^3	1.57	1.69	1.77	1.64

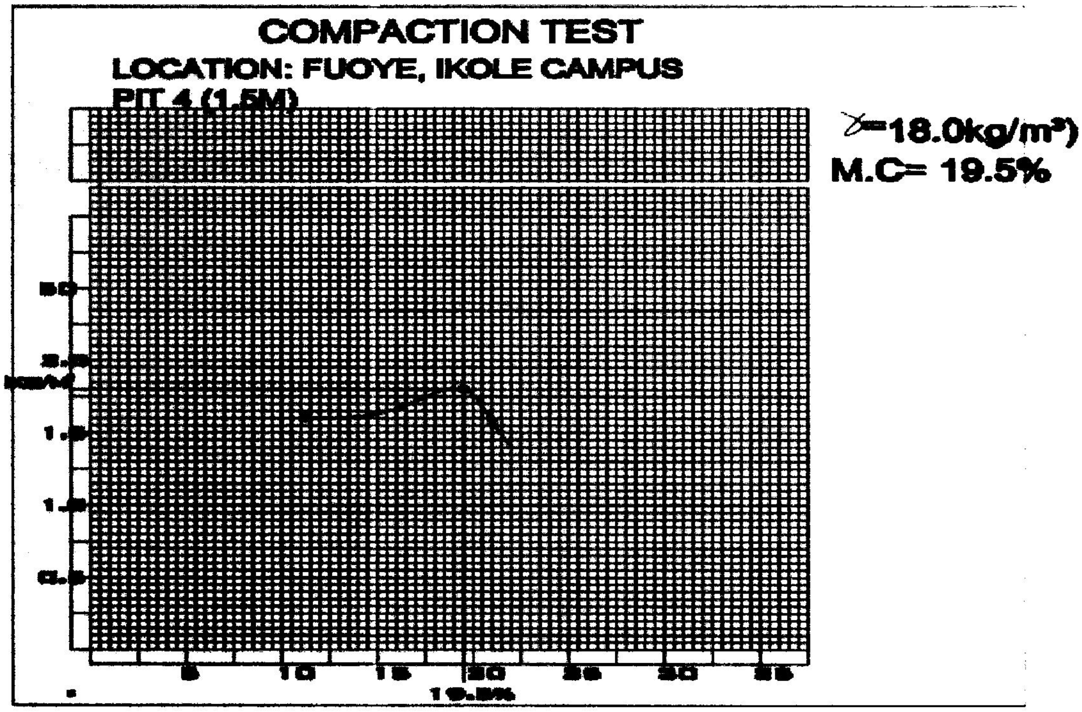
GRAPHS OF COMPACTION TEST

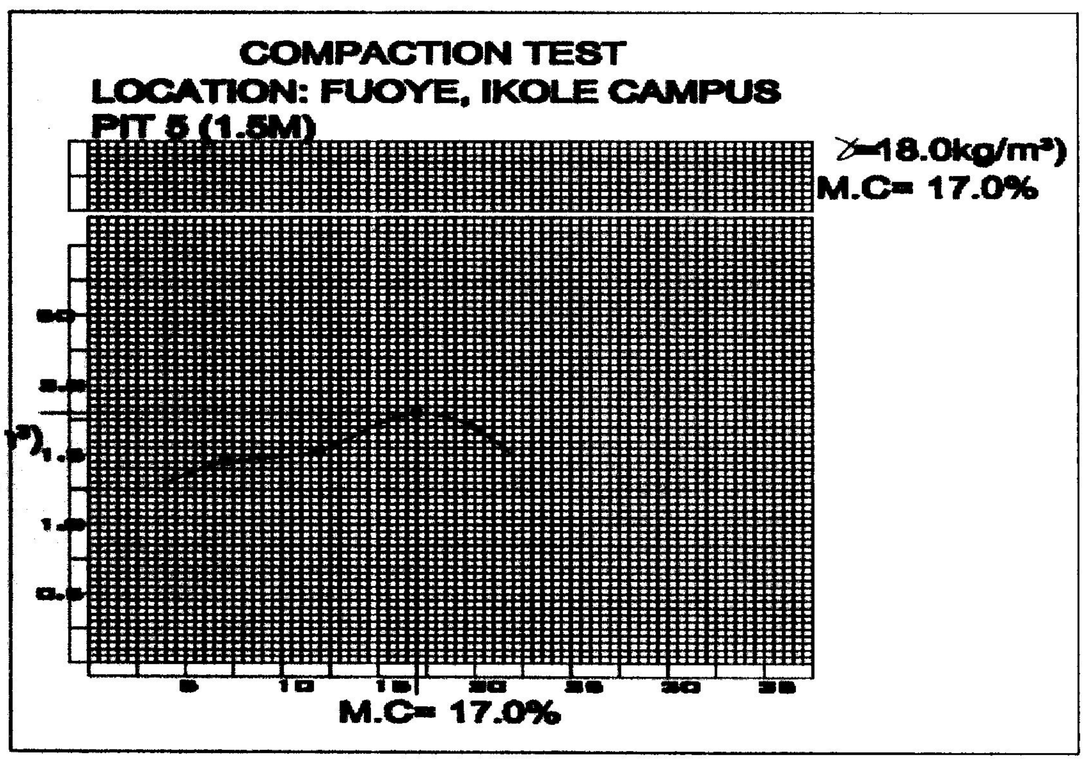


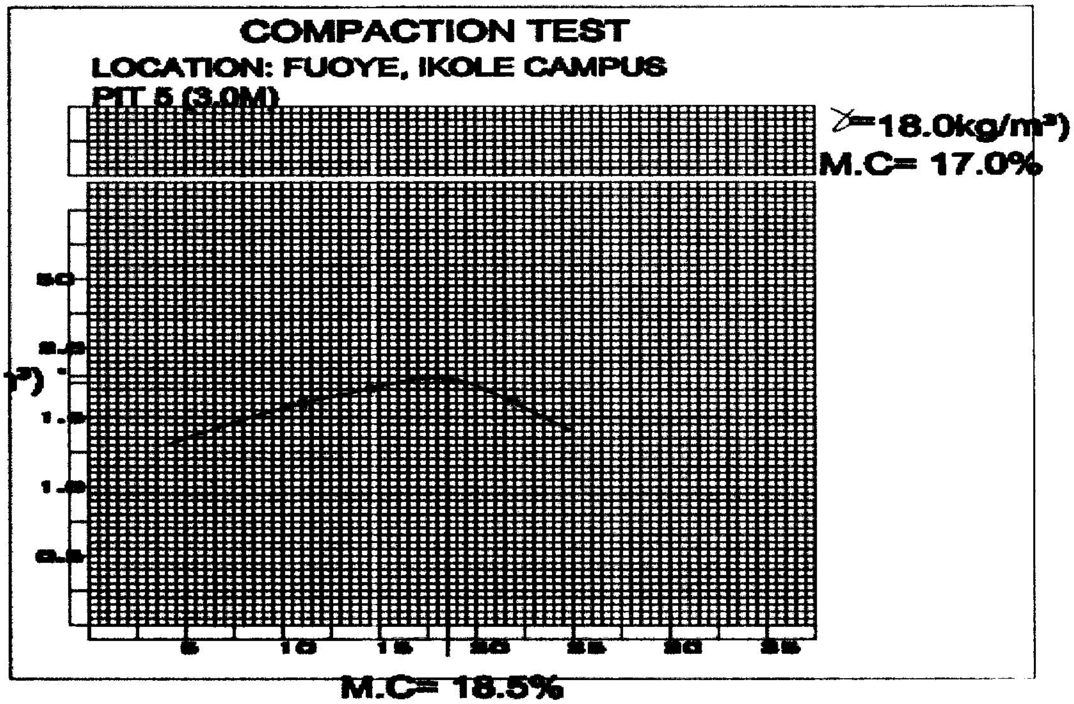



COMPACTION TEST
LOCATION: FUOYE, IKOLE CAMPUS
PIT 2 (3.0M)
M.C= 18.5
10
15
20
25
30
35


COMPACTION TEST
LOCATION: FUOYE, IKOLE CAMPUS
PIT 3 (1.5M)
M.C= 19.5%
10
15
20
25
30
35


COMPACTION TEST
LOCATION: FUOYE, IKOLE CAMPUS
PIT 4 (1.5M)
ɣ=18.0kg/m³)
M.C= 19.5%


COMPACTION TEST
LOCATION: FUOYE, IKOLE CAMPUS
PIT 5 (1.5M)
ɣ=18.0kg/m³)
M.C= 17.0%
M.C= 17.0%


COMPACTION TEST
LOCATION: FUOYE, IKOLE CAMPUS
PIT 5 (3.0M)
γ=18.0kg/m³)
M.C= 17.0%
M.C= 18.5%

APPENDIX FIVE

CALIFORNIA BEARING RATIO

RESULTS OF CALIFORNIA BEARING RATIO TEST

TP 1

	1.5m depth				3.0m depth			
	TOP		BOTTOM		TOP		BOTTOM	
S/N	DR	LOAD	DR	LOAD	DR	LOAD	DR	LOAD
50	57	1.43	73	1.83	59	1.48	15	2.68
100	82	2.05	100	2.50	1.07	2.68	191	4.78
150	107	2.68	1.74	4.35	174	4.35	251	6.28
200	165	4.13	261	6.53	210	5.25	300	7.50
250	149	6.23	302	7.55	298	7.45	364	9.10
300	320	8.00	388	9.80	358	8.95	401	10.03
350	401	10.03	4.71	11.78	436	10.90	453	11.33
400	486	12.15	586	14.65	528	13.20	501	12.53
450	563	14.00	673	16.83	601	15.03	567	14.18
500	652	16.13	705	17.63	671	16.72	694	17.35
550	697	17.43	768	19.20	739	18.48	740	18.50
600	734	18.35	815	20.38	780	19.50	291	19.78
650	789	19.73	866	21.65	932	23.30	859	21.48
700	833	20.83	902	22.55	976	2.44	892	22.3
750	896	24.0	942	23.55	009	25.23	947	23.68

TP 2

	1.5m depth				3.0m depth			
	TOP		BOTTOM		TOP		BOTTOM	
S/N	DR	LOAD	DR	LOAD	DR	LOAD	DR	LOAD
50	12	0.3	18	0.45	13	0.33	14	0.10
100	15	0.38	23	0.46	17	0.43	20	0.50
150	17	0.43	28	0.75	20	0.35	23	0.58
200	20	0.5	32	0.8	27	0.68	29	0.73
250	22	0.55	37	0.68	30	0.75	34	0.85
300	25	0.63	32	1.05	35	0.88	49	1.23
350	29	0.73	48	1.20	38	0.95	53	1.33
400	32	0.8	55	1.38	41	1.03	57	1.43
450	41	10.3	61	1.53	44	1.1	61	1.53
500	44	11.25	68	1.75	47	1.18	64	1.60
550	47	11.8	73	1.83	50	1.25	70	1.75
600	55	12.78	80	2.00	42	1.35	84	2.10
650	55	13.38	93	2.33	44	1.40	89	2.23
700	61	15.3	100	2.5	48	1.20	93	2.33
750	67	16.7	104	2.6	410	10.25		

TP 3

	1.5 m depth				3.0 m depth			
	TOP		BOTTOM		TOP		BOTTOM	
S/N	DR	LOAD	DR	LOAD	DR	LOAD	DR	LOAD
50	30	0.75	74	1.85	65	1.63	25	0.63
100	65	1.63	102	2.55	110	2.75	46	1.15
150	102	2.55	149	3.73	179	4.48	74	1.85
200	143	3.58	200	5.0	205	5.13	100	2.5
250	205	5.13	250	6.25	261	6.53	148	3.7
300	269	6.73	351	8.78	338	8.45	220	5.5
350	377	9.43	436	10.9	418	10.45	331	8.28
400	438	11.0	510	12.75	505	12.63	417	10.43
450	570	13.0	618	15.45	566	14.15	525	13.13
500	695	17.38	715	17.88	642	16.05	653	16.33
550	756	18.9	800	20.0	691	17.28	699	17.48
600	797	19.93	884	22.1	738	18.45	761	19.03
650	836	20.9	965	24.13	784	19.6	794	19.85
700	859	21.48	1007	25.18	820	20.5	842	21.05
750	881	22.03	1036	25.9	861	21.53	891	22.28

TP 4

	1.5m depth				3.0m depth			
	TOP		BOTTOM		TOP		BOTTOM	
S/N	DR	LOAD	DR	LOAD	DR	LOAD	DR	LOAD
50	2	0.05	10	0.25	1	10.03	4	0.10
100	5	0.13	15	0.38	3	10.08	17	0.43
150	8	0.20	22	0.55	6	10.15	24	0.60
200	12	0.30	26	0.66	10	0.25	30	0.75
250	17	0.43	31	0.78	14	0.35	36	0.90
300	21	0.53	40	1.00	20	0.50	43	1.08
350	27	0.68	49	1.23	27	0.68	49	1.23
400	30	0.75	59	1.48	29	0.73	25	0.63
450	37	0.93	65	1.63	32	0.80	29	0.98
500	49	1.23	72	1.80	39	0.98	35	0.88
550	52	1.30	79	1.98	46	1.15	46	1.15
600	57	1.43	86	2.15	49	1.23	49	1.23
650	50	1.25	95	2.38	57	1.48	53	1.33
700	65	1.63	107	2.68	63	1.58	58	1.45
750	69	1.73	110	2.75	66	1.65	63	1.58

TP 5

	1.5m depth				3.0m depth			
	TOP		BOTTOM		TOP		BOTTOM	
S/N	DR	LOAD	DR	LOAD	DR	LOAD	DR	LOAD
50	20	5.00	43	1.08	61	1.53	98	2.45
100	49	1.18	81	2.03	100	2.50	128	3.20
150	82	2.05	102	2.55	139	3.4	177	4.43
200	112	2.80	122	3.05	189	4.73	209	5.23
250	138	3.45	179	4.42	210	5.25	234	5.85
300	179	44.7	212	5.30	242	6.05	275	6.88
350	201	5.03	281	7.03	290	7.25	310	7.75
400	234	5.83	318	7.03	290	7.25	310	7.75
450	282	7.05	370	9.25	385	9.63	404	11.08
500	315	7.88	401	10.03	418	10.45	443	11.08
550	338	8.45	415	10.38	455	11.38	486	12.15
600	371	9.28	443	11.08	491	12.27	528	13.2
650	402	10.05	492	12.30	525	13.13	581	14.5
700	408	10.20	509	12.73	551	13.78	602	15.1
750	411	10.27	527	13.18	593	14.83	626	15.65

CBR VALUES

PIT 1

Depth 1.5m

	Top	**Bottom**
2.5	6.23 = 45.4	7.55 = 54.8
5.0	16.13 = 81.9	17.63 = 88.7

Depth 3.0m

	Top	**Bottom**
2.5	7.45 = 54.0	9.10 = 59.8
5.0	16.78 = 82.7	17.35 = 87.8

PIT 2

Depth 1.5m

	Top	**Bottom**
2.5	0.30 = 2.3	0.68 = 5.1
5.0	0.85 = 4.3	1.45 = 7.4

Depth 3.0m

	Top	**Bottom**
2.5	0.53 = 4.0	0.60 = 4.5
5.0	0.93 = 4.7	1.10 = 5.6

PIT 3

Depth 1.5m

	Top	**Bottom**
2.5	6.23 = 47.1	7.45 = 56.3
5.0	16.13 = 81.9	17.63 = 89.5

Depth 3.0m

	Top	Bottom
2.5	7.45 = 56.3	9.10 = 68.7
5.0	16.74 = 85.0	17.35 = 88.1

PIT 4

Depth 1.5m

	Top	Bottom
2.5	0.43 = 3.3	0.78 = 5.9
5.0	1.28 = 6.5	1.80 = 9.1

Depth 3.0m

	Top	Bottom
2.5	0.35 = 2.6	0.90 = 6.8
5.0	0.98 = 5.0	0.88 = 4.5

PIT 5

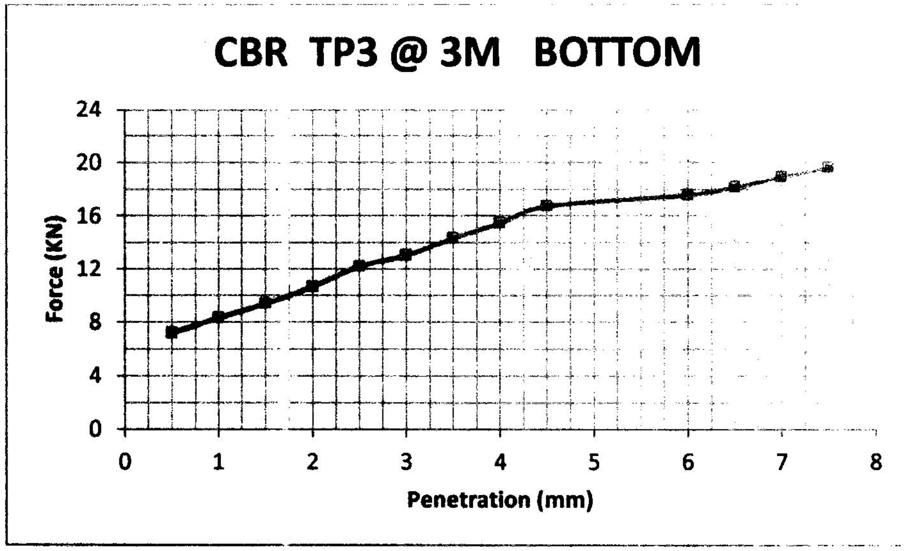
Depth 1.5m

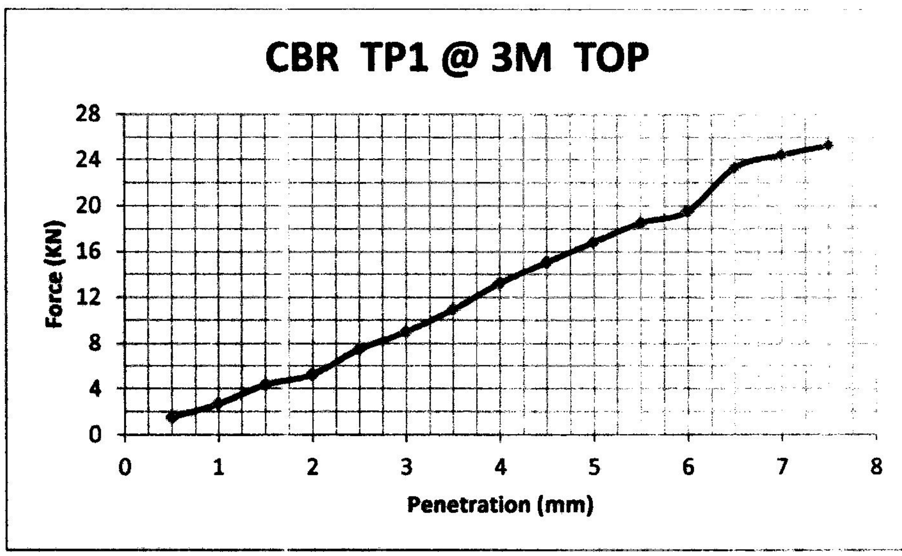
	Top	Bottom
2.5	3.43 = 25.9	4.48 = 33.8
5.0	7.88 = 40.0	10.03 = 50.9

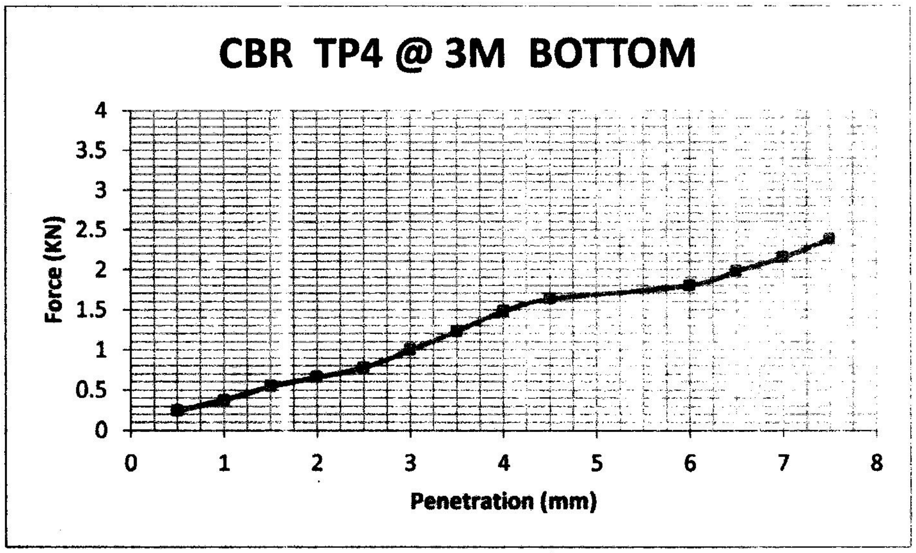
Depth 3.0m

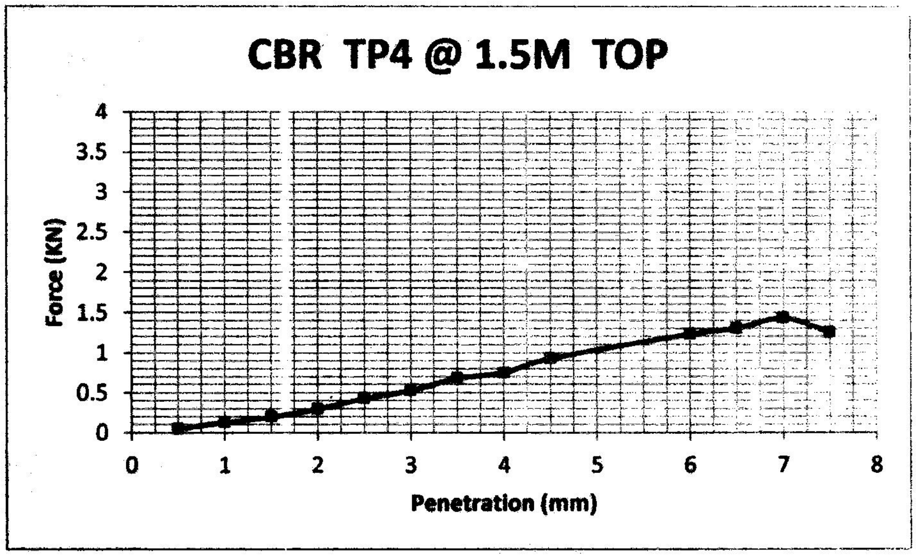
	Top	Bottom
2.5	5.25 = 39.7	5.85 = 74.2
5.0	10.45 = 53.1	11.08 = 56.3

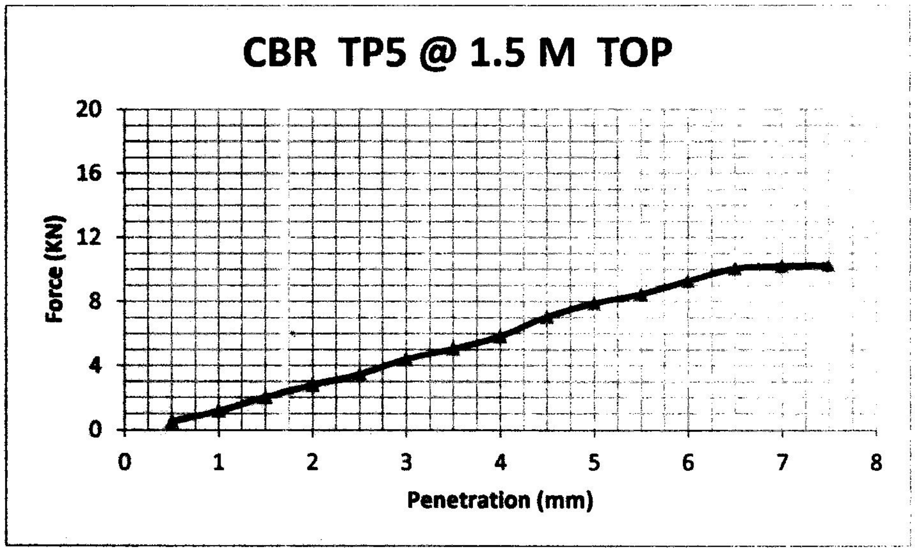
GRAPHS OF CBR

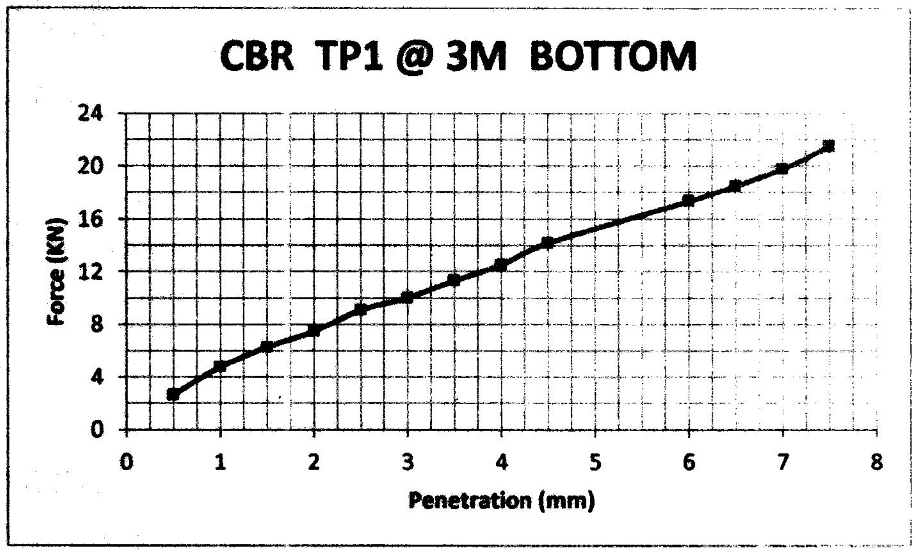


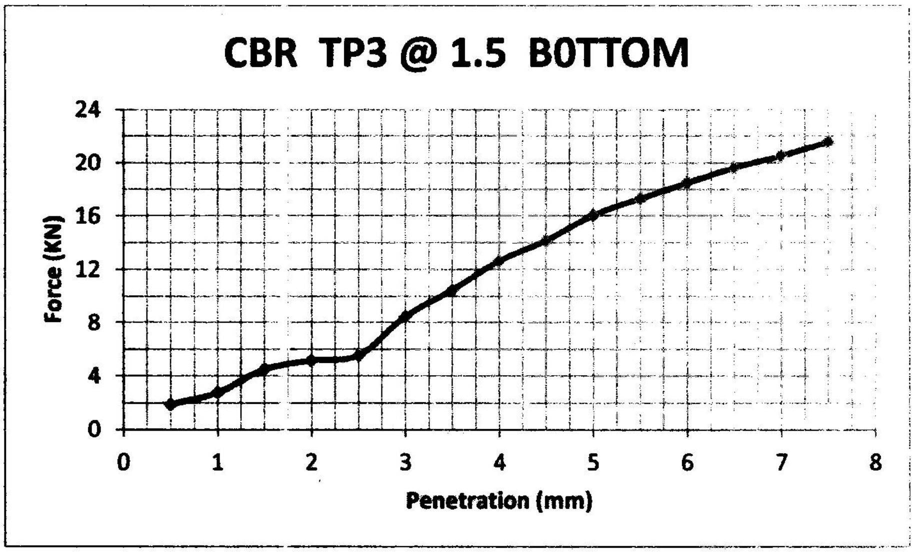


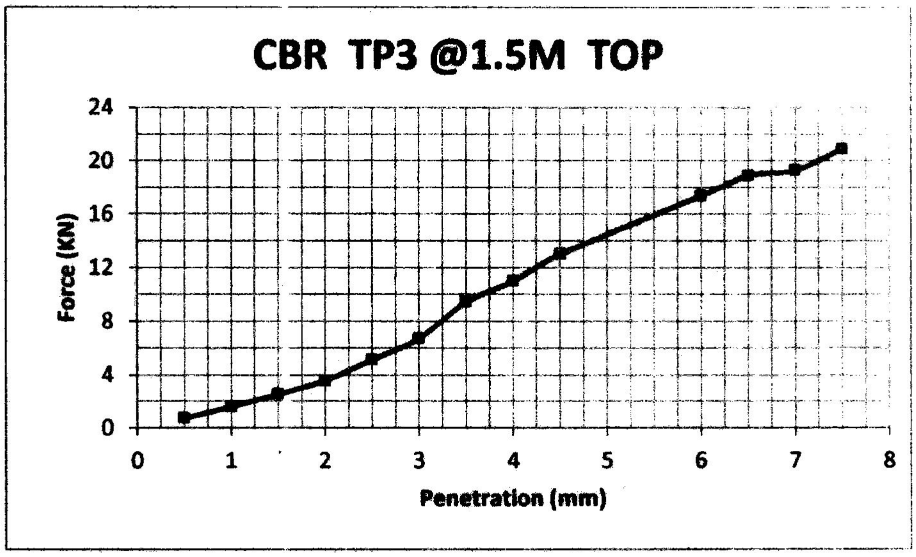

CBR TP4 @ 3M BOTTOM
Force (KN)
Penetration (mm)
0
0.5
1
1.5
2
2.5
3
3.5
4
0 1 2 3 4 5 6 7 8

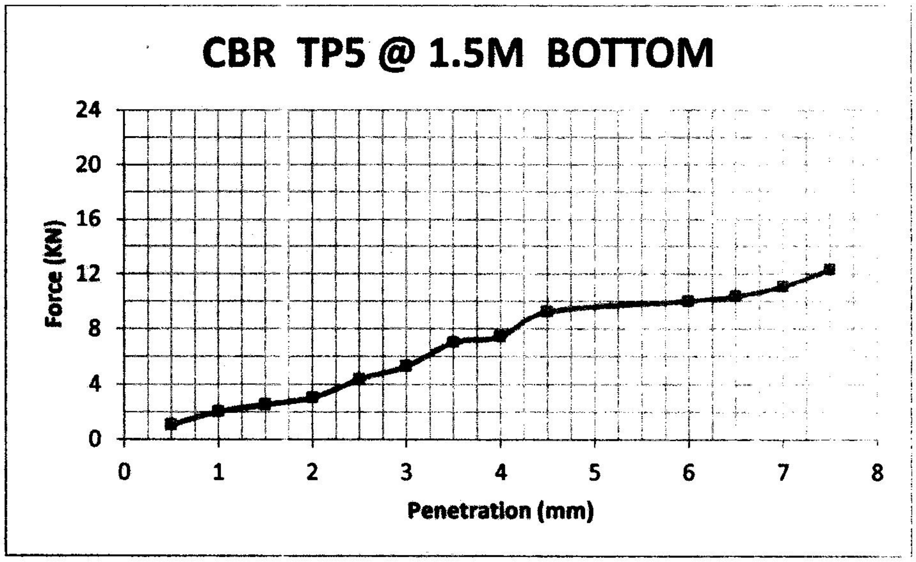

CBR TP4 @ 1.5M TOP
Force (KN)
Penetration (mm)
0
0.5
1
1.5
2
2.5
3
3.5
4
0 1 2 3 4 5 6 7 8

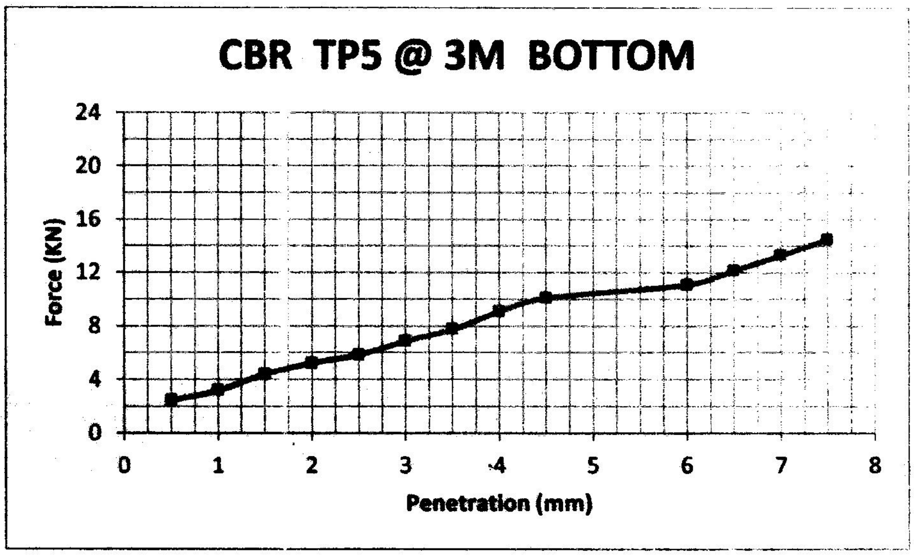

CBR TP5 @ 1.5 M TOP
Force (KN)
20
16
12
8
4
0
0
1
2
3
4
5
6
7
8
Penetration (mm)

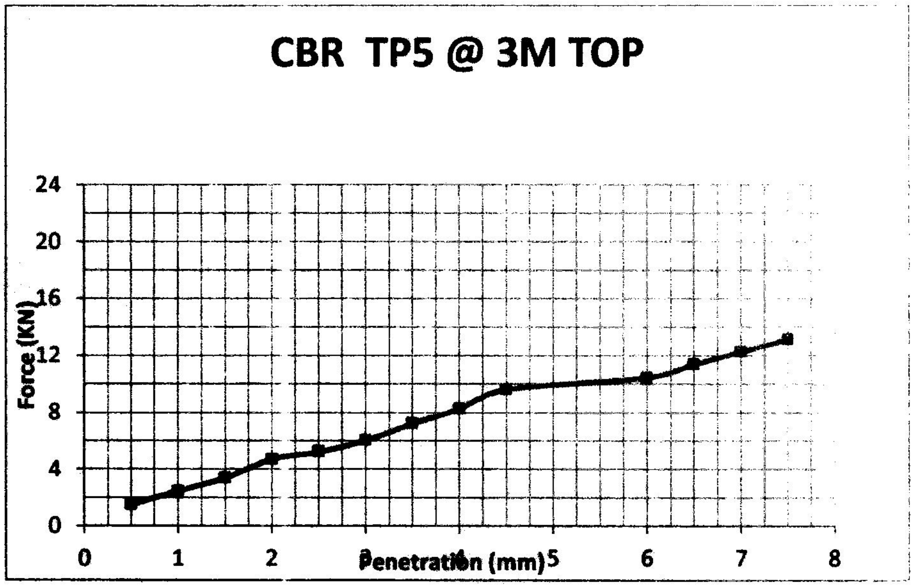

CBR TP1 @ 3M BOTTOM
Force (KN)
24
20
16
12
8
4
0
0
1
2
3
4
5
6
7
8
Penetration (mm)


CBR TP3 @ 1.5 B0TTOM
Force (KN)
0
4
8
12
16
20
24
Penetration (mm)
0
1
2
3
4
5
6
7
8


CBR TP3 @1.5M TOP
Force (KN)
0
4
8
12
16
20
24
Penetration (mm)
0
1
2
3
4
5
6
7
8


CBR TP5 @ 1.5M BOTTOM
24
20
16
12
8
4
0
Force (KN)
0
1
2
3
4
5
6
7
8
Penetration (mm)


CBR TP5 @ 3M BOTTOM
24
20
16
12
8
4
0
Force (KN)
0
1
2
3
4
5
6
7
8
Penetration (mm)


CBR TP5 @ 3M TOP
Force (KN)
0
4
8
12
16
20
24
Penetration (mm)
0
1
2
3
4
5
6
7
8

APPENDIX SIX

TRIAXIAL TEST

TRIAXIAL RESULT AND GRAPH

TP 1 (1.5m Depth)

Force	Change in length (Lo)	Strain ΔL/Lo X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	2.50	361.84	11.739	405	0.81	690
200	4.00	526.32	11.971	455	0.91	760
300	5.50	723.68	12.226	562	1.12	919

TP 1 (3.0m Depth)

Force	Change in length (Lo)	Strain ΔL/Lo X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	5.00	657.90	12.140	240	0.48	395
200	3.00	394.74	11.807	330	0.66	559
300	4.50	592.11	12.055	390	0.78	647

TP 2 (1.5m Depth)

Force	Change in length (Lo)	Strain ΔL/Lo X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	1.50	197.57	11.573	290	0.58	501
200	3.00	394.74	11.807	420	0.84	711

TP 2 (3.0m Depth)

Force	Change in length (Lo)	Strain ΔL/Lo X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	4.50	592.11	12.055	160	0.32	265
200	3.50	460.53	11.561	302	0.61	522

TP 3 (1.5m Depth)

Force	Change in length (Lo)	Strain ΔL/Lo X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	3.00	394.74	11.807	285	0.57	483
200	2.00	263.16	11.655	415	0.83	712
300	4.00	526.23	11.971	598	1.20	999

TP 3 (3.0m Depth)

Force	Change in length (Lo)	Strain ΔL/Lo X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	2.50	361.84	11.739	315	0.63	537
200	3.00	394.74	11.807	610	1.22	1033

TP 4 (1.5m Depth)

Force	Change in length (Lo)	Strain $\Delta L/Lo$ X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	4.00	526.32	11.971	350	0.70	585
200	3.50	460.53	11.561	540	1.08	934
300	4.50	592.11	12.055	740	1.48	1228

TP 4 (3.0m Depth)

Force	Change in length (Lo)	Strain $\Delta L/Lo$ X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	2.00	263.16	11.655	272	0.54	467
200	3.50	460.53	11.561	458	0.92	792
300	7.00	921.05	12.492	674	1.35	1079

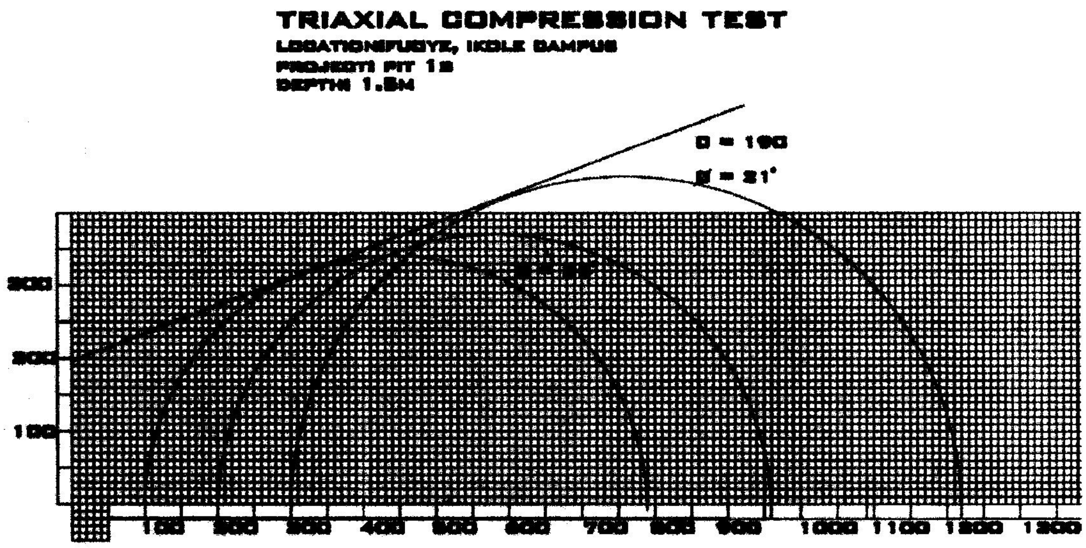
TP 5 (1.5m Depth)

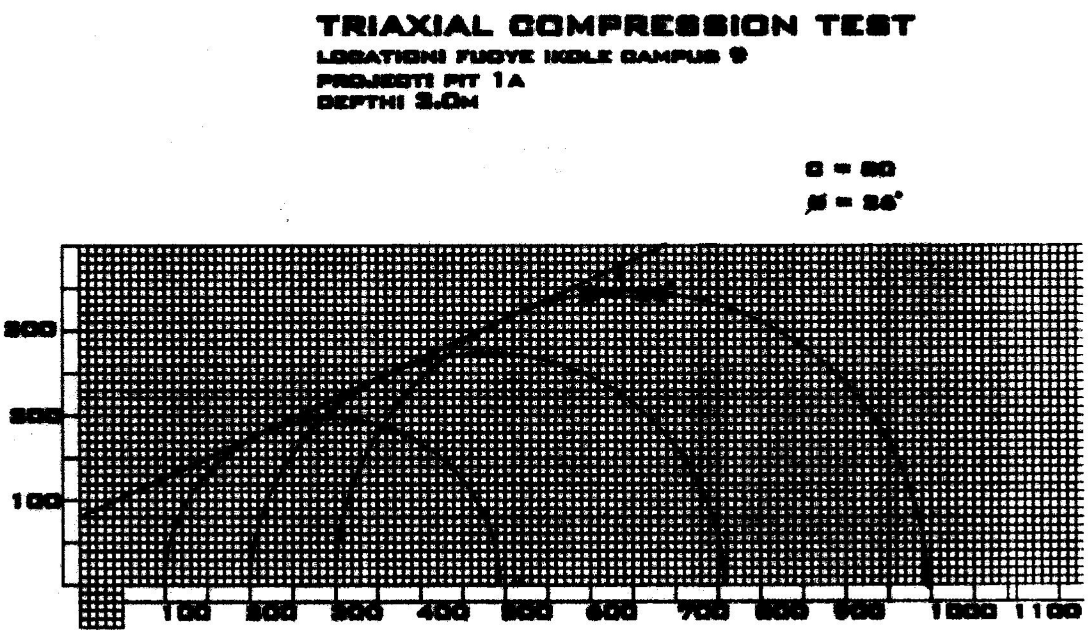
Force	Change in length (Lo)	Strain $\Delta L/Lo$ X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	3.50	460.53	11.561	250	0.50	429
200	7.00	921.05	12.492	421	0.84	674
300	5.50	723.68	12.226	530	1.06	867

TP 5 (3.0m Depth)

Force	Change in length (Lo)	Strain ΔL/Lo X 10^{-4}	Corresponding Area, Ac X 10^{-3} m^2	Load Dial Reading, LRD (Div)	Deviator load, Lo = LRD x DRF (kN)	Deviator Stress, D = LRD x Ac (kN/m^2)
100	4.50	592.11	12.055	270	0.54	448
200	**5.50**	**723.68**	**12.226**	**425**	**0.85**	**695**
300	6.50	835.26	12.402	592	1.18	955

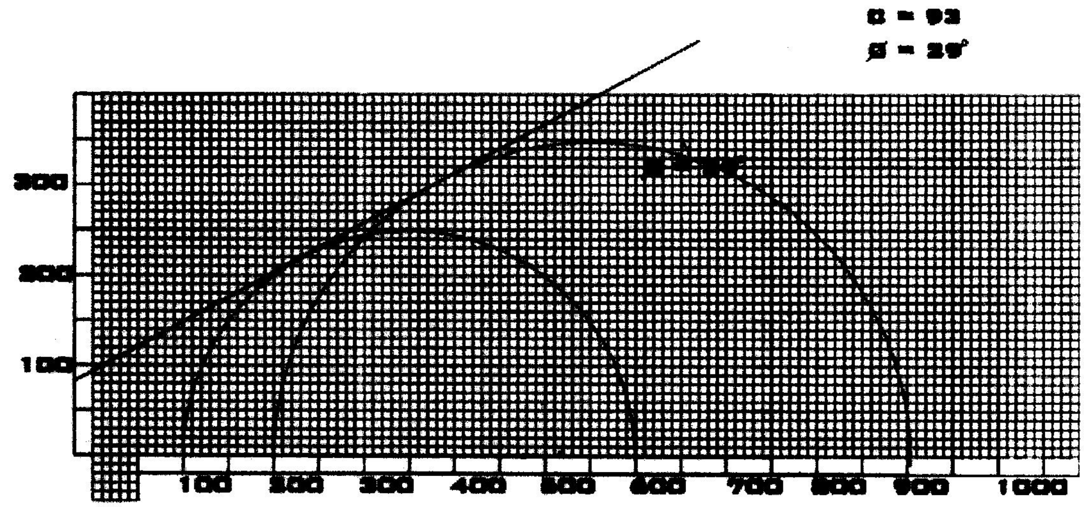
GRAPHS OF THE TRIAXIAL RESULTS





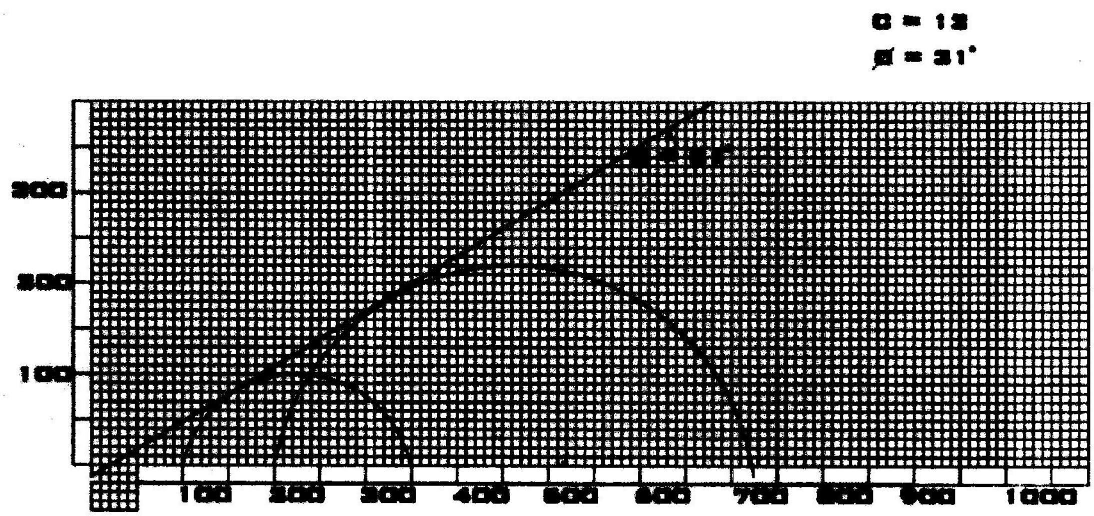
TRIAXIAL COMPRESSION TEST

LOCATION: FUOYE IKOLE CAMPUS 3
PROJECT: PIT 2B
DEPTH: 1.5M

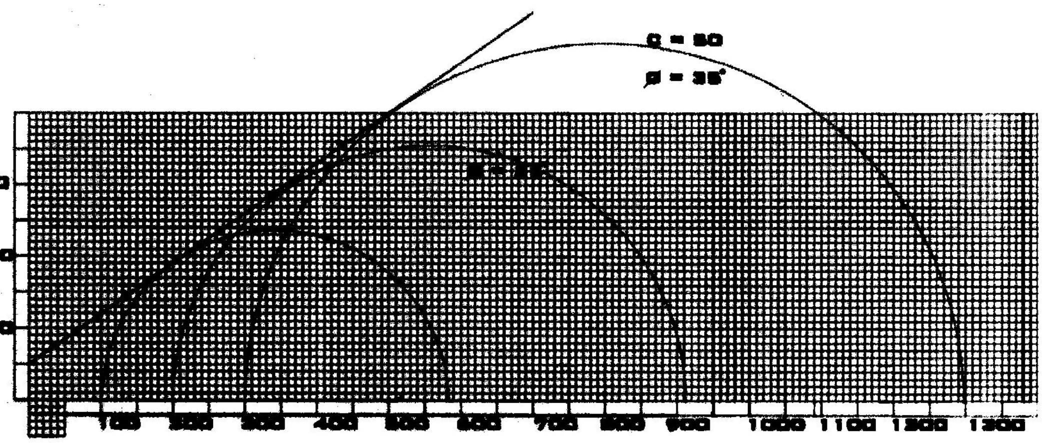


TRIAXIAL COMPRESSION TEST

LOCATION: FUOYE IKOLE CAMPUS
PROJECT: PIT 2A
DEPTH: 3.0M



TRIAXIAL COMPRESSION TEST
LOCATION: FUOYE, IKOLE CAMPUS
PROJECT: PIT 3B
DEPTH: 1.5M

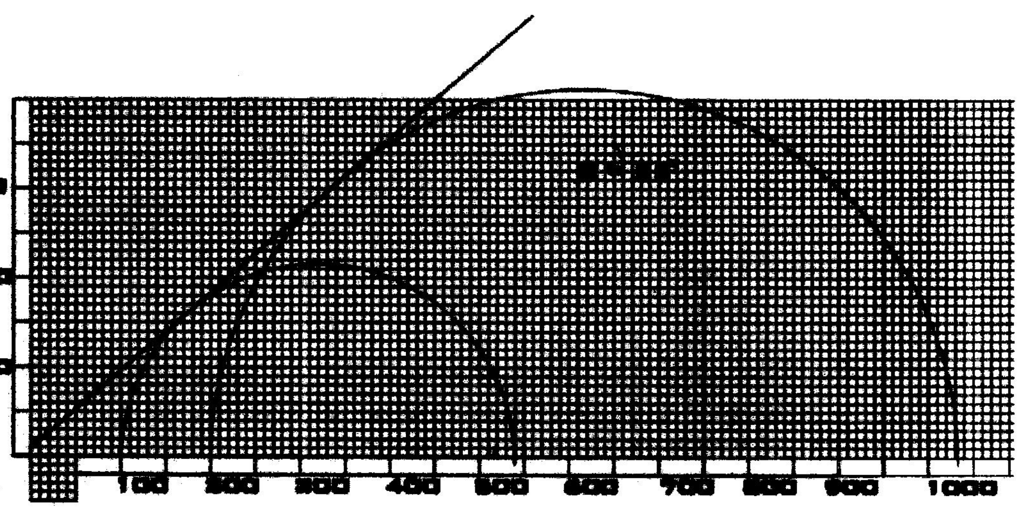


TRIAXIAL COMPRESSION TEST
LOCATION: FUOYE IKOLE CAMPUS

PROJECT: PIT 3A

DEPTH: 3.0M

Ø = 41°
C= 10



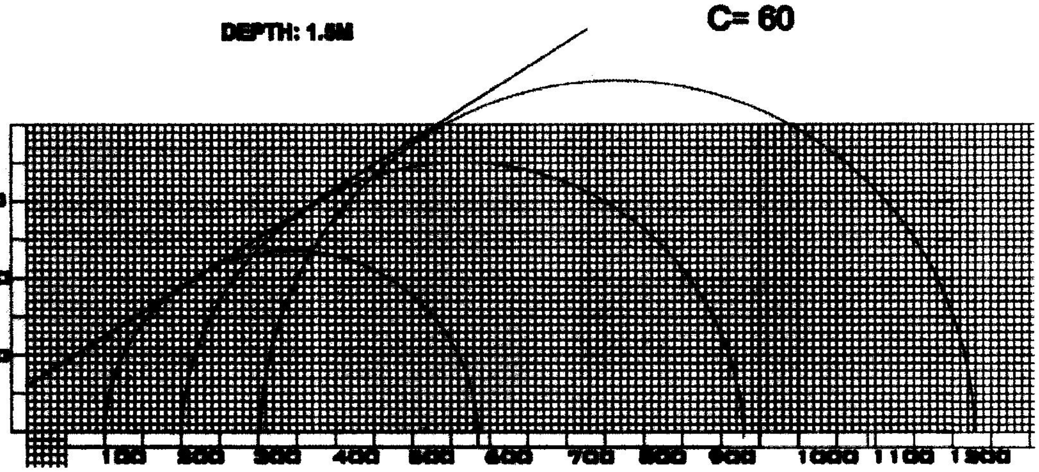
TRIAXIAL COMPRESSION TEST

LOCATION: FUOYE IKOLE CAMPUS

PROJECT: PIT 4

DEPTH: 1.8M

Ø = 34°
C= 60



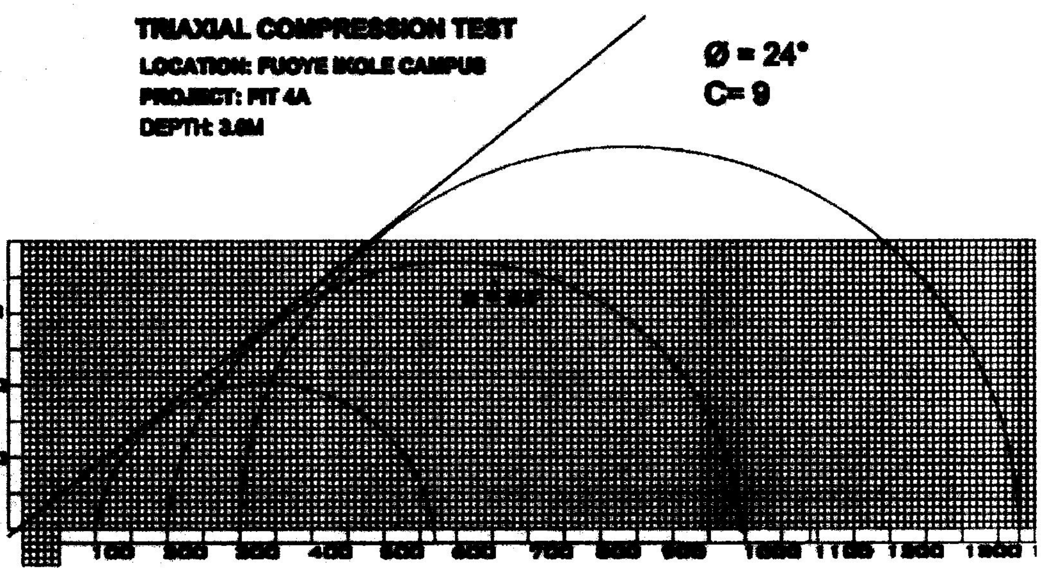
TRIAXIAL COMPRESSION TEST

LOCATION: FUOYE IKOLE CAMPUS

PROJECT: PIT 4A

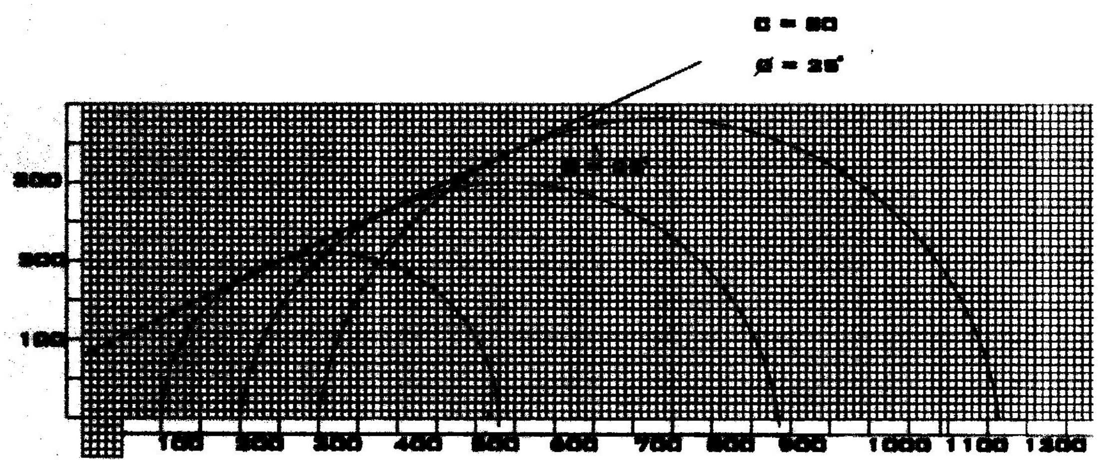
DEPTH: 3.0M

Ø = 24°
C= 9



TRIAXIAL COMPRESSION TEST

LOCATION: FUOYE IKOLE CAMPUS
PROJECT: PIT 5A
DEPTH: 3.0M



TRIAXIAL COMPRESSION TEST

LOCATION: FUOYE, IKOLE CAMPUS
PROJECT: PIT 5B
DEPTH: 3.0M