CLASSIFICATION AND STRENGTH CHARACTERISTICS OF SOILS ALONG FACULTY OF ENGINEERING ROAD FEDERAL UNIVERSITY OYE EKITI, IKOLE CAMPUS

By

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ABSTRACT

This research project aims at classifying and determining the strength characteristics of soils along Faculty of Engineering road, Federal University Oye Ekiti, Ikole campus. The road spans about 338m. The relevant soil tests include; particle size analysis, consistency limits (liquid, plastic, and shrinkage limits), natural moisture contents, specific gravity, compaction test, consolidation and shear strength tests. Liquid limit (LL) is defined as the moisture content, in percent, required to close a distance of 12.7mm along the bottom of the groove after 25 blows. Plastic limit (PL) is the transition from the plastic state to the semisolid state is termed the plastic limit, w. At this state the soil rolled into threads of about 3 mm diameter just crumbles. In this research project, disturbed and undisturbed soil samples is obtained from five labelled TP 1, TP 2, TP 3, TP 4, TP 5 at different locations at an interval of 67.6m along the road and taken to laboratory for relevant soil engineering tests carried out includes particle size distribution, natural moisture content, specific gravity test, compaction, consolidation, consistency limit and direct shear test. The reason for spacing sampling is to obtain variation of soil properties.

Significant amounts of the particulate constituents of the samples are shown to be fines (percentage passing No. 200 BS sieve). All the samples showed medium to low values of both liquid limit (LL) and plasticity index (PI). Looking at the sample some having liquid limit (LL) less than 40 and plasticity index (PI) less than 20. This probably indicates that the soil contains clay minerals of low plasticity. The samples classified as A-7-5 and A-2-6 (following the AASHTO classification system) and CI and CL (according to USCS classification system). The samples again recorded appreciable linear shrinkage values (8.6-10.7%). The results of the compaction test are values ranges from 14.0-20.9% and 1.53-1.83kg/m^3, the specific gravity of the samples ranges within 2.29 to 2.39 respectively. For the shear strength test, the tested samples recorded low values of cohesion (c) at TP2 and TP3 and high values of cohesion,(c) at TP1, TP4 & TP5 with moderately low values of angle of internal friction (phi) and the bearing capacity of various sampling points ranges from 299.1kPa to 3406.6kPa.

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DEDICATION

This report is dedicated to the Almighty God for being a source of knowledge, guidance and inspiration.

CERTIFICATION

This is to certify that this proposal was prepared by ALIU, Muhammed Olaotan (CVE/12/0826) under my supervision, in partial fulfillment of the requirements for the award of a Bachelor of Engineering (B.Eng) degree in Civil Engineering, Federal University Oye Ekiti, Ekiti State Nigeria.

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LIST OF ABBREVIATIONS

AASHTO – American Association of State High and Transportation Officials
USCS – Unified Soil Classification System
PL – Plastic Limit
LL – Liquid Limit
LI – Liquidity Index
TP – Trial Pit
Φ - Angle of Internal Resistance
C – Cohesion
MDD – Maximum Dry Density
OMC – Optimum Moisture Content
N_q - Terzaghi's Bearing Capacity Factor
N_c - Terzaghi's Bearing Capacity Factor
$N\gamma$ - Terzaghi's Bearing Capacity Factor

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accumulation of mineral particles formed by the weathering of rocks as part of the rock cycle, the void space between the particles containing water and/or air. Weak cementation can be due to carbonates or oxides precipitated between the particles, or due to organic matter, Knappett and Craig (2012). Soil consist of an aggregation of solid particles, water, and air. This fundamental composition gives rise to unique engineering properties, and the description of its mechanical behavior requires some of the most classic principles of engineering mechanics.

1.2.1 Soil Formation

Engineering soils are formed from the physical and chemical weathering of rocks. Soils may also contain organic matter from the decomposition of plants and animals. Physical weathering involves reduction of size without any change in the original composition of the parent rock, Craig (2012). The main agents responsible for this process are exfoliation, unloading, erosion, freezing, and thawing. Chemical weathering causes both reductions in size and chemical alteration of the original parent rock. The main agents responsible for chemical weathering are hydration, carbonation, and oxidation. Often chemical and physical weathering takes place in concert, Muni (2015).

According to Murthy (2004) Soils are formed from parent materials that resulted from the disintegration of rocks by various processes of physical and chemical weathering. The nature and structure of a given soil depends on the processes and conditions that formed it namely:

i. Breakdown of parent rock: weathering, decomposition, erosion.
ii. Transportation to site of final deposition gravity, flowing water, ice, wind.
iii. Environment of final deposition: flood plain, river terrace, glacial moraine lacustrine, or marine.
iv. Subsequent conditions of loading and drainage: little or no surcharge, heavy surcharge due to ice or overlying deposits, change from saline to freshwater, leaching contamination.

CHAPTER ONE

INTRODUCTION

1.1 General Background

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). The behavior 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 (2007).

1.2 Soil

To the civil engineer, soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks, the void space between the particles containing water and/or air, Craig (2004). Soil can be defined as an assemblage of non-metallic solid particles (mineral grains), and it consists of three phases: solid, liquid (water), and gas (air). Commonly used terms such as 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, Isao and Hemanta (2015). The term soil can have different meaning depending upon the field in which it is considered. To a geologist it is the material in the relative thin zone of the earth's surface within which roots occur, and which are formed as the products of past surface processes. The rest of the crust is grouped under the term "rocks". To a pedologist, is the substance existing on the surface, which supports plant life. To the civil engineer, soil is any uncemented or weakly cemented

1.2.2 Types of soil

In terms of soil texture, soil type usually refers to the different sizes of mineral particles in a particular sample. Texture refers to the appearance or feel of a soil, sands and gravels are grouped together as coarse grained soils. Clays and silts are fine-grained soils. Coarse-grained soils feel gritty and hard, fine grained soils feel smooth, Muni (2015). Soil types include:

i. Gravel
ii. Sand
iii. Silt
iv. Clay

On the basis of origin of their constituents, soils can be divided into two large group, Murthy (2004):

a. Residual soils
b. Transported soils

1.2.3 Residual Soils

These soils are formed in situ by chemical weathering and may be found on level rock surfaces where the action of the elements has produced a soil with little tendency to move. Residual soils can also occur whenever the rate of breakup of the rock exceeds the rate of removal. If the parent rock is igneous or metamorphic the resulting soil sizes range from silt to gravel, Smith (2014). Residual soils are found at the same location where they have been formed. Generally, the depth of residual soils varies from 5 to 20m.

1.2.4 Transported Soils

Weathered rock materials can be moved from their original site to new locations by one or more of the transportation agencies to form transported soils. Transported soils are classified based on the mode of transportation and the final deposition environment. Transported soils can be subdivided into five major categories based on the transporting agent, Das (2012):

i. Gravity transported soil.
ii. Lacustrine (lake) deposits.
iii. Alluvial or fluvial soil deposited by running water.

iv. Glacial deposited by glaciers.

v. Aeolian deposited by the wind.

1.3 Soil Description and Classification

It is essential that a standard language should exist for the description of soils. A comprehensive description should include 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, Knapett and Craig (2012). 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. 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 macrofabric, should be carefully described, as this can influence the engineering behaviour of the in-situ soil to a considerable extent, Craig (2004). It is important to distinguish between soil description and soil classification. 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, Knapett and Craig (2012).

1.4 Problem Statement

The road along Faculty of Engineering Federal University Oye Ekiti is located in Ikole campus. Little or no work has been done to determine the soil strength and soil type of the study area hereby justifying the research work.

1.5 Geology of the Study Area

The study area is situated at the Federal University Oye Ikole campus, Ikole local government area of Ekiti state, Ikole is located at 861170mN; 777566mE. 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). The figure below shows the geological distribution of minerals in different parts of Ekiti state.

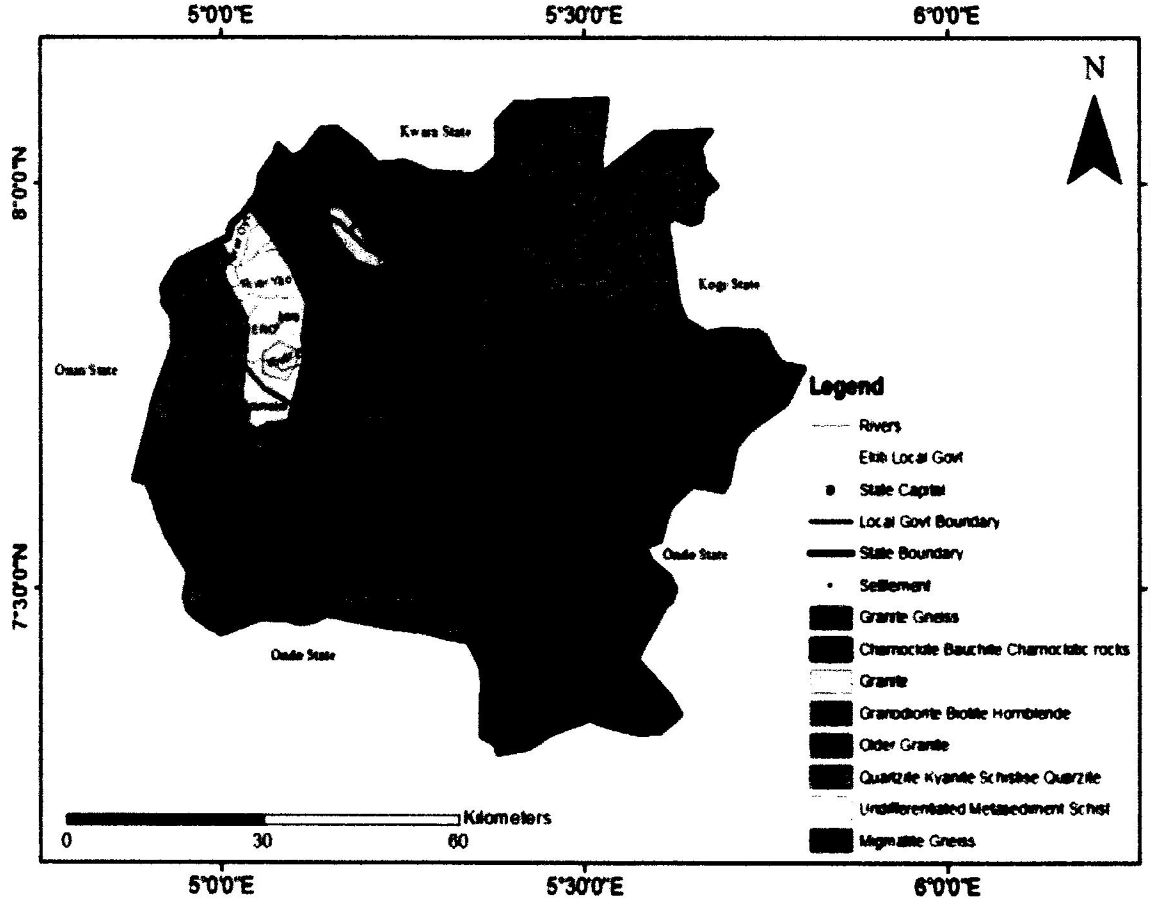


Figure 1.1 Geologic Map of Ekiti-State (Digitized from Ademilua 2014)

The basement rocks are concealed in places by a variably thick overburden, Bayowa et al (2014). The major lithologic units according to Rahaman (1976), (1988) are the migmatite-gneiss complex; the older granites; the charnockitic rocks; the slightly migmatised to unmigmatised paraschists and metaigneous rock and the unmetamorphosed granitic rocks. The migmatitegneiss complex is composed mainly of early gneiss, mafic and ultramafic bands and the granitic or felsic components. The rock type is the most widespread, covering about half of the study area. In the investigation conducted by Adeyeri et al (2017), they concluded that the soils are mostly lateritic and are suitable as subgrade, subbase and base course materials in highway construction. The lateritic soils encountered at the site can comfortably support shallow foundations for loads of the order of 50kN/m^2 – 200kN/m^2. In their own work Adeyeri et al (2017), investigated the stratigraphic profile and geotechnical properties of soils in Ikole area of Ekiti State. The site investigation revealed a subsoil stratification consisting of reddish brown granitic clayey sand (Laterite) top layer from existing ground level to about 12.0m depth. This is then underlain by a layer of mottled, brown, decomposed micaceous sand to a depth of 16.5m to 18m. Immediately after this is the layer of mottled grey, decomposing quartzite sand to 19.0m and this is further underlain by fragments of granitic rock (freshly weathered) to the exploratory termination depth of 19.5m. The figure below describes the stratigraphic soil profile.

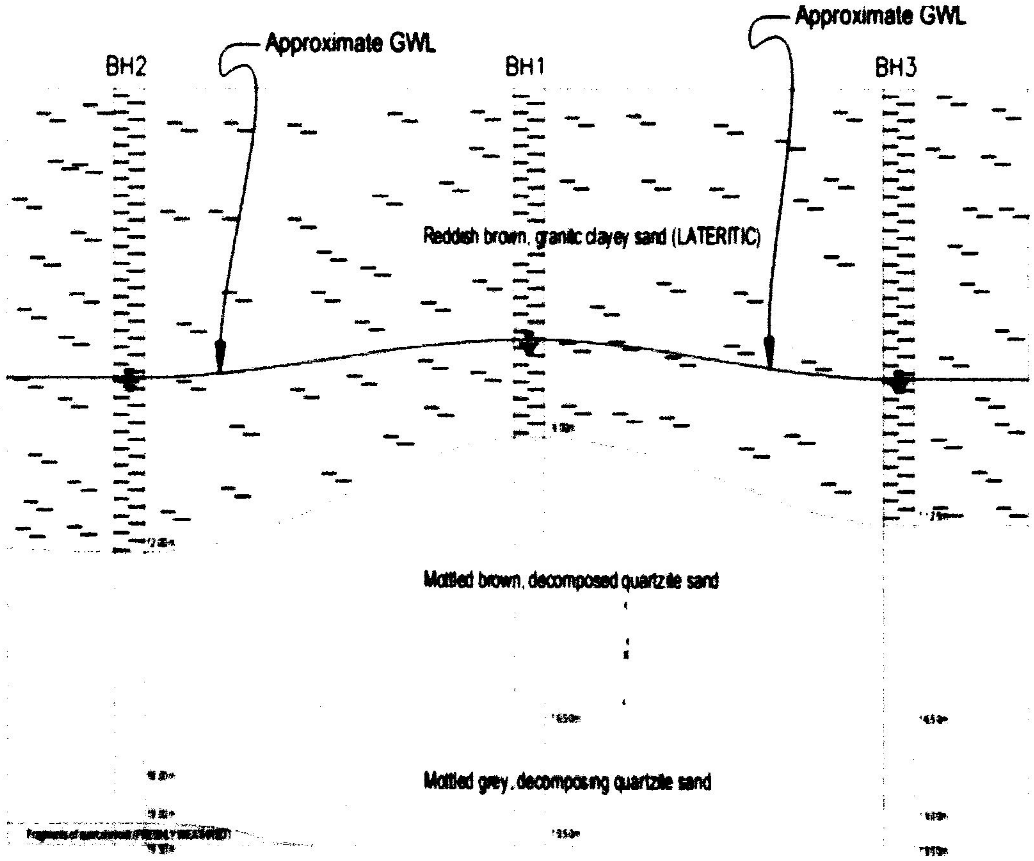


Figure 1.2 Stratigraphic Description of the Soil Profile Bolarinwa et al (2017)

In the work of Bolarinwa et al (2017), from the soil exploration and laboratory analysis, it was inferred that, the soils encountered from the superficial to about 12m depth are mostly lateritic soils because they possess both cohesive and cohesionless soil properties. It can be concluded from the recommendations made that apart from the soils suitability as a subgrade, subbase and base course materials in highway construction, they can be recommended for making mud blocks which is useful in building works.

1.6 Aim and Objectives

The aim of this research is to classify the soil and determine its strength characteristics. The study area is along the Faculty of Engineering road, Federal University, Oye Ekiti, Ikole Campus. This research is carried out to obtain the following objectives;

i. To select five (5) locations along this road from which to take soil samples to be representative of soils on Ikole campus along the faculty of engineering road.
ii. To carry out some required geotechnical tests such as, specific gravity, gradation, direct shear test, consistency limits, and compaction tests.
iii. To carry out strength tests on the soil samples
iv. To classify the soils, using the American Association of State Highway and Transport Officials (AASHTO) and Unified Soil Classification System (USCS) methods.
v. To establish the economic value of the soils.
vi. To make recommendations based on the outcome of the laboratory test results.

1.7 Significance of Research

The study is considered to be very important as it will investigate the properties of soil along the road, and signify the classification of the soil, which in turns guides the use of the soil as a construction material. This research work would aid future works on the soils of this area perhaps with respect to research, construction uses, etc.

1.8 Scope and Limitations of Study

The samples of disturbed soils will be collected from five (5) locations along the faculty of engineering road in The Federal University Oye Ekiti in Ikole-Ekiti and will be subjected to the following tests:

1. **Physical properties tests;**
   i. Specific gravity
   ii. Natural moisture content
   iii. Sieve analysis
   iv. Consistency test
2. **Strength tests;**
   i. Compaction test
   ii. Direct Shear test

The study is an investigation of the soil along the faculty of engineering road inside the Federal University Oye Ekiti, Ikole campus in Ikole area of Ekiti state. The research is limited to only five (5) locations along the road.

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 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 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). According to knapett and Craig (2012), to the civil engineer, soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks as part of the rock cycle, the void space between the particles containing water and/or air. 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. Particle sizes in soils can vary from over 100 mm to less than 0.001mm, Isao and Hemantha (2015).

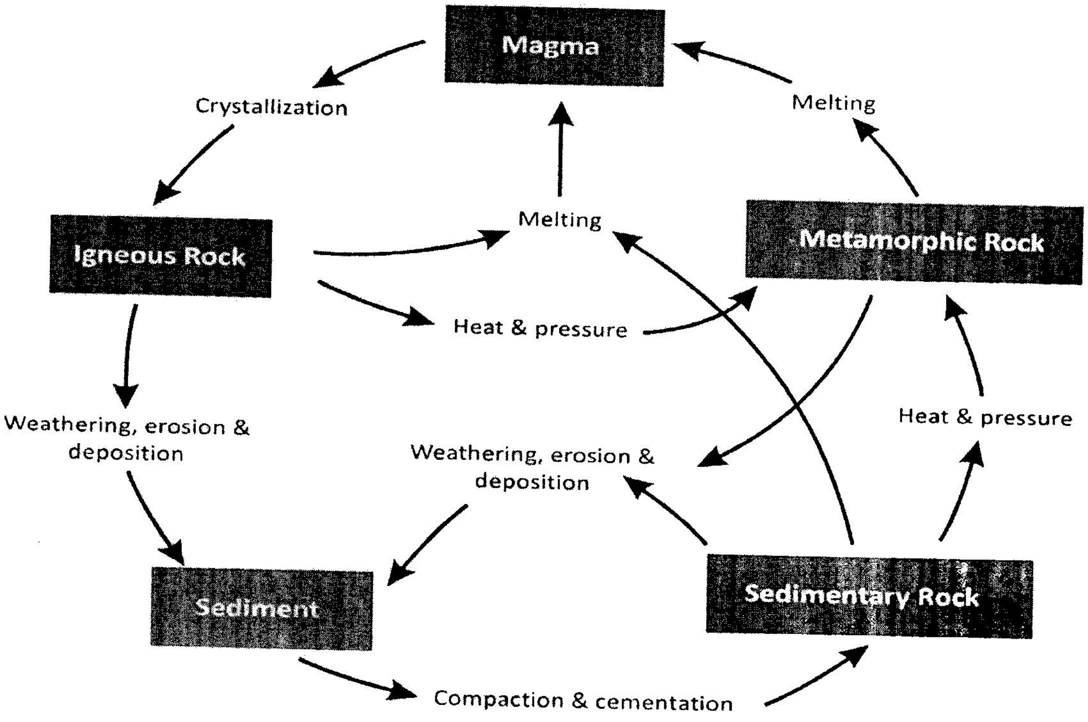


Figure 1.1 Rock Cycle.

2.3 Physical and Geotechnical Properties of Soil

2.3.1 Permeability

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

2.3.2 Moisture content

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

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

2.3.3 Particle Size and Gradation

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.

Table 2.1 Classification of materials based on particle size distribution (BS 1924: Part 1: 1990)

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.

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.

Table 2.2 Soil classifications and properties (Townsend, 1973)

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 (cm²)	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 moisture change needs to be addressed at design.	Uniform mixing may be difficult, but clay is easily stabilised. Clay minerals can react with binders to form cementitious products.

Particle size plays a dominant role in distinguishing soil types. Commonly used names of soil such as gravel, sand, silt, and clay are based on their grain sizes. The boundary particle sizes are slightly different depending on the standards. 2.0 mm in AASHTO or 4.75mm in USCS (Unified Soil Classification System) and in the ASTM Soil Classification System are the boundary particle sizes between gravel and sand. 75μm (0.075mm) is the boundary between sand and silt in both standards, and 5μm is the one between silt and clay in AASHTO. In USCS (and also in ASTM), materials that are finer than 75μm are called "fine." Note that in some other standards, such as British Soil Classification (BS8004, 1986), 2μm is used as the boundary between silt and clay. In order to separate grain sizes of soil assembly, a set of sieves is used for larger grain sizes. In particular, the boundary of 75μm grain size is important; 75μm is the opening size of a No. 200 sieve, which is practically the smallest size of sieves. Particles that are smaller than No. 200 sieve (minus No. 200 material) cannot be mechanically sieved easily due to developed static electricity on the surface of particles. If water is poured on dry minus No. 200 material, particles are easily suspended in the water and the water gets dirty.

That is a good indication of an existence of minus No. 200 or "fine" material in it. Gravel and sand are called cohesionless (granular) soils, and clay is called cohesive soils. Silt is a transitional material between granular soils and cohesive soils. These two soil groups have distinguished differences in engineering behavior. Granular soils' resistance upon shearing mostly comes from their surface friction and interlocking mechanisms. On the other hand, cohesive soils' resistance comes from short-range particle-to-particle interactive forces. To identify grain size characteristics of soils, a grain size distribution curve is developed. First, sieve analysis is conducted. A variety of sieves with different openings are stacked, with the largest opening sieve on the top and smaller ones on the lower sections. The smallest (usually a No. 200 sieve) is placed at the second from the bottom and a pan with no opening at the bottom. Table 2.3 shows US standard sieve numbers and their corresponding openings.

Table 2.3 US Standard Sieve Numbers and Openings (Isao and Hemantha, 2015)

US Standard Sieve No.	Opening
4	4.75
10	2.00
20	0.85
40	0.425
60	0.25
100	0.15
140	0.106

2.3.4 Atterberg Limits

The water contents corresponding to the transition from one state to another are termed as Atterberg Limits and the tests required to determine the limits are the Atterberg Limit Tests. The testing procedures of Atterberg were subsequently improved by Casagrande (1932).

2.3.5 Plastic Limits

The plastic limit is defined as the moist content, in percent, at which the soil crumbles when rolled into threads of 3.2mm diameter. The plastic limit is the lower limit of the plastic stage of soil. The plastic limit test is simple and is performed by repeated rolling of an ellipsoidal size soil mass by hand on a ground glass plate. The procedure for the plastic limit test is given by ASTM Test Designation D-4318, Das (2008).

2.3.6 Liquid Limit

The transition state from the liquid state to a plastic state is called the liquid limit, at this stage all soils possess a certain small shear strength. This arbitrarily chosen shear strength is probably the smallest value that is feasible to measure in a standardized procedure, Murthy (2007).

2.3.7 Shrinkage Limit

Shrinkage Limit (SL) is defined as the moisture content at which no further volume change occurs with further reduction in moisture content. (SL represents the amount of water required to fully saturate the soil (100% saturation). The consistency of soils according to Atterberg limits gives the following diagram.

2.3.8 Plasticity index

Describes the range of plastic behavior and is found as a difference between the LL and PL Plastic Index (PI) = Liquid Limit (LL) – Plastic Limit (PL).

Table 2.4 Plasticity Index Ranges

	Plasticity Index (PI)
Non-plastic	0
Slightly plastic	0 – 5
Low plasticity	5 – 20
Medium plasticity	10 – 20
High plasticity	20 – 40
Very high plasticity	> 40

2.3.9 Liquidity Index

A measure of the soils sensitivity in respect to the soil response to sudden shear forces, such as vibrations and earthquakes. At LI = 1.0, the soil exhibits liquid properties and so is very sensitive while at L.I = 0.0 indicates a soil at the plastic limit and soil is no longer sensitive.

Where; Wn – water content in natural conditions

PL – Plastic Limit

PI – Plastic Index

2.4 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 cohesivity of a soil assists in the selection of Stabilization/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 exsitu 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. 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). 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) stated 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 stabilize a soil. Although soils with liquid limits >60% can be stabilized, the amounts of cement required can be uneconomical and result in unacceptable volume increase.

2.5 Strength

According to Brady and Weil (1996), the strength of a soil measures its capacity to withstand stresses without collapsing or becoming deformed. Soil strength can be considered in terms of the ability 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 interparticle contracts. Normal stress may be resisted by the soil skeleton due to an increase in the interparticulate 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 stress can be influenced by a number of related soil characteristics, amongst which are: soil compressibility; soil compatibility; and bearing resistance. These factors in turn are determined by parameters such as soil moisture content, particle size distribution and the mineralogy of 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 subjected to little compressibility whereas silicate clays are easily compressed. The bearing capacity of the materials can be important both in terms of long-term engineering performance to carry loads and also supporting heavy plant in the short-term.

2.5.1 Compaction and Consolidation

The terms "Compaction" and "Consolidation" are often interchangeably used. 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, while on the other hand, consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to drainage of some of the pore water, the process is continued until the excess pore water pressure set up by increase in total stress has completely dissipated; the simplest case is that of one dimensional consolidation in which a condition of zero lateral strain is implicit, Craig (2004). According to Bolarinwa et al (2017), Compaction is an artificial process, which basically involves densification of the soil mass through reduction of air in voids of the soil mass while the latter is a natural process of gradual reduction in volume of the soil mass (settlement) through expulsion of the excess pore water in the soil over a period of time. It should also be noted that compaction is not time dependent while time is a major factor for completion in consolidation process. According to Terzaghi's theory of consolidation the following conditions are assumed;

i. Homogenous soil.
ii. Complete saturation.
iii. Incompressible water and soil grains.
iv. Compression and flow in one direction.
v. Action of differential soil mass similar to the action of large soil mass.
vi. Linear relationship between pressure and void ratio.

2.6 Soil density

This is the ratio of mass to volume of a soil. In simpler terms, it is a measure of the heaviness of soil. The density of soils is determined according to ASTM D85400, Standard Test Methods for Specific Gravity of Soils by Water Pycnometer. The density of the soils is used in the calculation of soil particle size distribution as specified in ASTM D422-63.

2.7 Specific gravity

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

2.8 Soil Classification Systems

Soil classification systems divide soils into groups and subgroups based on common engineering properties such as the grain-size distribution, liquid limit, and plastic limit. The two major classification systems presently in use are:

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

(2) The Unified Soil Classification System (also ASTM). The AASHTO system is used mainly for the classification of highway subgrades. It is not used in foundation construction. Das (2012).

2.8.1 AASHTO Soil Classification System

The AASHTO Soil Classification System was originally proposed by the Highway Research Board's Committee on Classification of Materials for Subgrades and Granular Type Roads (1945). According to the present form of this system, soils can be classified according to eight major groups, A-1 through A-8, based on their grain-size distribution, liquid limit, and plasticity indices. Soils listed in groups A-1, A-2, and A-3 are coarse-grained materials, and those in groups A-4, A-5, A-6, and A-7 are fine-grained materials. Peat, muck, and other highly organic soils are classified under A-8. They are identified by visual inspection. The AASHTO classification system (for soils A-1 through A-7) is presented in Table 2.5. Note that group A-7 includes two types of soil. For the A-7-5 type, the plasticity index of the soil is less than or equal to the liquid limit minus 30. For the A-7-6 type, the plasticity index is greater than the liquid limit minus 30. For qualitative evaluation of the desirability of a soil as a highway subgrade material, a number referred to as the group index has also been developed. The higher the value of the group index for a given soil, the weaker will be the soil's performance as a subgrade. A group index of 20 or more indicates a very poor subgrade material. The formula for the group index is:

$$GI = (F_{200} - 35)[0.2 + 0.005(LL - 400] + 0.01(F_{200} - 15)(PI - 10)$$

Table 2.5 Classification of soils and soil-aggregate mixtures (AASHTO M 145-91).

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

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

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

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

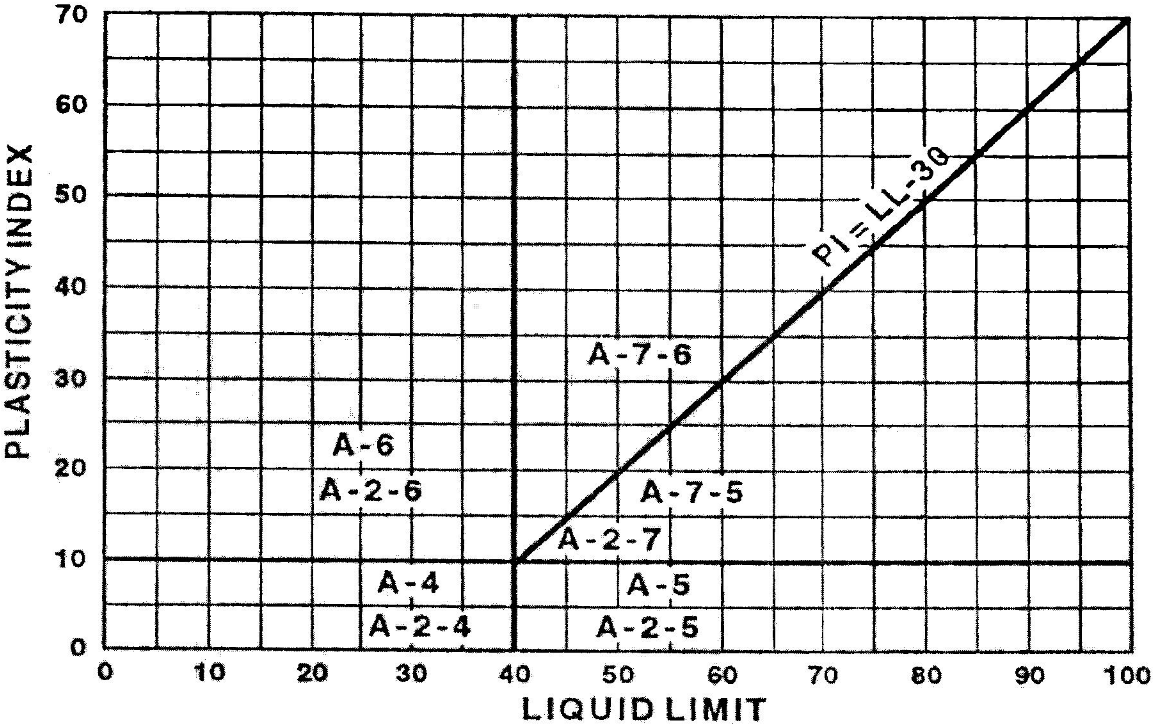


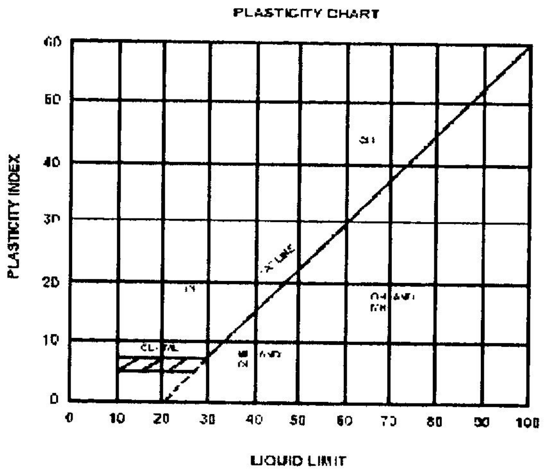
Figure 2.2 Plasticity Chart. British system (BS 5930: 1999).

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

Table 2.6 Unified Soil Classification System chart (after U.S. Army Corps of Engineers, Waterways Experiment Station, TM 3-357, 1953).

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

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

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

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

2.8.3 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 4.76millimeters (No. 4) sieve. The same coarse-grained soil could, therefore, have different percentages of gravel and sand in the USCS classification systems.

2.9 Ground investigation

The methods available for soil exploration may classify as follows:

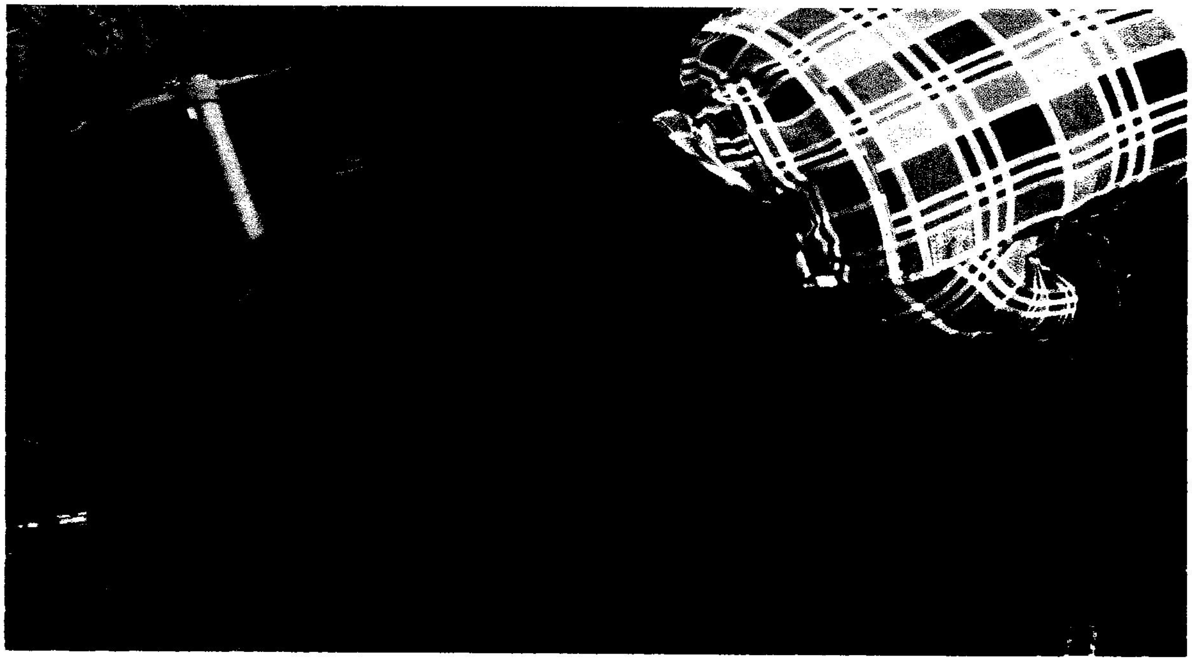
i. Direct methods – test pits, trial pits and trenches.

ii. Semi-direct methods – borings.

iii. Indirect methods- soundings or penetration tests and geophysical methods.

2.9.1 Test Pits

A test pit is a hole dug in the ground that is large enough for a ladder to be inserted thus permitting a close examination of the sides. They are normally limited to a depth not more than 3m and are more suitable where load bearing strata is at shallow depth allowing in-situ soil conditions such as stratification be observed directly. Disturbed and undisturbed samples can be taken from the sides and bottom of the pit at any orientation that may be required.



Plates 1 Measuring the trial pit using measuring tape

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.

3.1 Desk Study

It is the first step in an exploration exercise and involves collecting published information about the site under investigation and pulling it all together to build a conceptual model of the site. Most of the information gathered at desk study stage is contained in maps, published reports and aerial photographs. A study of the site geology is also important at this stage.

3.2 Field work

In order to carry out the geotechnical examination work, a trial pit will be dug at the locations chosen for collection of soil sample. Basically the scope of field work involves; the exploration of four trial pits by using digger and shovel for digging technique. Disturbed and Undisturbed 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. Soil sample collected will be labelled as shown below;

Table 3.1 Coordinates of Locations

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

3.3 Sampling

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 must be given to it. Sampling simply means going to the field to collect soil specimens at various locations depending on the types and nature of the tests that will be carried out. The sample must be enough and adequate for the test to prevent a second visit to the site. The locations for the collected samples are generally referred to as borrow pits, trial pits, bore holes etc. depending on the method employed. The general over view of the site must be accessed in terms of orientation, terrain, topography, valley and likely stream or river around the site. Also, the area of worst condition should also be identified.

3.3.1 Methods of Collecting Samples

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

i. Disturbed sampling

ii. Undisturbed sampling

3.3.2 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 should be well labeled and dated for the purpose of easy identification and to pervert mix up in the laboratory.

3.3.3 Undisturbed Sampling

These are being collected using a sampling tube which are hydraulically or electrically drilled into the soil mass to a specified depth and gently removed the sampling tube with the obtained sample.

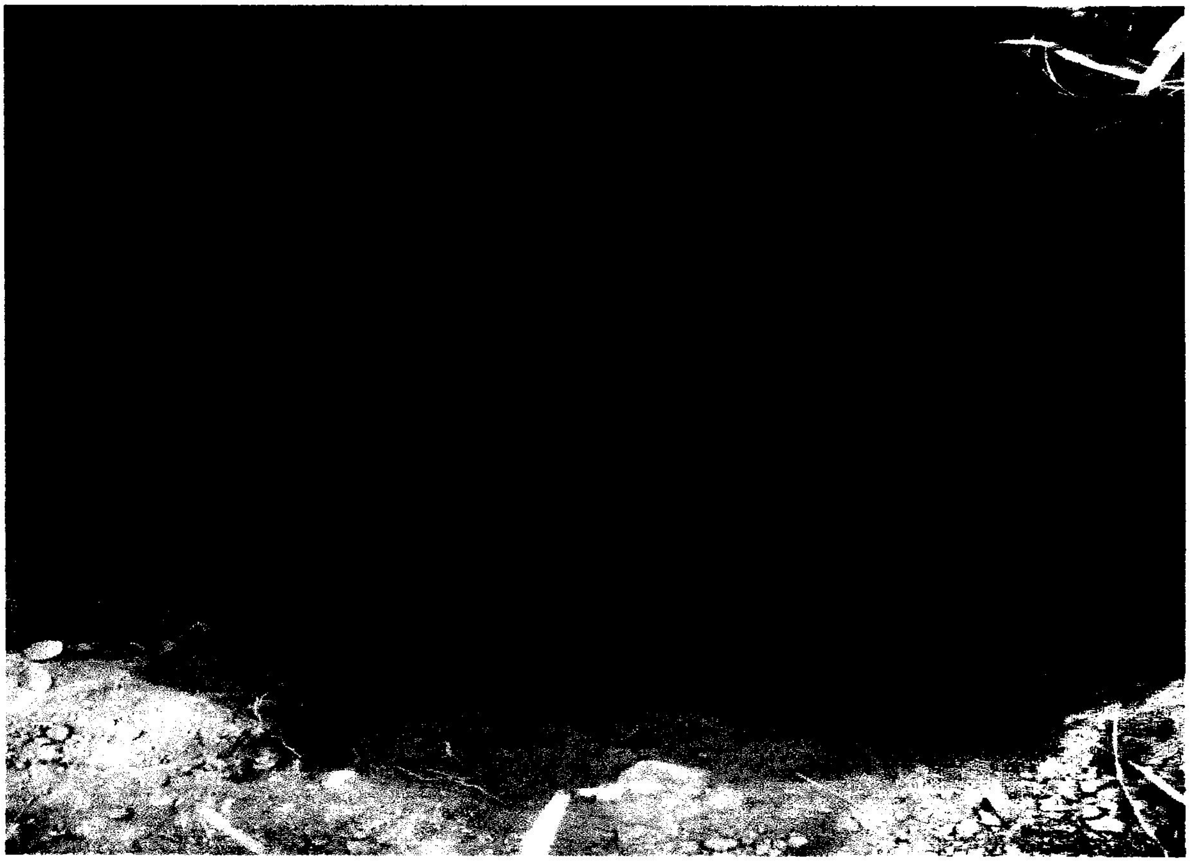


Plate 2: Samples being taken at the given location

3.3.4 Sampling Technique

The type of technique that will be adopted for taking the sample is hand dug method use for well method will be used 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). The laboratory test carried out to determine the suitability of the soils for use as base and sub-base material using the AASHTO standard method in relation to the generation specification for roads and bridges. Laboratory tests carried out are as follows:

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

- i. Particle size analysis,
- ii. Moisture content determination,
- iii. Consistency limit test {Atterberg},
- iv. Specific gravity test.

B. Determination of mechanical properties of soils:

- i. Compaction,
- ii. Direct shear test.

3.5 METHODS

3.5.1 Moisture Content Test

For determination of the moisture content of soil by oven drying method.

Equipment and Tools

Oven (1050C to 1100C min.)

Metal container

Balance (0.01 g accuracy)

Procedure

i. The number of the container is recorded, cleaned, dried and weighed, (W_1).

ii. About 15-30 g of soil is placed in the container and the weight of soil with the sample is recorded, (W_2).

iii. The can with the soil is placed in oven for 24hours maintained at a temperature 1050 to 1100C.

iv. After drying the container is removed from the oven and allowed to cool at room temperature.

v. After cooling the soil with container is weighed, (W_3).

Reporting of Results

The water content, $w = \frac{W2-W3}{W3-W1} \times 100$

An average of three determinations should be taken.

W_1=Mass of container, g

W_2=Mass of container and wet soil, g

W_3=Mass of container and dry soil, g

The water content of the soil is reported to two significant figures.

3.5.2 Atterberg Limits

Plastic Limit Test

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

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

Preparation of Sample

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

Procedure to determine the Plastic Limit of Soil

i) Take about 8g of the soil and roll it with fingers on a glass plate. The rate of rolling should be between 80 to 90 strokes per minute to form a 3mm diameter.

ii) If the dia. of the threads can be reduced to less than 3mm, without any cracks appearing, it means that the water content is more than its plastic limit. Knead the soil to reduce the water content and roll it into a thread again.

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

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

v) Repeat the process at least twice more with fresh samples of plastic soil each time.

Reporting of Results

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

Liquid Limit Test

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

Tools

i) Casagrande's liquid limit device
ii)Grooving tools of both standard and ASTM types
iii)Oven
iv) Evaporating dish
v) Spatula
vi) IS Sieve of size 425µm
vii) Weighing balance, with 0.01g accuracy
viii) Wash bottle
ix) Air-tight and non-corrodible container for determination of moisture content

Preparation of Sample

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

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

Procedure to Determine the Liquid Limit of soil

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

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

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

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

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

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

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

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

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

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

Reporting of Results

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

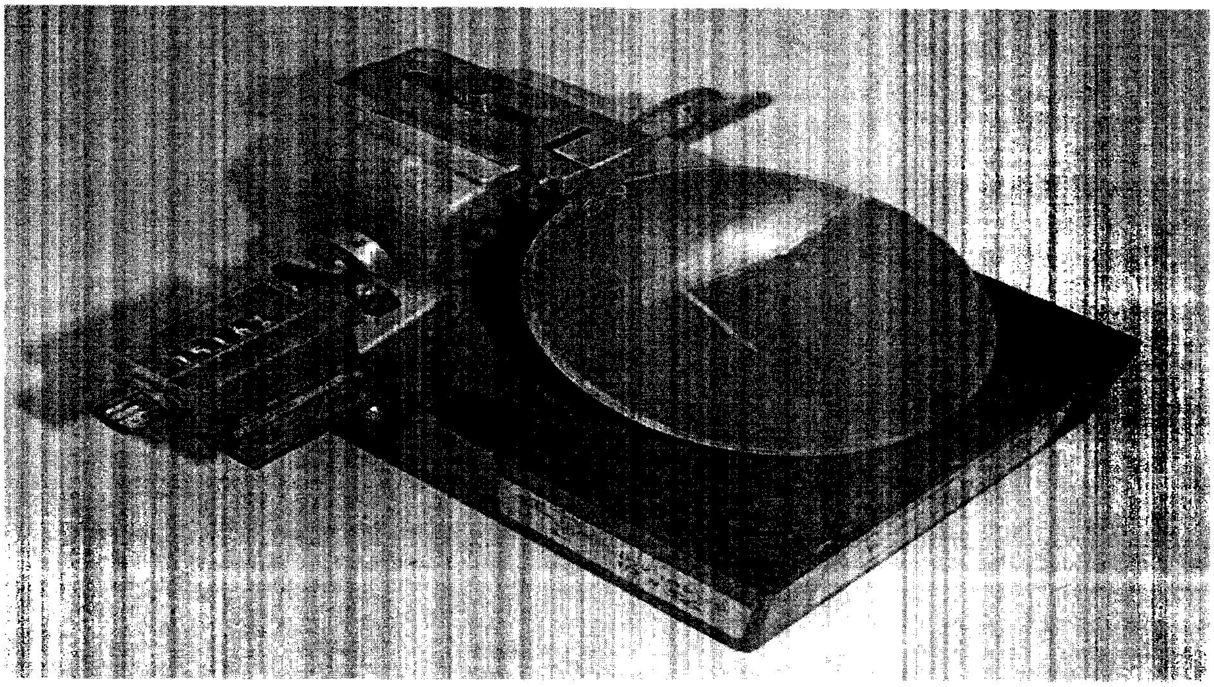


Plate 3: Atterberg limit apparatus

3.5.3 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, should be dried in air or in the sun. In wet weather, the drying apparatus may be used in which case the temperature of the sample should not exceed 60°C. Clod may be broken with wooden mallet to hasten drying. Tree roots and pieces of bark should be removed from the sample.

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

iii) A representative soil sample of required quantity as given below is taken and dried in the oven at 105 to 120°C.

Procedure to determine Particle Size Distribution of Soil

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

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

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

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

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

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

3.5.4 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.

Standard Reference

ASTM D854-00 – Standard test for specific gravity of soil solids by water pycnometer.

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

Preparation of Sample

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

Procedure to Determine the Specific Gravity of Fine-Grained Soil

i) The density bottle along with the stopper, should be 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 should be transferred to the density bottle directly from the desiccator in which it was cooled. The bottles and contents together with the stopper should be weighed to the nearest 0.001g (W_2).

iii) Cover the soil with air-free distilled water from the glass wash bottle and leave for a period of 2 to 3hrs. for soaking. Add water to fill the bottle to about half.

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

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

vi) Gently stir the soil in the density bottle with a clean glass rod, carefully wash off the adhering particles from the rod with some drops of distilled water and see that no more soil particles are lost.

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

viii) Observe the constant temperature in the bottle and record.

ix) Insert the stopper in the density bottle, wipe and weigh (W_3).

x) Now empty the bottle, clean thoroughly and fill the density bottle with distilled water at the same temperature. Insert the stopper in the bottle, wipe dry from the outside and weigh (W_4).

xi) Take at least two such observations for the same soil.

Reporting of Results

The specific gravity G of the soil = $(W_2 - W_1) / [(W_4 -_1)-(W_3-W_2)]$. The specific gravity should be calculated at a temperature of 27°C and reported to the nearest 0.01. If the room temperature is different from 27°C, the following correction should be done:- G' = Kg

Where,
G' = Corrected specific gravity at 27°C
k = [Relative density of water at room temperature]/ Relative density of water at 27°C.

A sample for the record of the test results is given below.

3.5.5 Compaction Test

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

Standard Proctor test

Modified AASHTO method

West Africa method

Tools

- 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

Preparation of Sample

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

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

A) Soil not susceptible to crushing during compaction –

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

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

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

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

B) Soil susceptible to crushing during compaction– Five or more 2.5kg samples of air-dried soil passing through the 19mm IS Sieve, should be taken. The samples should each be mixed thoroughly with different amounts of water and stored in a sealed container as mentioned in Part

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

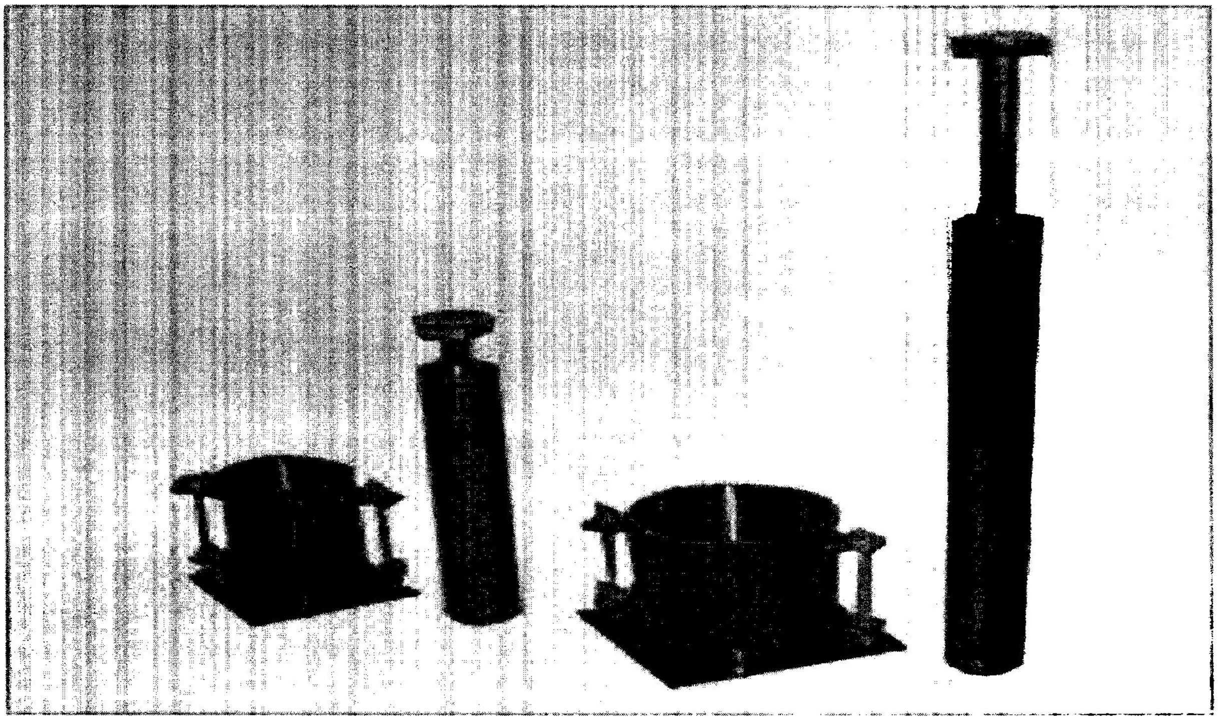


Plate 4: Moulds and Rammers

3.5.6 Direct Shear Test

To determine the shearing 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) Aluminum container.
ix) Spatula.

Procedure

1. Check the inner dimension of the soil container.

2. Put the parts of the soil container together.

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

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

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

6. Make the surface of the soil plane.

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

8. Measure the thickness of soil specimen.

9. Apply the desired normal load.

10. Remove the shear pin.

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

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

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

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

15. Take volume change readings till failure.

16. Add 5kg normal stress 0.5kg/cm^2 and continue the experiment till failure

17. Record 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 were included as necessary for clarity and further details were provided in the appendices. Laboratory tests were performed on the sample collected from the four locations used as case study. The assessment characteristics such as Atterberg limits, particle size distribution, specific gravity, compaction test, California bearing ratio and natural moisture content test were determined.

The summary of the results of the laboratory tests carried out on the shale sample is presented in table.

Table 4.1 Summary of all the Test Result Analysis

Location		**TP 1**	**TP 2**	**TP 3**	**TP 4**	**TP 5**
Natural Moisture Content	%	18.1	20.4	24.0	18.5	17.56
Sieve Analysis	2.36	94.2	95.8	93.9	93.5	**99.0**
	600	79.8	80.7	74.8	74.1	**88.2**
	0.075	48.7	56.1	47.9	45.9	**68.2**
Atterberg Limit	LL %	48.1	46.3	36.0	33.8	**46.0**
	PL %	18.4	23.3	17.2	21.4	**24.2**
	PI %	29.7	23.0	18.8	10.4	**21.8**
AASHTO Classification		A-2-6	A-2-6	A-7-5	A-7-5	**A-2-6**
USCS Classification		CL	CL	CH	CH	**CL**
Specific Gravity		2.29	2.14	2.43	2.40	**2.39**
Compaction	OMC %	20.9	18.5	17.2	14.0	**15.0**
	MDD Kg/m^2	1.69	1.76	1.80	1.83	**1.53**
Direct Shear	C	70	0	0	475	**100**
	Φ	35	48	45	30	**27**

4.1 Natural Moisture Content

The natural moisture content gives an idea of the state of the soil in the field. The natural water content also called natural moisture content is the ratio of the weight of the solids in a given mass of soil. The results from the trail pits have high values of moisture content which indicates high water retention. The moisture content values ranges from 17.56% to 24.0%. The table below shows the results of the moisture content test.

Table 4.2: Results of Moisture Content of Soil Samples

Location	TP 1	TP 2	TP 3	TP 4	TP 5
Depth(m)	1.0	1.0	1.0	1.0	1.0
M.C %	18.1	20.4	24.0	18.5	17.56

4.2 Particle Size Distribution

The result of the laboratory tests carried out on the soil samples shows that significant amounts of the constituents to be fines (% passing No. 200 BS. sieve). All The samples classified as A-7-5 and A-2-6 (following The AASHTO classification system). The soil samples classified under A-7-5 in the AASHTO classification table are silty-clay materials with (>35% passing the 0.075mm sieve). They possess liquid limit of (LL-41min) and plasticity index of (PI-11min). The usual types of significant constituent materials are clayey soils and the general rating as a subgrade material ranges from fair to poor. For the soil samples classified under the subgroup A-2-6 in the AASHTO classification table are materials wit (35% or less passing the 0.075mm sieve). They possess liquid limit of (LL-41min) and plasticity index of (PI-11min). The type of constituent materials are silty or clayey gravel and sand. The general rating as a subgrade material ranges from excellent to good.

Table 4.3: Results of Sieve Analysis Test of Samples

Location	Sieve size	TP 1	TP 2	TP 3	TP 4	TP 5
Sieve Analysis	2.36	94.2	95.8	93.9	93.5	99.0
	600	79.8	80.7	74.8	74.1	88.2
	0.075	48.7	56.1	47.9	45.9	68.2
AASHTO Classsification		A-2-6	A-2-6	A-7-5	A-7-5	A-2-6
USCS Classification		CL	CL	CH	CH	CL

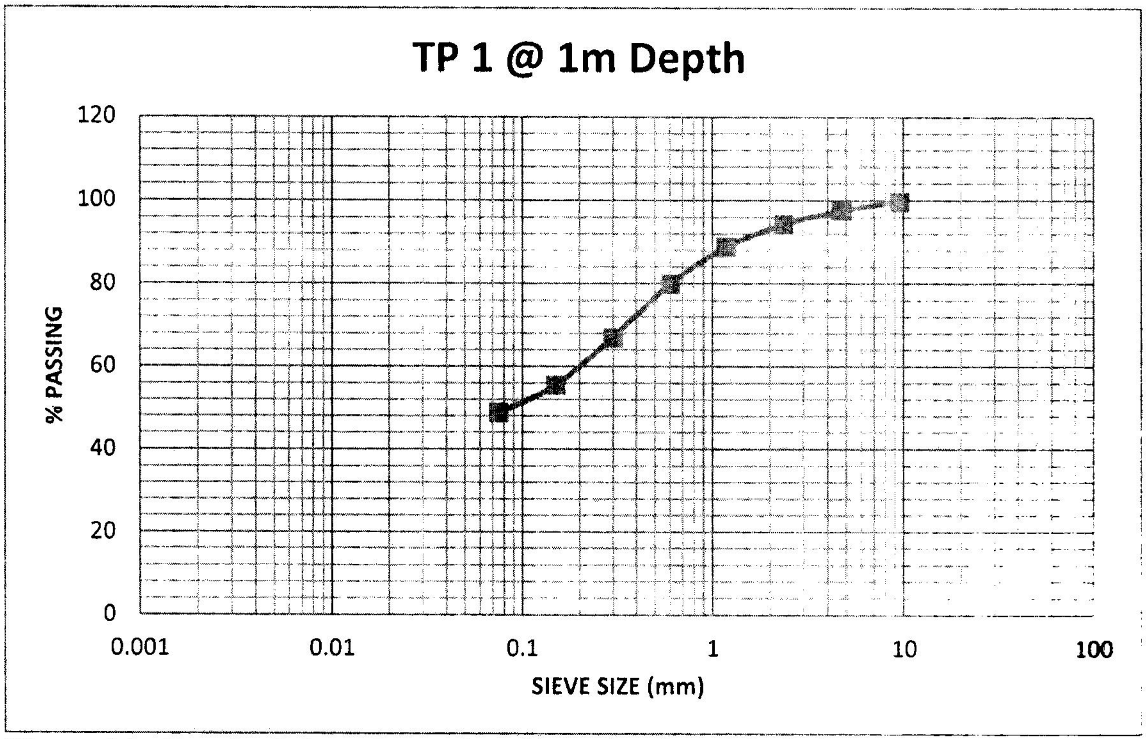


Figure 4.1: Graph of sieve size against %passing for trial pit 1

4.3 Consistency limit test

The samples showed medium to low values of both liquid limit (LL) and plasticity index (PI). Looking at the sample some having liquid limit (LL) less than 40 and plasticity index (PI) less than 20. This probably indicates that the soil contains clay minerals of low plasticity. Table 4.4 below shows the results of the test carried out.

Table 4.4: Results for consistency limit (Atterberg Limit) test

Location	TP1	TP2	TP3	TP4	TP5
LL %	48.1	46.3	36.0	33.8	46.0
PL %	18.4	23.3	17.2	21.4	24.2
PI %	29.7	23.0	18.8	10.4	21.8

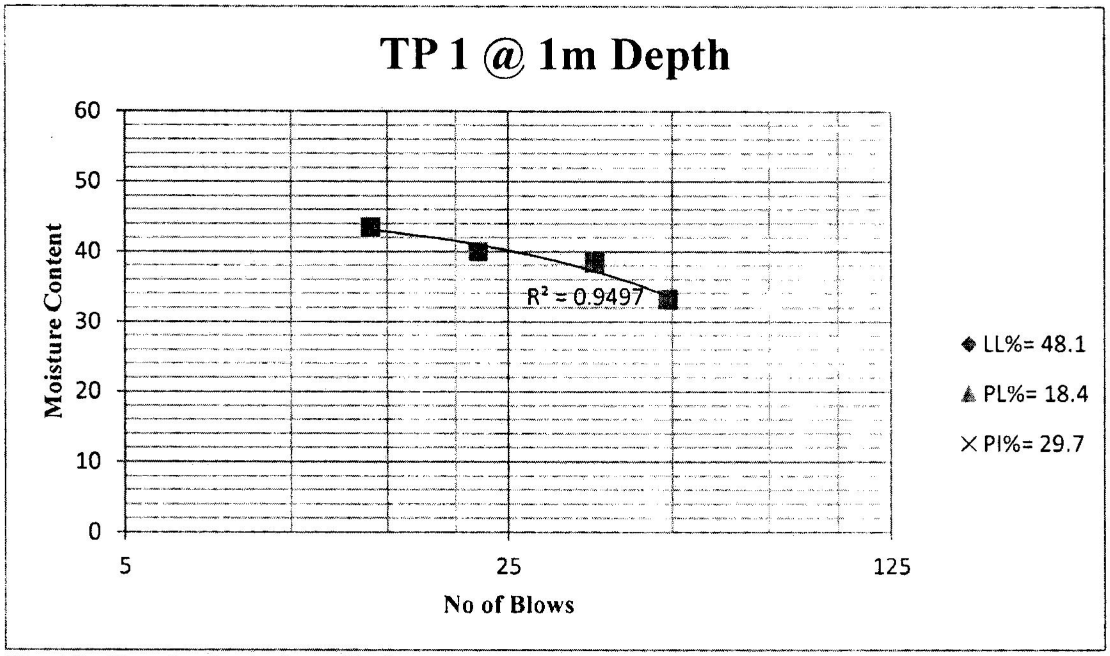


Fig.4.2: Graph of atterberg limit

4.4 Specific Gravity Test

The summary of results of specific gravity (S.G) are shown in the table 4.4 below, from the results obtained at the depth of 1m the values ranges from 2.29 - 2.39. This indicates that the soil contains some clay content since the average value for clay content is 2.36.

4.5 Compaction Test

The results and graphs are shown in appendix C. from the results obtained, the OMC and MDD were derived from the graphs. The results of the compaction test are values ranges from 14.0-20.9% and 1.53-1.83kg/m^3

4.6 Direct Shear

Appendix G shows the result and graphs of the direct shear test carried out. For the shear strength test, the tested samples recorded low values of cohesion (c) at TP2 and TP3 and high values of cohesion (c) at TP1, TP4 & TP5 with moderately low values of angle of internal friction (phi). The table 4.5 below shows the cohesion 'c' and angle of internal friction phi (φ) at 1m depth.

Table 4.5: Result of direct shear test

Location		TP1	TP2	TP3	TP4	TP5
Depth	(m)	1	1	1	1	1
Shear strength parameters	C	70	0	0	475	100
	Φ (°)	35	40	45	30	25

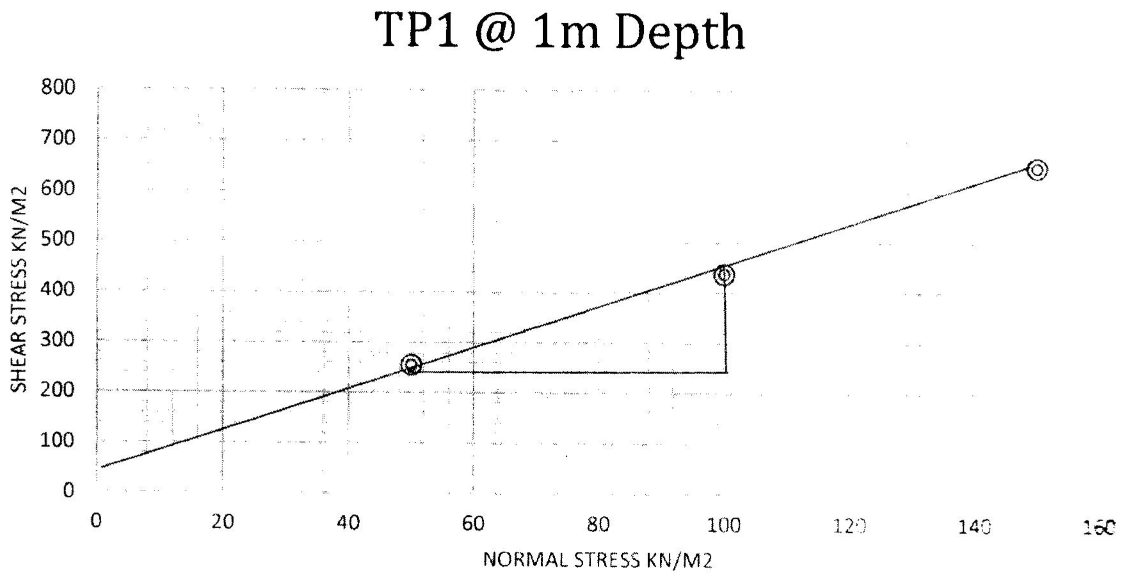
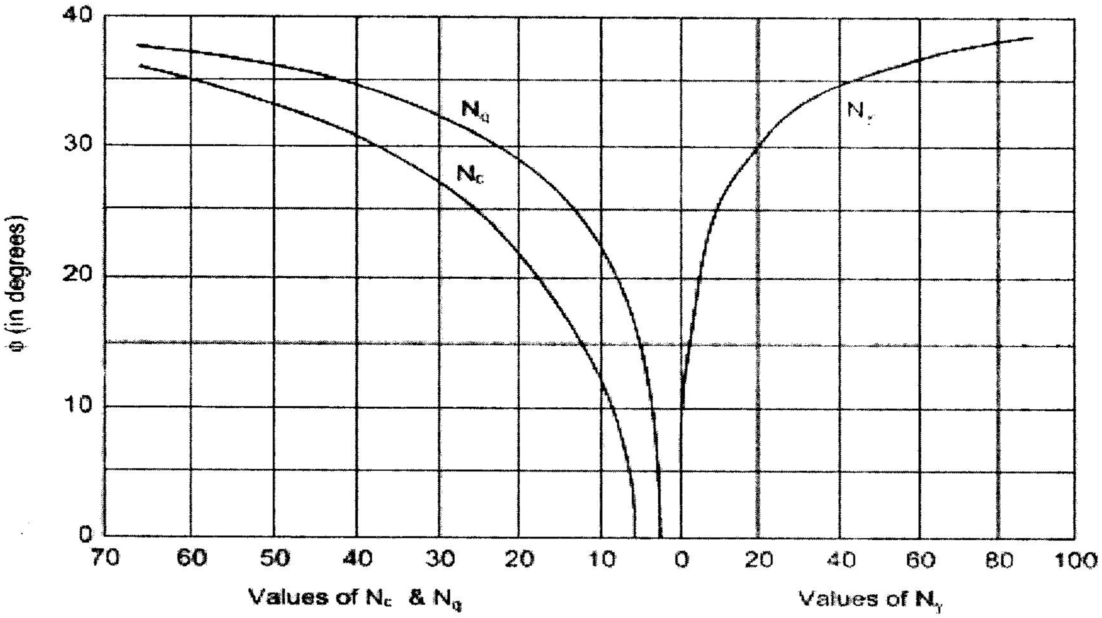


Figure 4.3: Graph of Normal Stress against Shear Strength for TP1

From the graph above,

The cohesion, (c) = 70 and the angle of internal friction, phi (φ) = 35. The shear strength parameters 'c' and phi are low this indicate that the area where soil is weak, the bearing capacity has to be computed to know the type of foundation to adopt before any construction work is carried out in such area because of the low and strength parameters. See appendix F for the results and graphs of the remaing four sampling points.



φ	0°	5°	10°	15°	20°	25°	30°	35°	40°	45°
N_c	5.7	7.3	9.6	12.9	17.7	25.1	37.2	57.8	95.7	172
N_q	1.0	1.6	2.7	4.4	7.4	12.7	22.5	41.4	81.3	173
N_γ	0.0	0.5	1.2	2.5	5.0	9.7	19.7	42.4	100	298

Fig 4.4. Terzaghi's bearing capacity coefficient

The increase in the value of Nc from 5.14 to 5.7 is due to the fact that Terzaghi allowed for frictional effects between the foundation and its supporting soil. The coeffcient N q allows for the surcharge effects due to the soil above the foundation level, and Nγ allows for the size of the footing, B. The effect of Nγ is of little consequence with clays, where the angle of shearing resistance is usually assumed to be the undrained value, φu, and assumed equal to 0°, but it can become significant with wide foundations supported on cohesionless soil, Smith (2014). From figure 5 the values of N_q, N_c, N_γ are obtained with respect to φ and shown below:

Location	Φ	N_c	N_q	N_γ
TP1	35	57.8	41.4	42.4
TP2	40	75.7	81.3	100
TP3	45	172	173	298
TP4	30	37.2	22.5	19.7
TP5	20	25.1	12.7	9.7

Using terzagh's bearing capacity coefficient,

$\Upsilon = 17$, $z = 1m$

$Qu = 1.3cN_c + \Upsilon z\, N_q + 0.4\Upsilon\, N_\gamma$

Where: C: Cohesion of soil, γ : unit weight of soil, z: depth of footing, B: width of footing Nc, Nq, Nr: Terzaghi's bearing capacity factors depend on soil friction angle, φ.

For TP1,

$$qu = 1.3(70)(57.8) + (17)(1)(41.4) + (0.4)(17)(42.4)$$

$$qu = 6251.9psi$$

$$qu = 906.8Kpa$$

For TP2,

C=0, therefore

$$qu = (17)(1)(81.3) + (0.4)(17)(100)$$

$$qu = 2062.1psi$$

$$qu = 299.1Kpa$$

For TP3,

C=0,

$$qu = (17)(1)(173) + (0.4)(17)(298)$$

$$qu = 4967psi$$

$$qu = 702.4Kpa$$

For TP4,

$$qu = (1.3)(475)(37.2) + (17)(1)(22.5) + (0.4)(17)(19.7)$$

$$qu = 23{,}487.4psi$$

$$qu = 3406.6Kpa$$

For TP5,

$$qu = (1.3)(100)(25.1) + (17)(1)(12.7) + (0.4)(17)(9.7)$$

$$qu = 3543.9psi$$

$$qu = 514Kpa$$

Allowable soil bearing capacity, $Qa = qu \div F.S$

Where Factor of safety = 3.

CHAPTER FIVE

CONCLUSION

The following conclusions are deduced from the summary of laboratory tests results.

Results of the soil classification tests conducted on the samples shows that the soils tested classifies as a CH and CL soil, using (USCS) classification. 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.

The shear strength parameters c and phi were slightly high while some are low that indicate the area where soil is weak, bearing capacity has to be computed to know the type of foundation to adopt before any construction work is carried out in such area. The bearing capacity of various sampling points ranges from 299.1kPa to 3406.6kPa.

From the compaction test values shows the range of compressibility of the different location. While the natural moisture content (NMC) of the soil at the time it was collected recorded natural moisture contents that was high, which is not favourable in engineering work.

5.1 RECOMMENDATIONS

Futher investigation should be carried out around the area so as to determine the bearing capacity for other types of foundation and it is also adviced to stabilize due to high rate of settlement. Mechanical stabilization should be adopted or other suitable stabilization method for clayey soil.

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APPENDIX A

PARTICLE SIZE DISTRIBUTION

RESULTS FOR PARTICLE SIZE DISTRIBUTION AT 1m DEPTH

TP 1			
Sieve Size	**Weight. Retained**	**% Retained**	**% Passing**
9.50	2.00	0.40	99.60
4.75	10.70	2.14	97.50
2.36	16.50	3.30	94.20
1.18	27.20	5.44	88.70
0.600	44.40	8.88	79.80
0.300	64.60	12.92	66.90
0.150	57.80	11.56	55.40
0.75	33.40	6.68	48.70
Total	256.60		

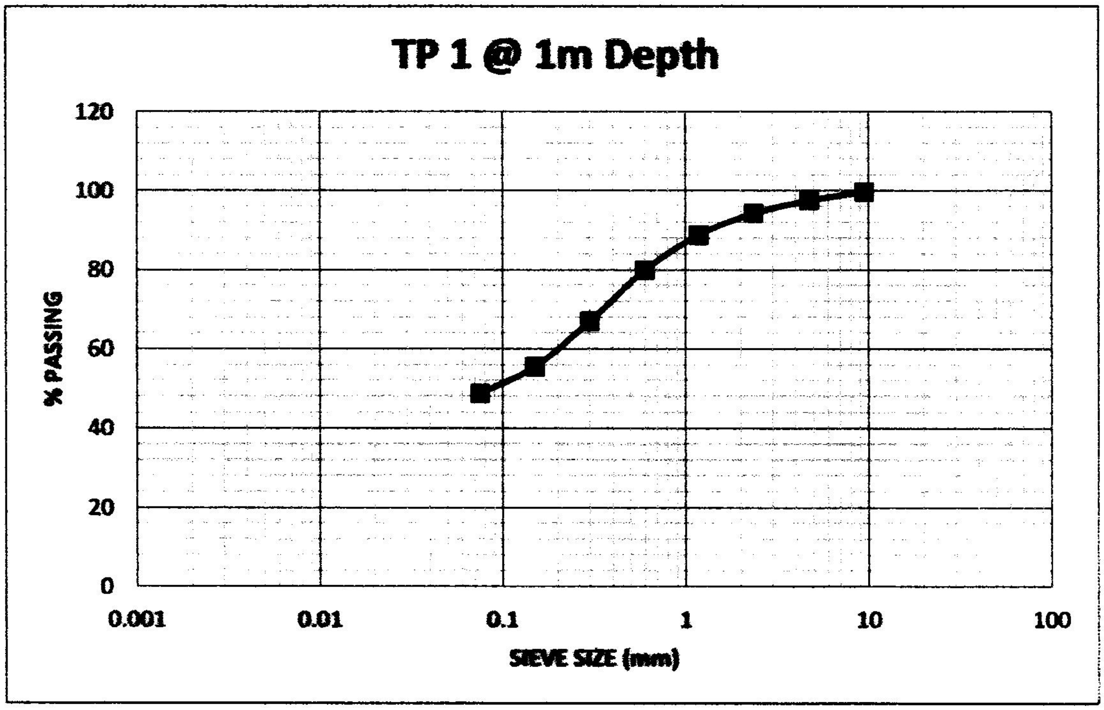
TP 2			
Sieve Size	**Weight. Retained**	**% Retained**	**% Passing**
9.50	0.00	0.00	86.40
4.75	5.80	1.16	77.50
2.36	15.40	3.08	69.40
1.18	29.80	5.96	62.20
0.600	45.40	9.08	54.90
0.300	52.00	10.40	70.30
0.150	44.60	8.92	61.40
0.75	26.50	5.30	56.10
Total	219.50		

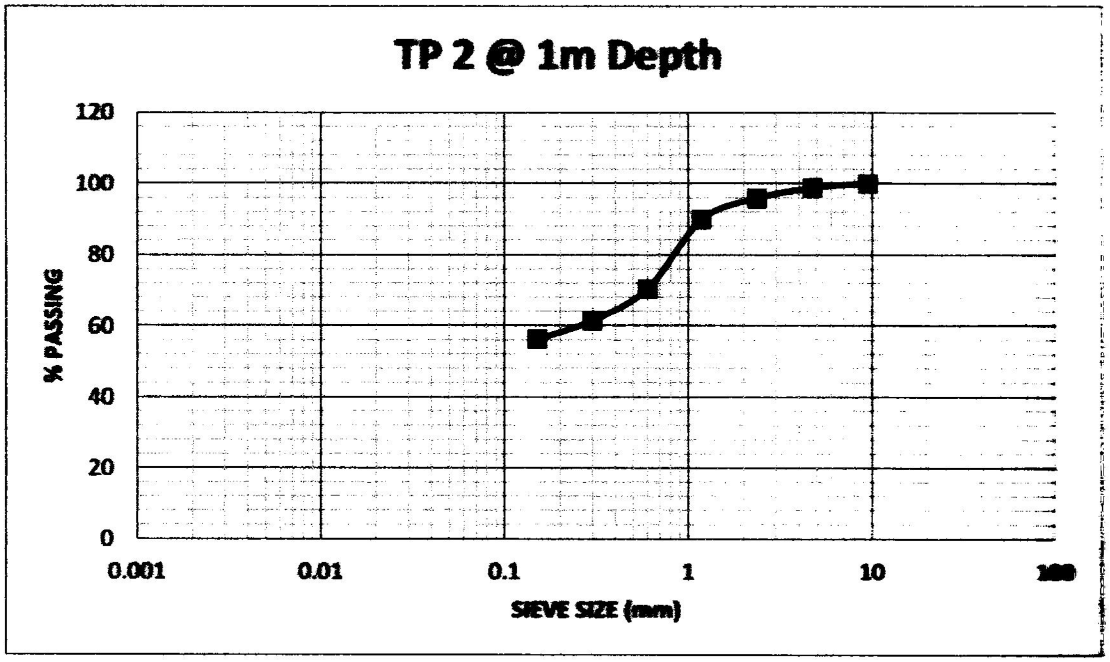
TP 3			
Sieve Size	Weight. Retained	% Retained	% Passing
9.50	0.00	0.00	100.00
4.75	6.00	1.20	98.80
2.36	24.50	4.90	93.90
1.18	39.40	7.88	86.00
0.600	56.30	11.26	74.80
0.300	55.50	11.10	63.70
0.150	48.50	9.70	54.00
0.75	30.20	6.04	47.90
Total	260.40		

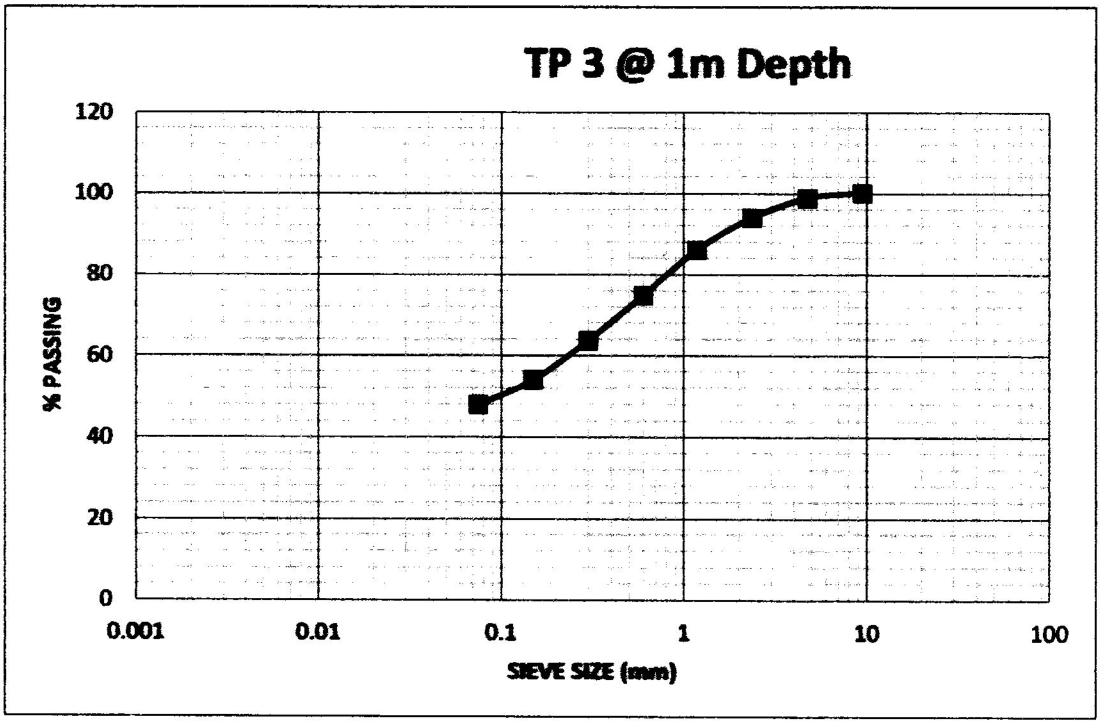
TP 4			
Sieve Size	Weight. Retained	% Retained	% Passing
9.50	2.40	0.48	99.50
4.75	7.70	1.54	98.00
2.36	22.50	4.50	93.50
1.18	40.20	8.04	85.40
0.600	56.70	11.34	74.10
0.300	61.90	12.38	61.70
0.150	50.60	10.12	51.60
0.75	28.70	5.74	45.90
Total	270.70		

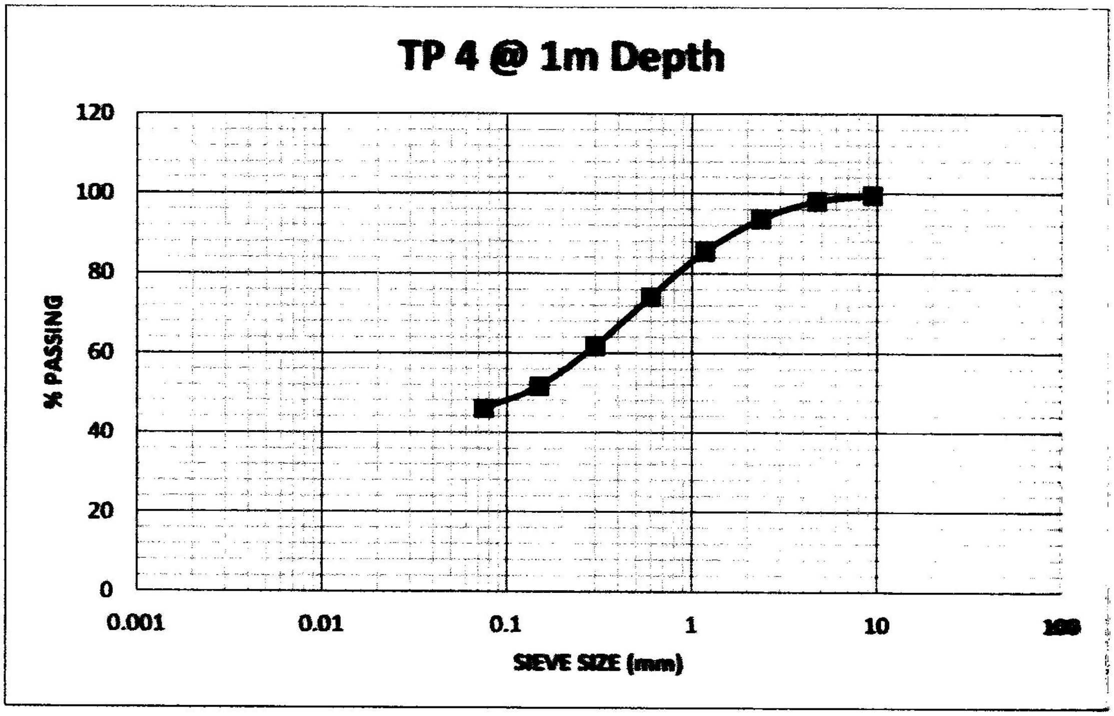
TP 5			
Sieve Size	Weight. Retained	% Retained	% Passing
9.50	0.00	0.00	100.00
4.75	0.00	0.00	100.00
2.36	5.20	1.04	99.00
1.18	20.40	4.08	94.90
0.600	33.40	6.68	88.20
0.300	44.00	8.80	79.40
0.150	38.50	7.70	71.70
0.75	17.70	3.54	68.20
Total	159.20		

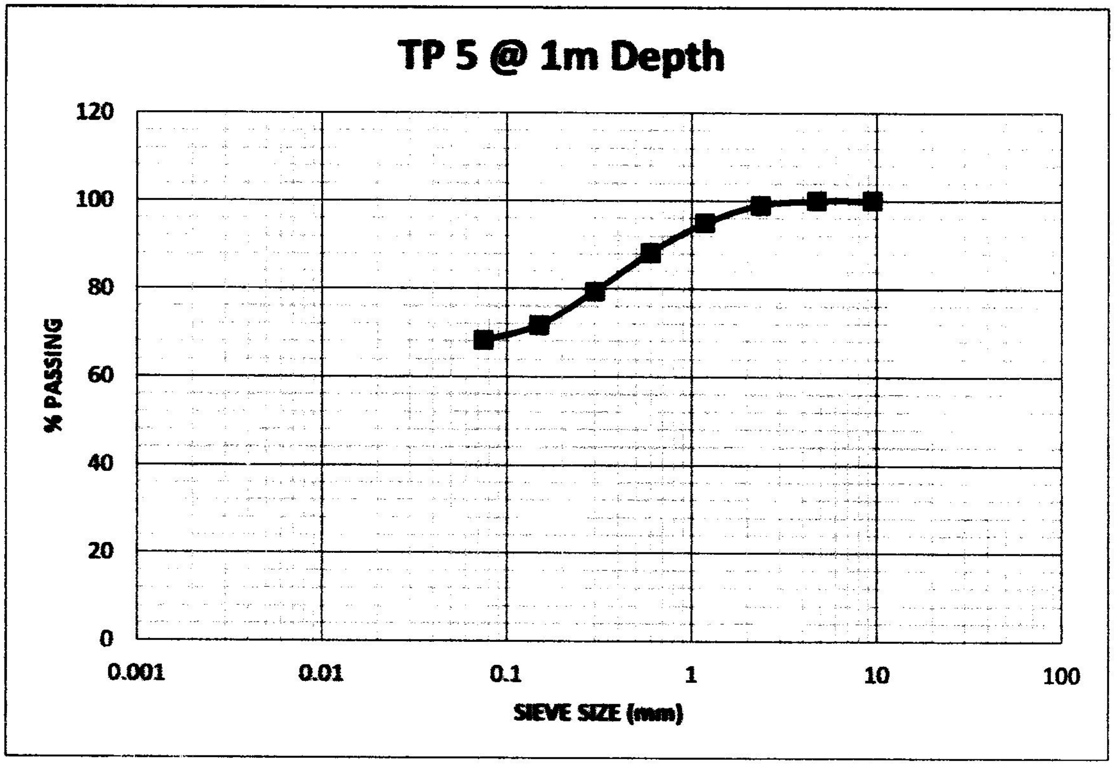
GRAPH OF PARTICLE SIZE DISTRIBUTION






TP 3 @ 1m Depth
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


TP 4 @ 1m Depth
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)


TP 5 @ 1m Depth
120
100
80
60
40
20
0
% PASSING
0.001
0.01
0.1
1
10
100
SIEVE SIZE (mm)

APPENDIX B
NATURAL MOISTURE CONTENT

RESULTS FOR NATURAL MOISTURE CONTENT @ 1m DEPTH

TRIAL PIT 1		
Trial no.	1	2
weight of container (g)	26.2	26.9
weight of container + soil+ water (g)	80.2	78.1
weight of container + dry soil (g)	72.1	70.1
Moisture content, M.C	17.6	18.5

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B-C}{C-A} \times 100 = 18.1$$

TRIAL PIT 2		
Trial no.	1	2
weight of container (g)	20.1	20.1
weight of container + soil+ water (g)	71.7	58.9
weight of container + dry soil (g)	62.4	52.8
Moisture content, M.C	22.0	18.7

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B-C}{C-A} \times 100 = 20.3$$

TRIAL PIT 3		
Trial no.	1	2
weight of container (g)	26.6	26.7
weight of container + soil+ water (g)	76.7	79.2
weight of container + dry soil (g)	65.2	71.1
Moisture content, M.C	29.8	18.2

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 24.0$$

TRIAL PIT 4		
Trial no.	1	2
weight of container (g)	10.0	9.9
weight of container + soil+ water (g)	60.0	57.4
weight of container + dry soil (g)	52.5	49.7
Moisture content, M.C	17.6	19.3

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B - C}{C - A} \times 100 = 18.5$$

TRIAL PIT 5		
Trial no.	1	2
weight of container (g)	19.7	26.7
weight of container + soil+ water (g)	60.2	64.7
weight of container + dry soil (g)	53.9	59.3
Moisture content, M.C	18.4	16.6

A = weight of container
B = weight of container + water + soil
C = weight of container + dry soil

$$M.C = \frac{B-C}{C-A} \times 100 = 17.5$$

APPENDIX C

COMPACTION

RESULTS FOR COMPACTION TEST

TRIAL PIT 1

Trial No.	1	2	3	4
Weight of mould + soil (g)	4800	5000	5200	5100
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1650	1850	2050	1950
wet density of soil (Kg/m^3)	1.7	1.9	2.1	2.0
Container identification no.	A_1	A_2	A_3	A_4
weight of container (g)	17.9	17.7	12.5	13.3
weight of wet soil + container (g)	68.5	79.9	75.2	74.9
weight of dry soil + container (g)	62.9	71.0	64.3	62.5
weight of water (g)	5.60	8.90	10.90	12.40
weight of dry soil (g)	45.0	53.3	51.8	50.2
moisture content	12.4	16.7	21.0	24.7
dry density	1.47	1.58	1.69	1.56

TRIAL PIT 2

Trial No.	1	2	3	4
Weight of mould + soil (g)	4950	5150	5250	5100
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1800	2000	2100	1950
wet density of soil (Kg/m^3)	1.80	2.00	2.10	1.95
Container identification no.	B_1	B_2	B_3	B_4
weight of container (g)	20.60	13.80	11.90	14.10
weight of wet soil + container (g)	70.90	63.20	61.70	65.70
weight of dry soil + container (g)	65.00	56.60	53.70	56.20
weight of water (g)	5.90	6.60	8.00	9.50
weight of dry soil (g)	44.40	42.80	42.00	41.70
moisture content	13.30	15.40	19.00	22.80
dry density	1.59	1.69	1.76	1.59

TRIAL PIT 3

Trial No.	1	2	3	4
Weight of mould + soil (g)	5350	5500	5900	5850
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1550	1700	2100	2050
wet density of soil (Kg/m^3)	1.55	1.70	2.10	2.05
Container identification no.	C_1	C_2	C_3	C_4
weight of container (g)	20.00	21.40	12.00	11.80
weight of wet soil + container (g)	93.70	70.40	58.20	56.60
weight of dry soil + container (g)	88.80	65.30	51.50	49.20
weight of water (g)	4.90	5.10	6.70	7.40
weight of dry soil (g)	68.80	43.70	39.50	37.40
moisture content	7.10	11.60	16.90	19.80
dry density	1.45	1.52	1.79	1.71

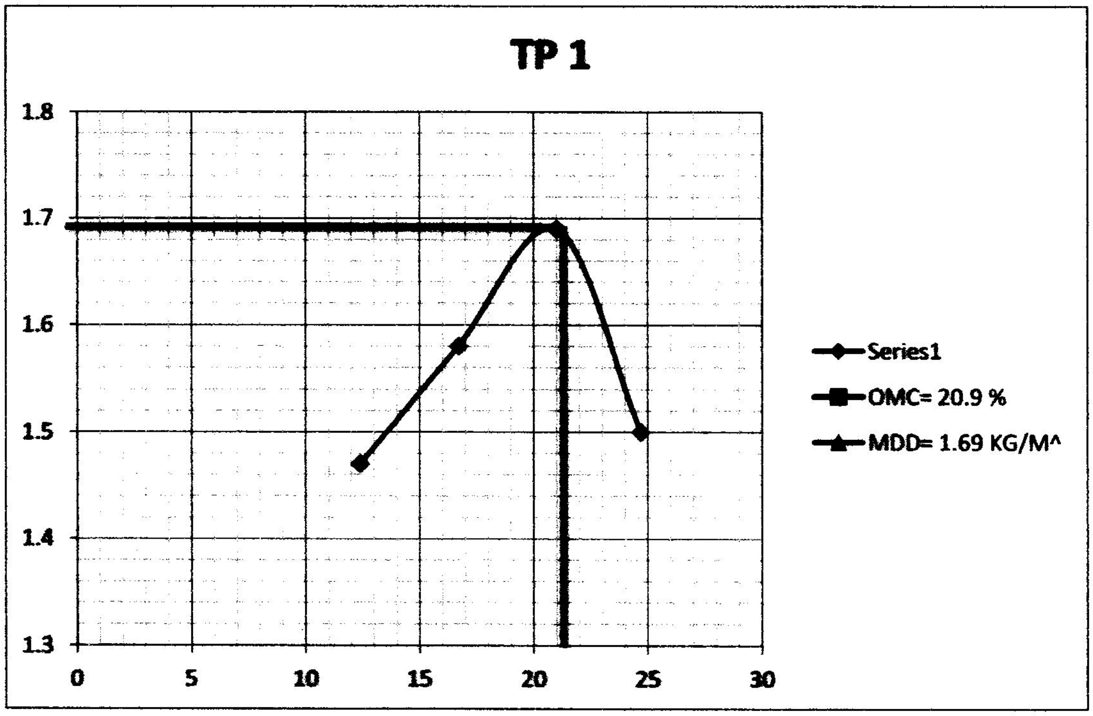
TRIAL PIT 4

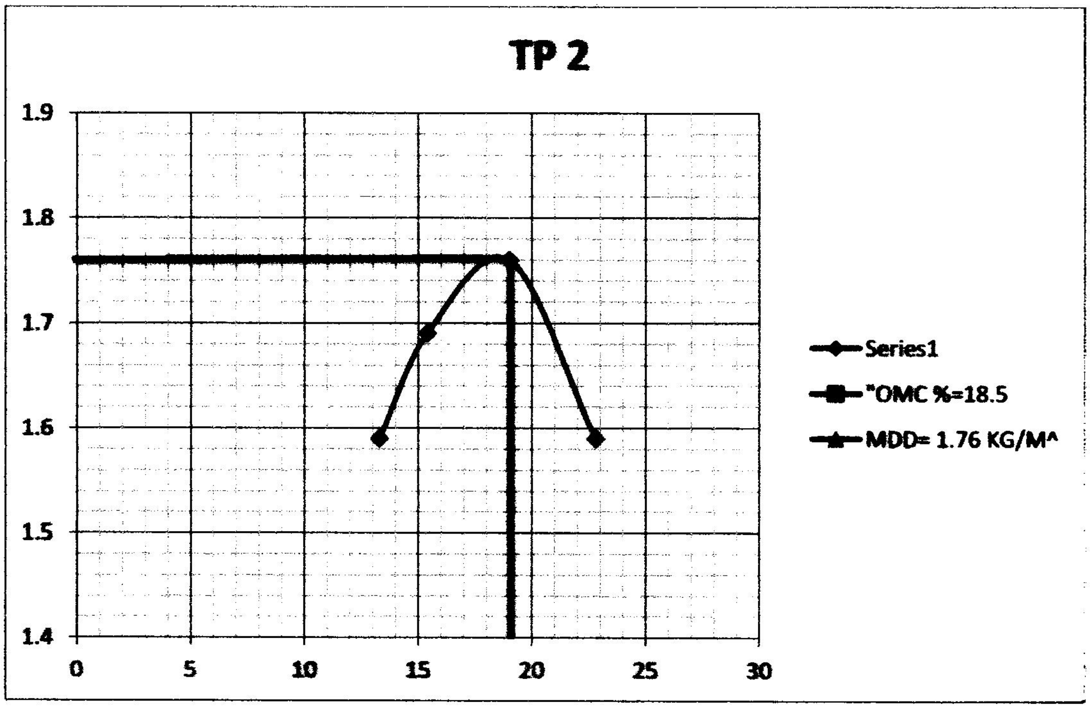
Trial No.	1	2	3	4
Weight of mould + soil (g)	4850	5050	5250	5150
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1700	1900	2100	2000
wet density of soil (Kg/m^3)	1.70	1.90	2.10	2.00
Container identification no.	D_1	D_2	D_3	D_4
weight of container (g)	26.60	11.60	17.70	12.10
weight of wet soil + container (g)	80.20	52.40	70.00	58.50
weight of dry soil + container (g)	75.80	47.10	63.60	51.20
weight of water (g)	4.40	5.30	6.50	7.30
weight of dry soil (g)	49.20	45.00	45.90	39.10
moisture content	8.90	11.80	14.20	18.70
dry density	1.52	1.71	1.83	1.68

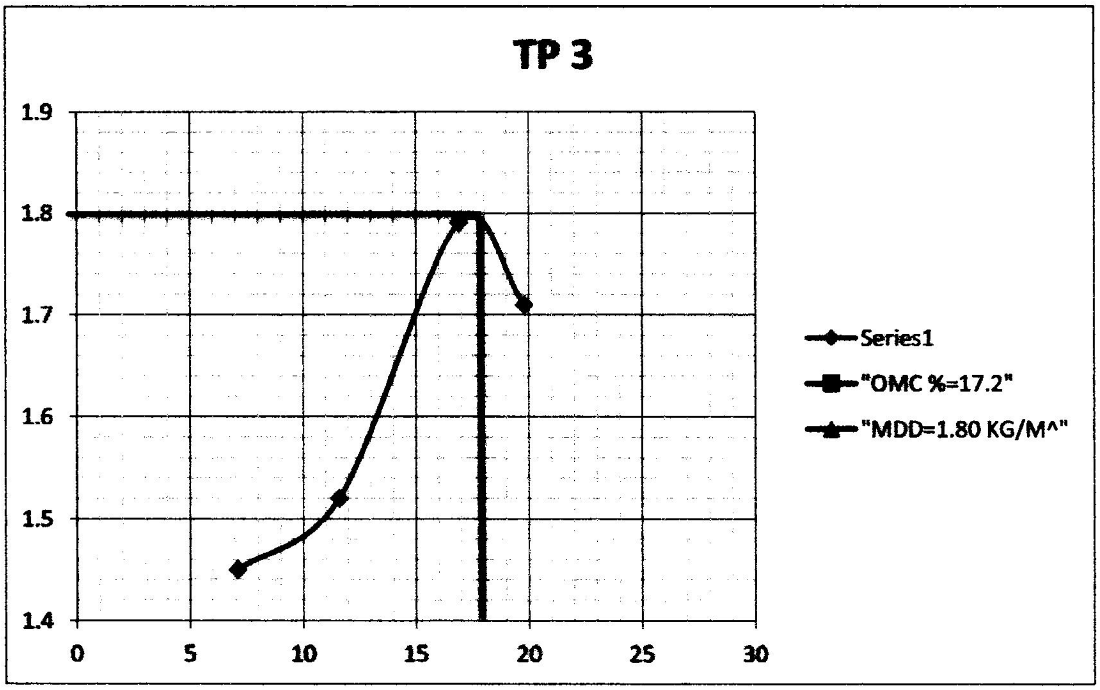
TRIAL PIT 5

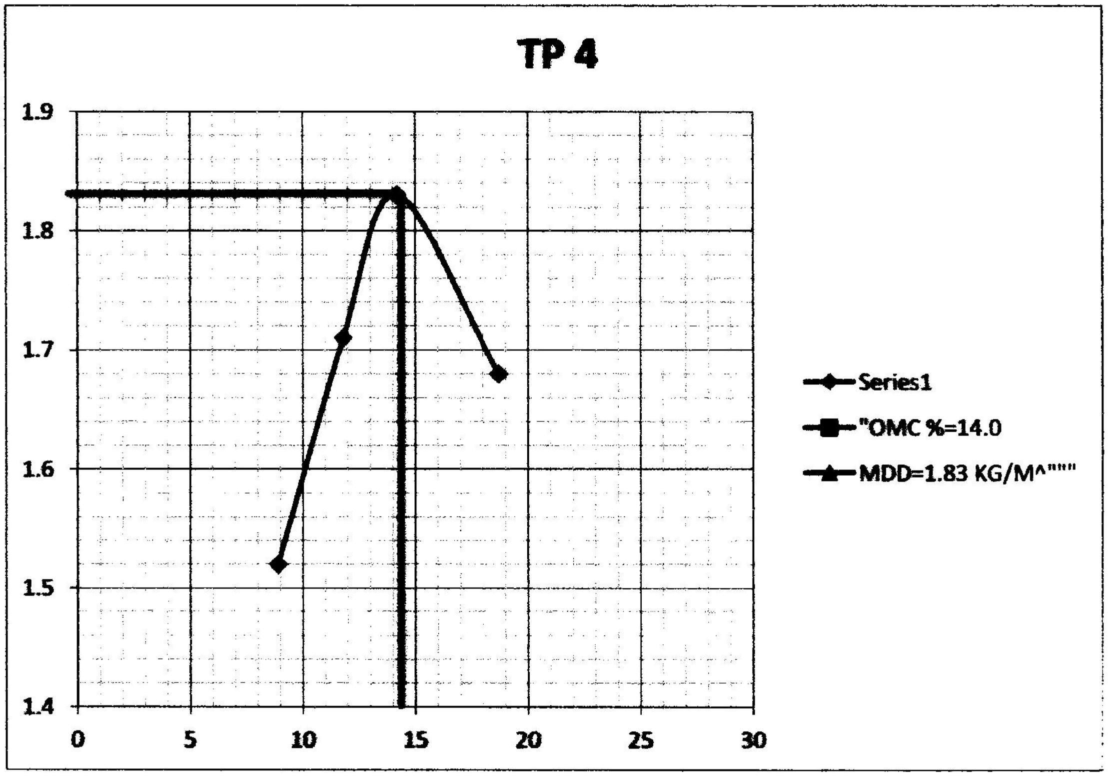
Trial No.	1	2	3	4
Weight of mould + soil (g)	4600	4700	4900	4850
weight of empty mould (g)	3150	3150	3150	3150
weight of wet soil	1450	1550	1750	1700
wet density of soil (Kg/m³)	1.45	1.55	1.75	1.70
Container identification no.	E_1	E_2	E_3	E_4
weight of container (g)	10.00	26.70	26.60	26.80
weight of wet soil + container (g)	78.60	79.70	68.80	69.00
weight of dry soil + container (g)	75.00	75.40	63.40	62.20
weight of water (g)	3.60	4.30	5.40	6.80
weight of dry soil (g)	65.00	48.70	36.80	35.60
moisture content	5.50	8.80	14.70	19.10
dry density	1.37	1.42	1.53	1.43

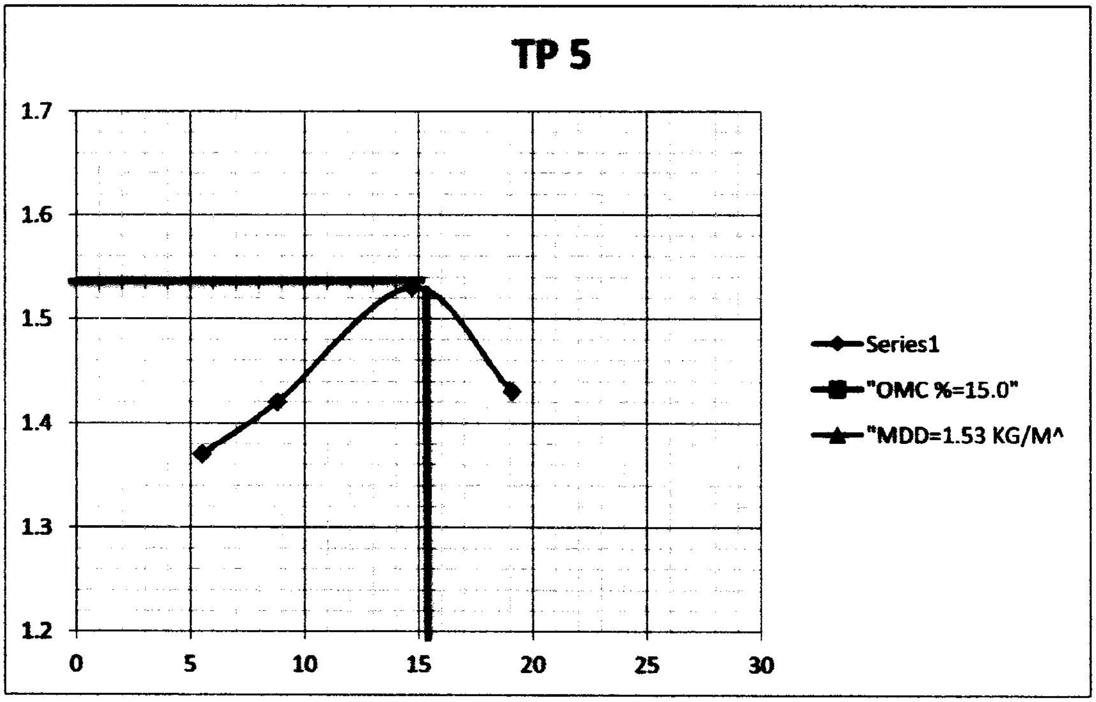
GRAPHS OF COMPACTION TEST






TP 3
1.9
1.8
1.7
1.6
1.5
1.4
0
5
10
15
20
25
30
Series1
"OMC %=17.2"
"MDD=1.80 KG/M^"


TP 4
1.9
1.8
1.7
1.6
1.5
1.4
0
5
10
15
20
25
30
Series1
"OMC %=14.0
MDD=1.83 KG/M^"""


TP 5
1.7
1.6
1.5
1.4
1.3
1.2
0
5
10
15
20
25
30
Series1
"OMC %=15.0"
"MDD=1.53 KG/M^

APPENDIX D

SPECIFIC GRAVITY

RESULTS FOR SPECIFIC GRAVITY TEST

TRIAL PIT 1		
Trial no.	1	2
weight of empty density bottle (g)	23.1	25.7
weight of density bottle + dry soil (g)	53.7	52.6
weight of density bottle + soil + water (g)	92.1	92.9
weight of density bottle + water (g)	74.8	77.8
Specific Gravity, S.G	2.30	2.28

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

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.29$$

TRIAL PIT 2		
Trial no.	1	2
weight of empty density bottle (g)	23.8	25.8
weight of density bottle + dry soil (g)	48.1	49.7
weight of density bottle + soil + water (g)	92.1	93.4
weight of density bottle + water (g)	78.3	79.0
Specific Gravity, S.G	2.31	2.52

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

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

TRIAL PIT 3

Trial no.	1	2
weight of empty density bottle (g)	25.8	25.8
weight of density bottle + dry soil (g)	49.3	48.5
weight of density bottle + soil + water (g)	92.2	91.1
weight of density bottle + water (g)	78.1	78.0
Specific Gravity, S.G	2.50	2.36

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

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.43$$

TRIAL PIT 4

Trial no.	1	2
weight of empty density bottle (g)	26.4	26.4
weight of density bottle + dry soil (g)	48.9	52.2
weight of density bottle + soil + water (g)	90.8	94.9
weight of density bottle + water (g)	77.6	79.9
Specific Gravity, S.G	2.42	2.39

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

$$S.G = \frac{W2 - W1}{(W4 - W1) - (W3 - W2)} = 2.40$$

TRIAL PIT 5		
Trial no.	1	2
weight of empty density bottle (g)	23.8	26.9
weight of density bottle + dry soil (g)	48.3	51.3
weight of density bottle + soil + water (g)	92.4	93.6
weight of density bottle + water (g)	78.2	79.4
Specific Gravity, S.G	2.38	2.39

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

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

APPENDIX E
CONSISTENCY LIMIT

RESULT FOR CONSISTENCY LIMIT TEST BY CASSAGRANDE METHOD

TRIAL PIT 1							
Trial no.	1	2	3	4			
no. of blows	49	36	22	14	PLASTIC LIMIT		
container identification no.	A_1	A_2	A_3	A_4	A_5	A_6	
weight of empty container (g)	19.8	26.7	19.8	13	9.8	11.6	
weight of container + wet soil (g)	45.1	54.1	51.3	45	27.9	32.7	
weight of container + dry soil (g)	38.8	46.5	42.3	36.7	25.1	29.4	
weight of water (g)	6.3	7.6	9.0	10.3	2.8	3.3	
weight of dry soil (g)	19.0	19.8	22.5	26.7	15.3	17.8	
moisture content	33.2	38.4	40.0	43.5	18.3	18.5	PL = 18.4%

TRIAL PIT 2							
Trial no.	1	2	3	4			
no. of blows	48	36	23	13	PLASTIC LIMIT		
container identification no.	B_1	B_2	B_3	B_4	B_5	B_6	
weight of empty container (g)	20.2	19.8	18.7	21.9	11.6	9.8	
weight of container + wet soil (g)	41.3	45.8	45.8	51	34.4	28.4	
weight of container + dry soil (g)	35.1	37.8	36.9	41.0	30.2	24.8	
weight of water (g)	6.2	8.0	8.9	10.0	4.2	3.6	
weight of dry soil (g)	14.9	18.0	18.2	19.1	18.6	15	
moisture content	41.6	44.4	48.9	52.4	22.6	24	PL = 23.3%

TRIAL PIT 3

Trial no.	1	2	3	4			
no. of blows	45	34	22	13	PLASTIC LIMIT		
container identification no.	C_1	C_2	C_3	C_4	C_5	C_6	
weight of empty container (g)	8.2	16.6	9.7	16.3	7.1	12.1	
weight of container + wet soil (g)	28.1	44.4	39.6	49.1	24.7	37	
weight of container + dry soil (g)	22.8	37.1	31.2	39.3	21.8	33.8	
weight of water (g)	5.3	7.3	8.4	9.8	2.9	3.2	
weight of dry soil (g)	16.6	20.5	21.5	23.0	14.7	21.7	
moisture content	31.9	35.6	39.1	42.6	19.7	14.7	PL = 17.2%

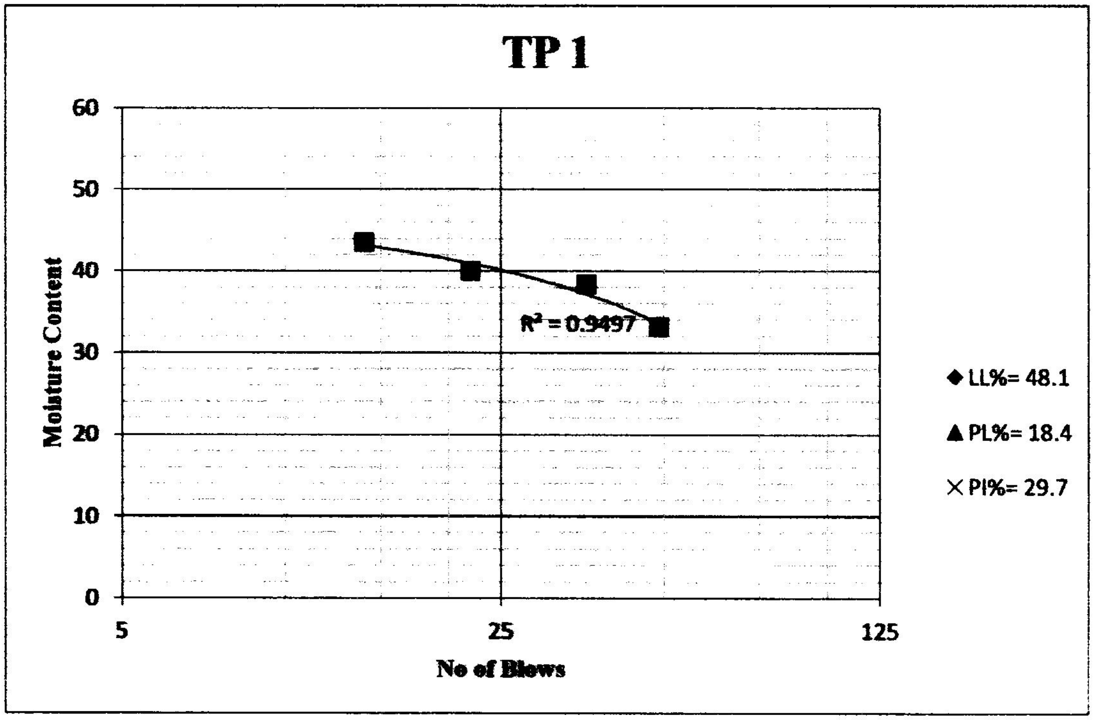
TRIAL PIT 4

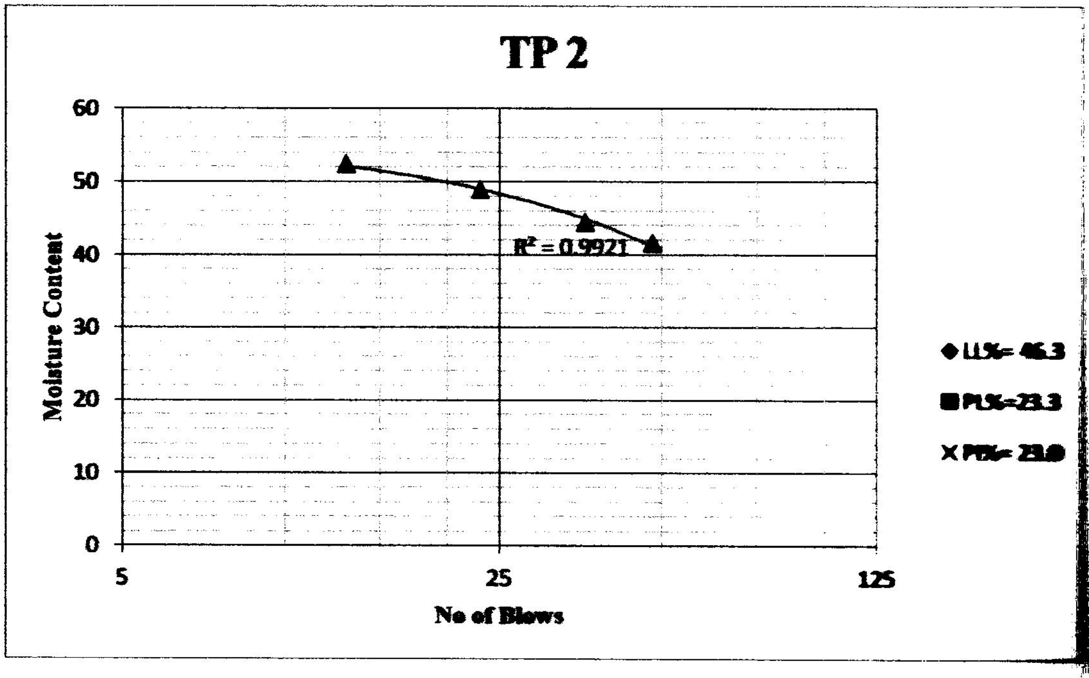
Trial no.	1	2	3	4			
no. of blows	48	37	21	12	PLASTIC LIMIT		
container identification no.	D_1	D_2	D_3	D_4	D_5	D_6	
weight of empty container (g)	16.1	9.8	8	14	18.6	10.5	
weight of container + wet soil (g)	43.7	39.3	41.9	49.6	35.2	28.6	
weight of container + dry soil (g)	36.8	32.1	33.0	39.5	32.2	25.5	
weight of water (g)	5.9	7.2	8.9	10.1	3	3.1	
weight of dry soil (g)	20.7	22.3	25.0	25.5	13.6	15	
moisture content	28.5	32.3	35.6	39.6	22.1	20.7	PL = 21.4%

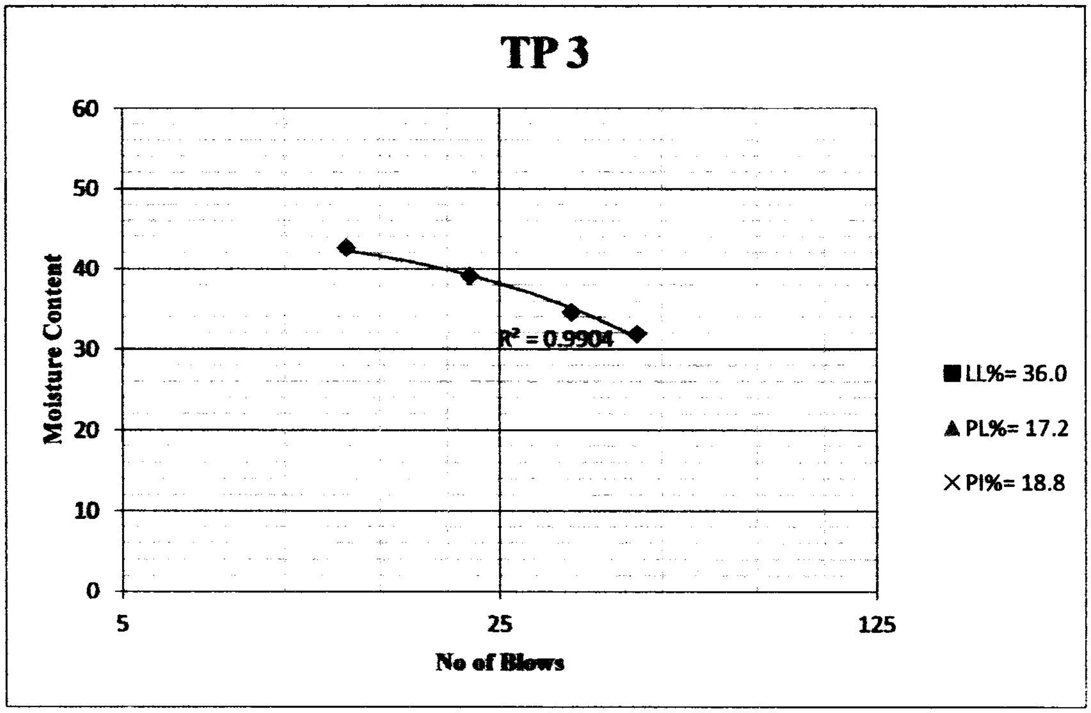
TRIAL PIT 5

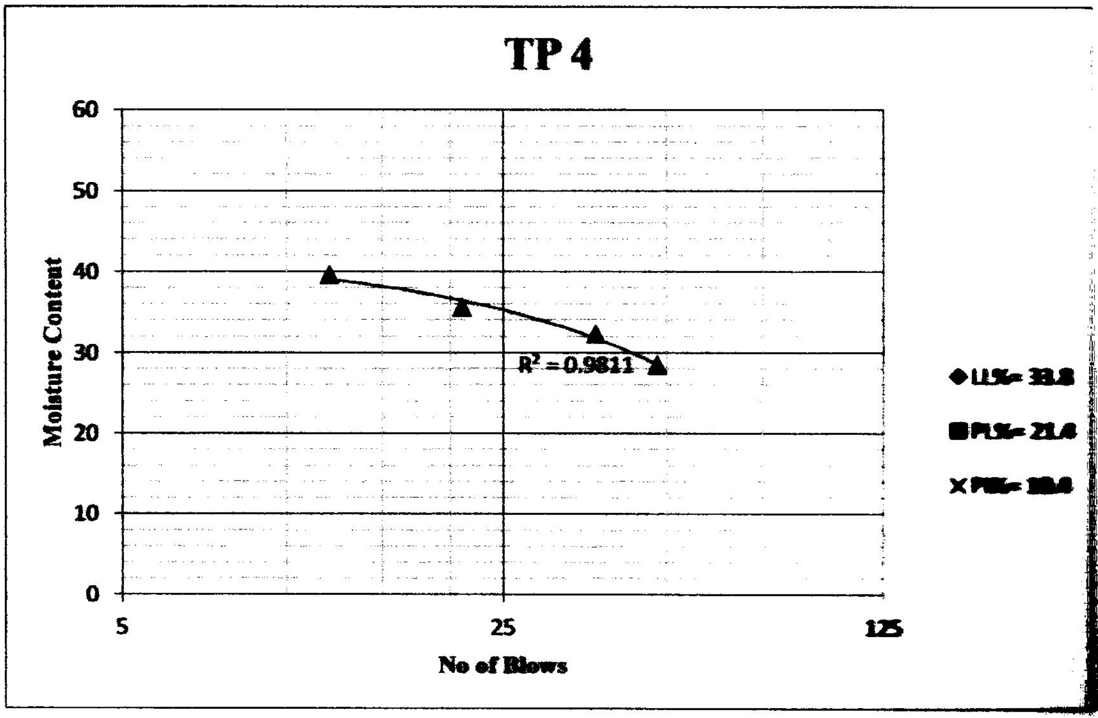
Trial no.	1	2	3	4			
no. of blows	48	38	23	13	**PLASTIC LIMIT**		
container identification no.	D_1	D_2	D_3	D_4	D_5	D_6	
weight of empty container (g)	19.8	27.6	14.2	16.4	11.5	13.3	
weight of container + wet soil (g)	37.8	50.3	38.3	41.9	31.4	34.5	
weight of container + dry soil (g)	34.0	44.5	30.4	32.3	27.5	30.4	
weight of water (g)	5.5	6.8	7.9	8.6	3.9	4.1	
weight of dry soil (g)	14.2	16.9	16.2	15.9	16	17.1	
moisture content	38.7	42.8	48.8	54.1	24.4	24	PL = 24.2%

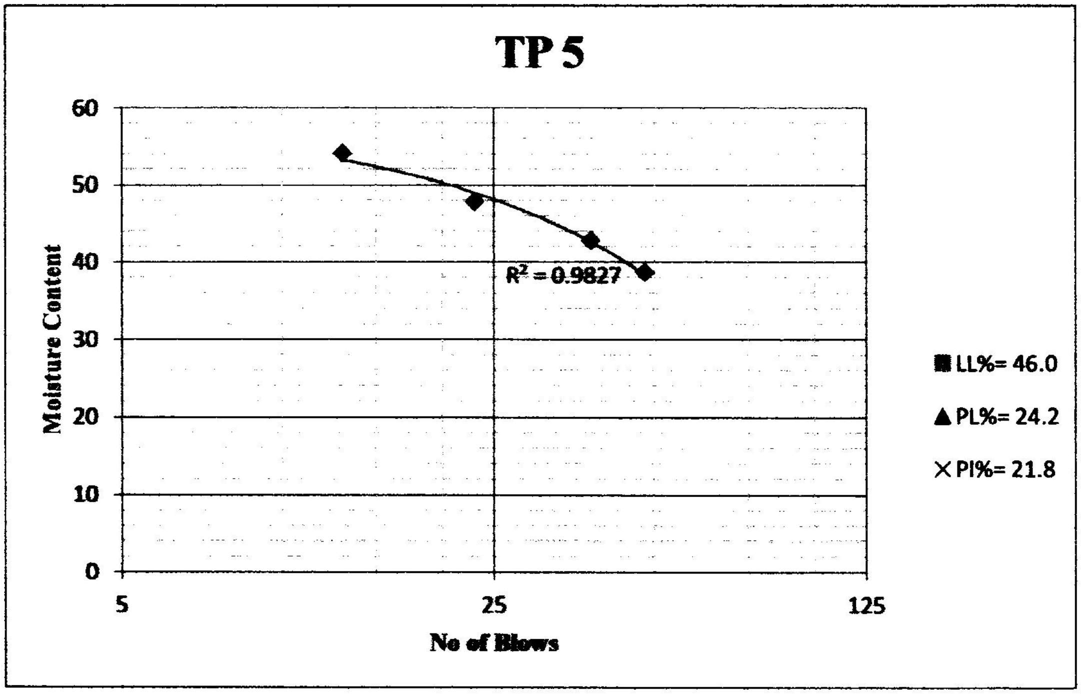
GRAPHS OF CONSISITENCY LIMIT TEST






TP 3
Moisture Content
60
50
40
30
20
10
0
5
25
125
No of Blows
R² = 0.9904
■ LL%= 36.0
▲ PL%= 17.2
× PI%= 18.8


TP 4
Moisture Content
60
50
40
30
20
10
0
5
25
125
No of Blows
R² = 0.9811
◆ LL%= 33.8
■ PL%= 21.4
× PI%= [illegible]


TP 5
Moisture Content
60
50
40
30
20
10
0
5
25
125
No of Blows
R² = 0.9827
■ LL%= 46.0
▲ PL%= 24.2
× PI%= 21.8

APPENDIX F
DIRECT SHEAR

RESULTS FOR DIRECT SHEAR TEST

Test no:	Normal load (KN)	Normal force (KN)	Normal strength (KN/m^2)	MAX DR DW	Shear force (KN)	Shear stress (KN/m^2)
1	50	0.4905	136.3	102	0.918	255
2	100	0.981	272.5	175	1.575	437.5
3	150	1.4715	408.8	260	2.34	650

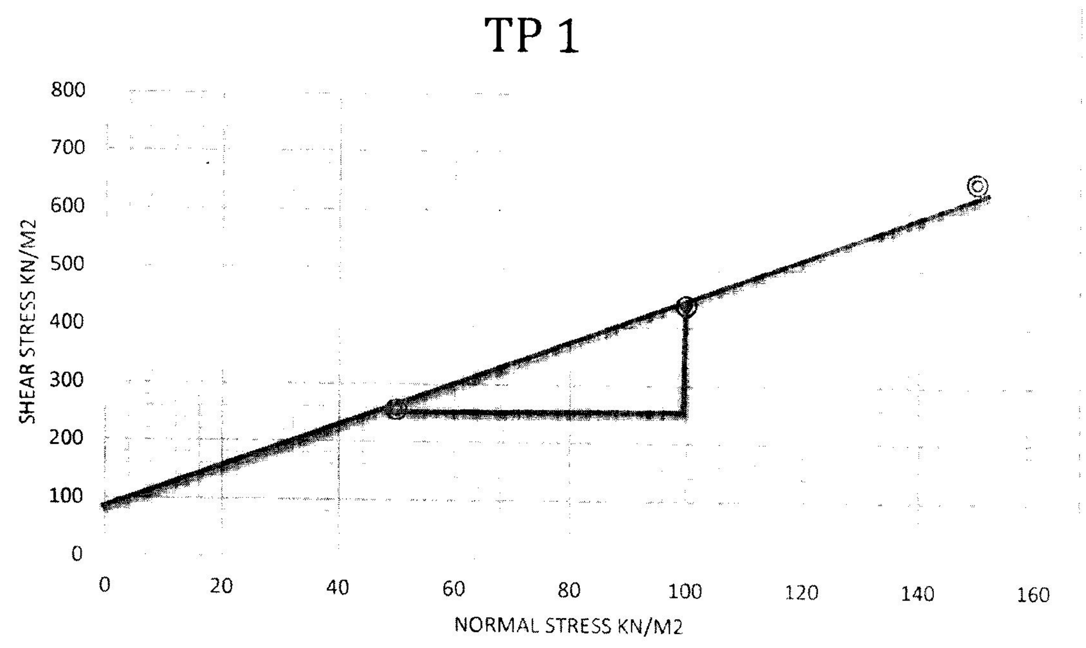
Test no:	Normal load (KN)	Normal force (KN)	Normal strength (KN/m^2)	MAX DR DW	Shear force (KN)	Shear stress (KN/m^2)
1	50	0.4905	136.3	118	1.602	295
2	100	0.981	272.5	202	1.818	505
3	150	1.4715	408.8	398	3.582	995

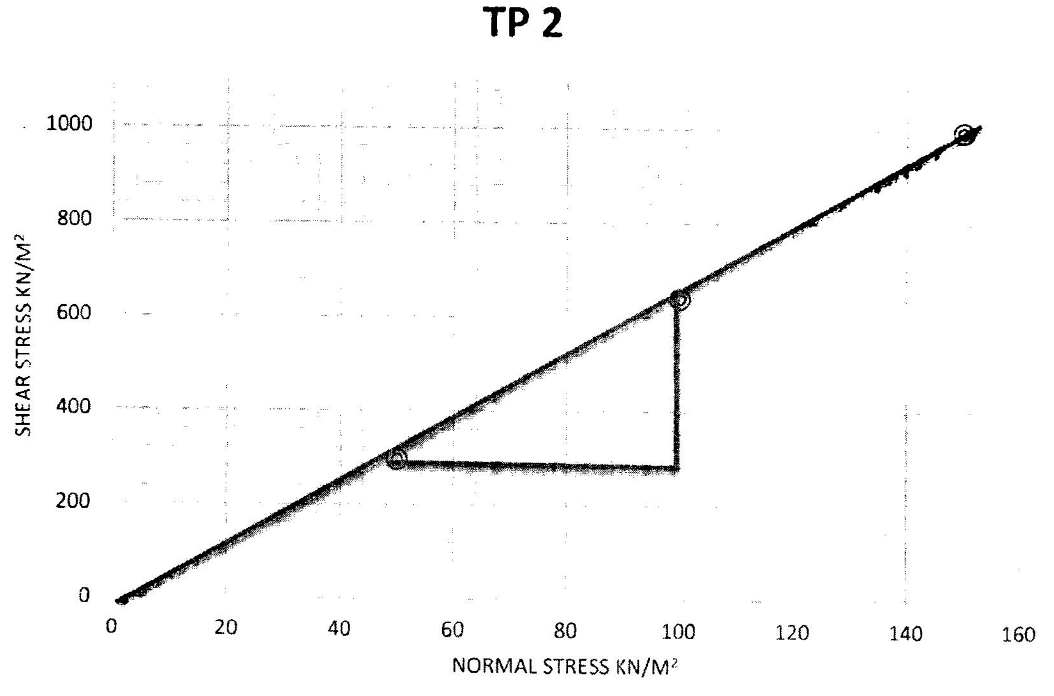
Test no:	Normal load (KN)	Normal force (KN)	Normal strength (KN/m^2)	MAX DR DW	Shear force (KN)	Shear stress (KN/m^2)
1	50	0.4905	136.3	215	1.935	537.5
2	100	0.981	272.5	445	4.005	1112.5
3	150	1.4715	408.8	686	6.174	1715

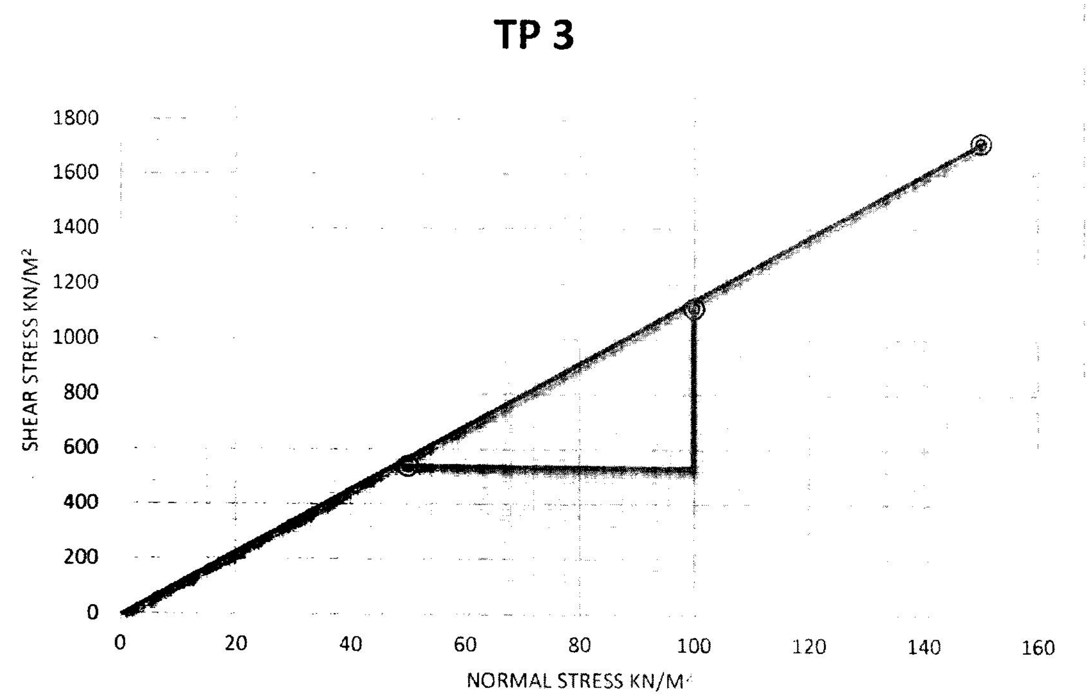
Test no:	Normal load (KN)	Normal force (KN)	Normal strength (KN/m^2)	MAX DR DW	Shear force (KN)	Shear stress (KN/m^2)
1	50	0.4905	136.3	318	2.862	795
2	100	0.981	272.5	425	3.825	1062.5
3	150	1.4715	408.8	603	5.427	1507.5

Test no:	Normal load (KN)	Normal force (KN)	Normal strength (KN/m^2)	MAX DR DW	Shear force (KN)	Shear stress (KN/m^2)
1	50	0.4905	136.3	97	2.007	242.5
2	100	0.981	272.5	155	1.395	387.5
3	150	1.4715	408.8	223	0.873	557.5

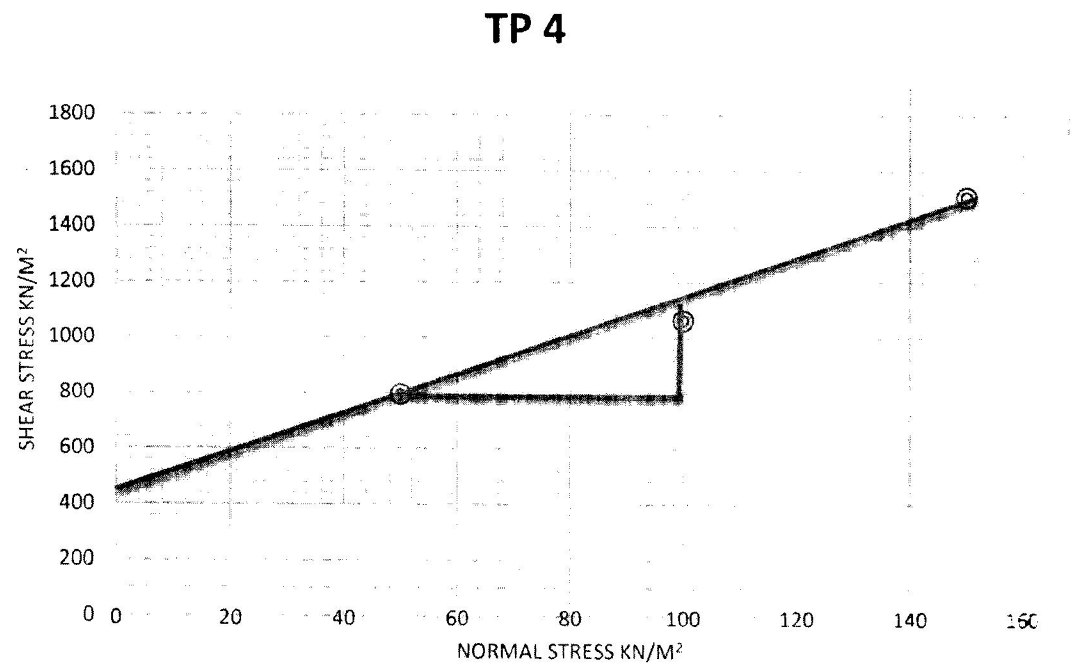
GRAPHS OF DIRECT SHEAR



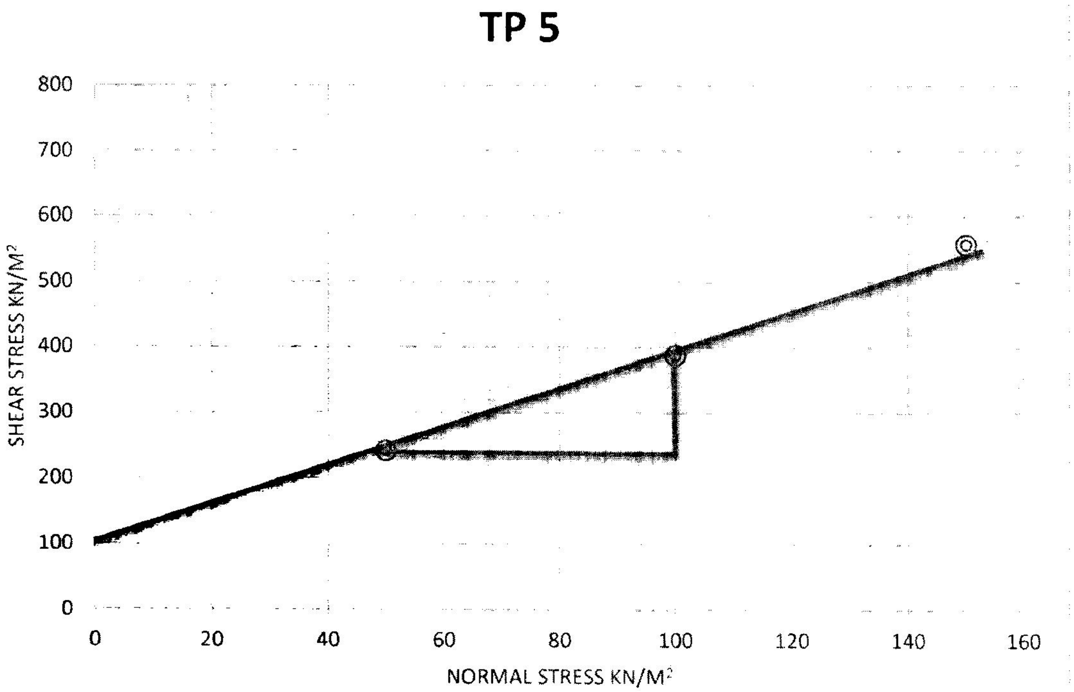



TP 3

1800
1600
1400
1200
1000
800
600
400
200
0
SHEAR STRESS KN/M²
0
20
40
60
80
100
120
140
160
NORMAL STRESS KN/M²


TP 4

1800
1600
1400
1200
1000
800
600
400
200
0
SHEAR STRESS KN/M²
0
20
40
60
80
100
120
140
160
NORMAL STRESS KN/M²


TP 5
SHEAR STRESS KN/M2
800
700
600
500
400
300
200
100
0
0
20
40
60
80
100
120
140
160
NORMAL STRESS KN/M2