ASSESSMENT OF GEOTECHNICAL PROPERTIES OF SELECTED LATERITIC SOIL AS FILL MATERIALS FOR EARTH DAM CONSTRUCTION

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ABSTRACT

This study presents assessment of geotechnical properties of selected lateritic soil as fill materials for earth dam construction. Standard tests were carried out on 27 lateritic soil samples collected from selected borrowed pits in Maikunkele, Pyata and Lapai Gwari. The three locations are all in the surbubs of Minna. Niger State. Particle size analysis revealed that ten of the lateritic samples were gap-graded, while six were poorly graded. The compaction characteristics; MDD ranged between 1.65 to 1.99 g/cm^3, with the OMC range of 9.8 – 21.2%. The hydraulic conductivity (k) ranged from 10^{-5} cm/s to 10^{-9} cm/s. The work further revealed average values of confined compressive strength (*CCS*), Cohesion (C), angle of internal friction (ϕ) as 397.7 kN/m^2, 99.49 kN/m^2 and 20.6^0 respectively. The soil is impervious and very good embankment fill material to serve as dam core and embankment fill due to its impermeable nature. The values of the geotechnical properties are within the threshold for suitable as fill material. Therefore, the test samples were confirmed suitable for use as fill material for earth dam construction.

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Abbreviations, Glossary and Symbols

AASHTO	American Association for State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BS	British Standard
BSH	British Standard Heavy
CCS	Confined Compressive Strength
CH	Clay of High Plasticity
CL	Clay Soil
LL	Liquid Limit
MDD	Maximum Dry Density
MH	Inorganic clayey silt, elastic silt
OL	Organic silt and silty clay of low plasticity
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plasticity Limit
USCS	Unified Soil Classification System

USBR	United States Bureau of Reclamation
USSD	United State Society on Dams

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CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

Dams are constructed to impound water for diverse uses and will not be considered adequate in its performance if the structure is susceptible to high degree of seepage. Usually, in evaluating materials to be used in dam construction, extensive study of the engineering properties are expected and these are done in relation to different location of the material sources. Earth dams have been the most usual types of dams constructed from earliest times. Its popularity was due to the availability of earthen materials, high cost of masonry and lack of concrete then. However, earth dams still strive in modern Africa because of the ease of construction and relatively lower cost compared to other types of dams (Ogedengbe & Oke, 2010).

The occurrence and distribution of soils in nature varies from location to location. The type of soil depends on the rock type, its mineral constituents and the climatic regime of the area. Soils are used as construction materials or the civil engineering structures are founded in or on the surface of the earth. Geotechnical properties of soils influence the stability of civil engineering structures. Most of properties of soils influence to each other. Different geotechnical properties of soils such as specific gravity, density index, consistency limits, particle size analysis, compaction, consolidation, permeability and shear strength and their interactions and applications for the purpose of civil structures (Surendra & Sanjeev, 2017)

Soil consistency is the most critical engineering property to be considered when evaluating earth materials for dam construction. Soil consistency is related to the liquid limit (LL), plastic limit (PL) and the plasticity index (PI). Other engineering properties of equal importance particularly for the earth materials include the grain size distribution analysis, compaction characteristics, shear strength, consolidation characteristics and permeability.

It is the usual practice to move earth materials that satisfy the engineering standards from a location to another during construction of dams and many other engineering structures. The properties of soil source from other places are primarily soil specific. The compromises of guidelines and standards for materials have contributed to the collapse of dams and such hydraulic structures in Nigeria in the past years (Ogedengbe & Oke, 2006).

As water from the reservoir seeps through the pores of an earth dam, seepage forces are exerted on the soil particles in the direction of the flow. Within a single embankment zone, the individual particles acted on by the seepage forces cannot move because they are held in place by neighboring particles. At a point inside the zoned dam, where the water discharges from fine materials into coarse materials. However, it is possible for the finer soil particles to be washed into the void spaces of the coarser materials, and this may initiate complete dislocation in the embankments (James *et al*., 1963) hence the need for thorough compaction.

Depending on the availability of suitable materials to build the embankment, the dam can be earth-fill, it can be made with one single relatively impermeable earth material (homogeneous fill), or this material can be used only for the core of the embankment and on either side are zones of permeable granular or rock-fill material also termed zoned filled (FAO, 2001).

According to United States Society on Dam (2011), embankment dams are constructed of all types of geological materials, with the exception of organic soils and peats. Most embankments are designed to utilize the economically available on-site materials for the bulk of construction. Special zones such as filters, drains and riprap, may come from off-site sources. Soil materials used in embankment dams are commonly obtained by mass production from local borrow pits, and from required excavations where suitable.

Two basic soil features are considered when constructing an earth dam; soil permeability and soil strength. Soil permeability predicts the seepage rate of water stored in a dam reservoir, and soil strength indicates stability and strength of the dam embankment. The properties that affect both permeability and strength are numerous and complex. Therefore, fill materials must be studied properly.

1.2 Statement of the Research Problem

In earth dam construction, the selection of the appropriate soil for use in the central core and the embankment is very crucial and key to the sustainability of the dam. Also during construction of an earth dam, highly impermeable soil is not required in the upstream shoulder of the dam because it can lead to undesirable uplift pressures developing beneath the embankment.

Leakage due to improper fill materials has been linked to 40% failure in dam (FAO, 2001). Fill materials must be properly selected and studied to be free from soluble element such as sodium which can easily be mobilize when wet and under load thereby making the dam unstable.

Food and Agricultural Organization (FAO, 2001). Inadequate control of the compaction characteristics of fill materials causes the occurrence of internal erosion and piping, it is in these areas that erosion and piping processes begin in an embankment of the dam, therefore causing failure through seepage (Flores-Berrones & López-Acosta, 2019).

Hydraulic fracture due to concentration of flow lines at downstream toe leads to the increase in hydraulic gradient. Upward seepage force causes reduction in effective stresses in foundation.

1.3 Aim and Objectives of the Study

1.3.1 Aim of the Study

The aim of this study is to assess the geotechnical properties of selected lateritic soil as fill materials for earth dam construction.

1.3.2 Objectives of the Study

To achieve the above aim, the objectives are as follows:

1. Determination of the physical properties of sampled lateritic soil obtained from the selected borrow pits.
2. Determination of the compaction characteristic of selected lateritic soil using British Heavy Compaction effort.
3. Determination of the permeability (hydraulic conductivity) of sampled lateritic soil.
4. Determination of the shear strength the sampled lateritic soil.

1.4 Scope of the Study

This work focused on geotechnical properties of selected lateritic soil obtained from borrow pits in selected locations in the suburb of Minna. The materials were collected from different borrow pits and the following tests were performed on the sample to evaluate the physical properties of the soils. The following tests will be conducted; compaction, compressibility, permeability and shear strength. Grading (both mechanical sieving and hydrometer test) were carried out to determine the particle size distribution of the soil. Also Atterberg limit tests to measure the plastic and liquid limits of soils were carried out to enable the classification and its suitability as fill materials. British standard heavy test were carried out to determine the maximum dry density and optimum content for maximum strength.

These tests were carried out in the Civil Engineering Laboratory, Federal University of Technology Minna. Equipment used include triaxial machine for shear strength test, mechanical sieve shaker, hydrometer and standard and modified Proctor compactor.

1.5 Justification of the Study

This work revealed the geotechnical properties of sampled lateritic soil in Minna. This information is useful in the selection, sorting and grading of materials to be used as fill materials for construction of earth dam. It also furnished geotechnical engineers, designers and other researchers with useful parameters, which are required for geotechnical design of fill material especially for earth dam construction. In addition, the work will not only add to knowledge, but will also lead to materials optimization, efficiency and improved design and construction of earth dam, with the sorted lateritic soil as the fill materials.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Dam

Elshemy *et al.* (2002) described a dam as a built barrier across a river for specific functions such as water supply, irrigation, flood control, livestock farming and hydroelectric power generation. Most of the large dams in the world were built during the middle decades of the twentieth century. There are two types of modern dams, namely: embankment dam and concrete dam. Embankment dams can be classified into two main categories earth-fill dams and rock-fill dams. Embankment dams represent about 85% of dams all over the world. There are several factors to be considered in selecting an earth dam type. These includes; topography, foundation conditions, environmental impacts, construction facilities and socio-economic consideration. A feasible dam should be built from locally available materials; stable under all operating and loading conditions; watertight enough to control seepage; have appropriate outlet works to arrest dam overtopping (Greager & Hinds, 2002).

2.2 Earth Dams

Earth fill dams are simple structures which stand on their self-weight to prevent sliding and overturning (Arora, 2001). Earth-fill dams are the most common type of dams known in the world. At the earlier time the earth-fill dams are constructed to divert massive water body and protect the community. Later it was structurally improved and used to construct reservoirs. Earth-fill dams have some variety of advantages both technically and economically such as;

(i) Construction materials are readily available (ii) Simple design criteria (iii) Less foundation preparation required when compared with other dams. (iv) Quite flexible than other rigid dam structures and suitable for seismic sensitive regions. On the other hand, some of it disadvantages when compared with other dam types includes; (i) Higher possibility to slide (ii) Lack of compaction of material leads to increased seepage (iii) Continuous monitoring and assessment are required to prevent slope erosion, abnormal seepage and growing plants (Omofunmi *et al.,* 2017).

Omofunmi *et al.* (2017) considered an earth-fill dam to be made of earth (or soil) built up by compacting successive layers of earth, using the most impervious materials to form a core and placing more permeable substances on the upstream and downstream sides. Many earth-fill dams are vulnerable to failures due to seepage problems that take place in the core since all soils are pervious to a smaller or larger extent (Kanchana *et al.,* 2015). The earth-fill dam can be of the following three types: Homogeneous Embankment, Zoned Embankment and Diaphragm Embankment (Craig, 2004).

The basis of design of earth dams is focused on ensuring the stability of the structure under a set of conditions expected to occur during its life. The stability of the upstream and downstream slopes must be guaranteed at the end of the construction but also during reservoir impoundment and the operational phase, including drawdown and long-term steady state conditions as a limiting case. A fundamental aspect of the analysis is the generation of pore pressures during the construction and during the first filling, reservoir impounding and cases of rapid drawdown. Other aspects are also of concern, such as the deformation of the structure during the construction and operational stages, and also incidents caused by hydraulic fracture, internal erosion, long term effects and other combined cases (Alonso *et al.,* 2014)

The hydro-mechanical behaviour of the materials used in the construction of the earth structures are used to explain their failure. An additional source of complexity is the fact that different types of materials are used, for earth dams, impervious clayey materials are used for the core, rock-fill materials are used for shells and granular materials are used for filters (USSD, 2011).

2.2.1 Homogeneous embankment

The homogeneous dam is a simple embankment, which is essentially homogeneous throughout, that, is the use of single type of material to fill. Although a blanket of relatively impervious material may be placed on the upstream face. Homogeneous dams also have seepage control features such as chimney drains, blanket drains, other materials including a filter zone between the main embankment material and the drain. The homogeneous structure is generally more massive and usually has flatter slopes than a zoned embankment of the same height (Craig, R. F., 2004). These characteristics compensate for a tendency toward a higher phreatic line in the homogeneous embankment. They also tend to provide better slope stability during rapid drawdown (USBR, 2006).

2.2.2 Zoned embankment

This is a more common type of embankment dam constructed using, basically, pervious and impervious materials. The impervious materials, called the core, is placed at the center and is bordered by zones of pervious materials called shells. The central core is supported and protected by the shells. The upstream shell provide stability against sudden draw-down, and the downstream one acts as a drain to control the line of seepage. The materials for the pervious zones may be sands, gravel, cobbles, or rocks or mixtures of these materials and width of the core is controlled by the availability of material and design requirements, such as stability and seepage. If the major part of the dam is composed of rock, it is called a rock-fill dam (USBR, 2006)

This type of dam reduce seepage to minimum compared to homogeneous embankments, its costs are likely to be higher, mainly because the earthworks material is divided into three categories: pervious for the downstream section, impervious for the core (or hearting) and semi-impervious for the upstream section, all of which has to be excavated from separate borrow areas.

2.2.3 Diaphragm embankment

According to United State Bureau of Reclamation (USBR, 2006) diaphragm type of dam is an earth-filled dam where the major component of construction materials is pervious and have a thin central section of concrete, steel, bitumen or timber which serves as a water barrier, while the surrounding earth or rock fill provides stability. Differential earth loads easily crack thin concrete sections, and it is difficult to form a perfect watertight barrier of steel or timber. The diaphragm may be located in the center as a vertical core or placed as a blanket on the upstream face. In addition, the diaphragm must be anchored into bedrock or an impermeable material if excessive under seepage is to be avoided.

2.2.4 Components of a typical earth-filled dam

Embankment dam consists of three basic components; these are foundation, shell, and core. These are shown on Figure 2.1.

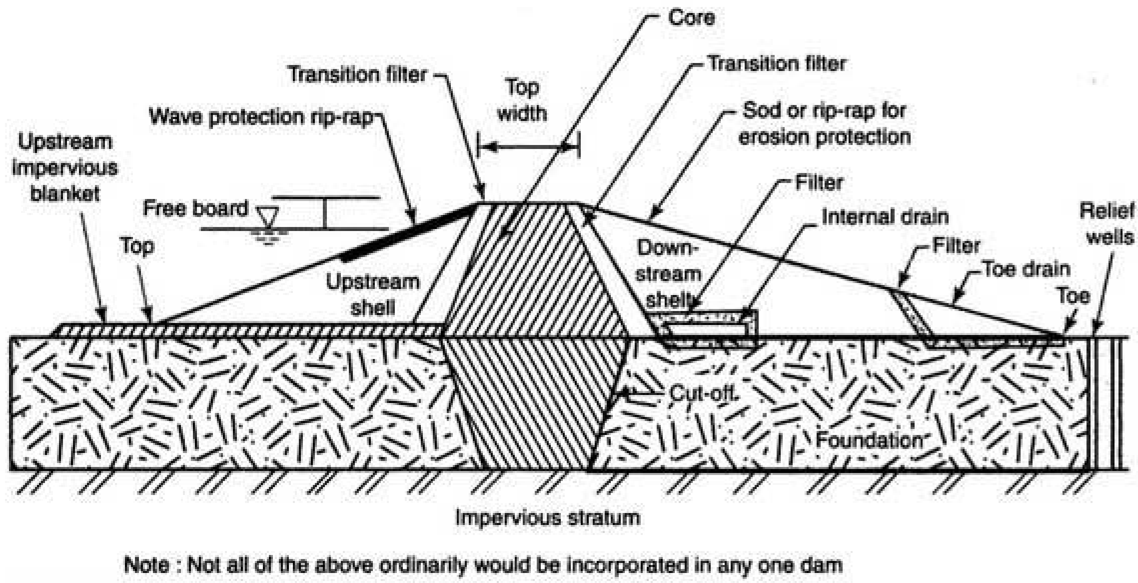


Figure 2.1: Typical section through an earth dam (FAO, 2010)

2.3 Fill Materials

The materials used to construct earth dams, road embankment, rail embankment and foundation filling of a building are referred to as fill materials which includes soil and rock. Sometimes geotextile and geo-membrane can also be used. Soil is classified by particle size from the smallest, submicroscopic particle classified as clay or silt, which are very fine ranging from fine to coarse. The fine grains are the smallest soil particle eyes can see. Coarser fragments called cobbles and boulders are also used in the dam construction but as protective outer layer.

Specific soil types and size ranges are needed to construct the zones within the dam and exploration of the dam foundation, reservoir and surrounding area to locate construction materials. Samples for construction materials are tested in a soil laboratory for grain size, moisture content, dry density (weight), plasticity and permeability. The combination of fine size and plastic behaviour also causes the clay to be less permeable to water. If clay is available near the site, the dam can be built with an impermeable core or central zone that prevent water from passing through the dam. Various soils have different characteristics that can be determined in the laboratory and used during the construction.

Fill materials also includes geotextiles and geo-membranes. Geotextiles are nonwoven fabrics that are strong and puncture resistance. They can be used as filter to wrap coarser drain material to limit migration of fine grain soil into drainage material. Geo-membranes are made of high density polyethylene (HDPE) plastics and are impermeable. They can be used to line the upstream face of a fill dam or line the reservoir. For the purpose of this research, the emphasis is placed mainly on earth dam which is based purely on soil and rock.

2.3.1 Soil as fill material for earth dam

Embankment dams are constructed with all types of geologic materials, with the exception of organic soils and peats (USSD, 2011). Most embankments are designed to utilize the economically available materials on-site for the bulk of construction. Special zones such as filters, drains and riprap, may come from off-site sources. Soil materials used in embankment dams commonly are obtained by mass production from local borrow pits

2.4 Geotechnical Properties of Soil as Fill Material

Civil engineering structures like building, bridge, highway, tunnel, dam and tower are founded below or on the surface of the earth. For their stability, suitable foundation soil is required. To check the suitability of soil to be used as foundation or as construction materials, its properties are required to be assessed (Laskar & Pal, 2012). According to Oke *et al.* (2008), assessing geotechnical properties of subsoil at project site is necessary for generating germane input data for design and construction of foundations for the proposed structures. Oghenero *et al.* (2014) affirm that proper design and construction of civil engineering structures prevent an adverse environmental impact or structural failure or post construction problems. For complex projects involving heavy structures, such as bridges, dams, multi-storey buildings, it is essential to have detail exploration. The purpose of detailed explorations is to determine the engineering properties of the soils for different strata (Arora, 2018).

When the foundations of any structure are constructed on compressible soil, it leads to settlement. Knowledge of the rate at which the compression of the soil takes place is essential for design consideration. The properties of the soil such as plasticity, compressibility or strength of the soil always affect the design in the construction. Lack of understanding of the properties of the soil can lead to the construction errors. The suitability of soil for a particular use should be determined based on its engineering characteristics and not on visual inspection or apparent similarity to other soils. The loading capability of soil depends upon the type of soil.

Generally, fine grained soils have a relative smaller capacity in bearing of load than the coarser grained soils (Jain *et al.,* 2015). Plasticity index and liquid limit are the important factors that help an engineer to understand the consistency or plasticity of clay. Though shearing strength constants at liquid limits but varies for plastic limits for all clays (Murthy, 2002). Permeability influences the civil engineering structures. According to Karsten *et al.,* 2006, the shear strength of soils is of special relevance among geotechnical soil properties because it is one of the fundamental parameters for analyzing and solving stability problems of calculating earth pressure, bearing capacity of footings and foundations, slope stability or stability of embankments and earth dam.

2.4.1 Fine-grained soils (silts and clays)

Fine grained soils are often used in homogeneous dams, and in impervious core sections of zoned embankment dams. The general characteristics of fine grained soils in embankment dams are described.

2.4.2 Shear strength of silt and clay soil

There is a strong correlation between incidence of embankment slope failures and the use of fine grained/highly plastic soil in embankments. Excess pore pressures often develop during rapid construction of fine-grained fill zones, resulting in reduced shear strength and potentially unstable conditions during or shortly after construction. The report of Sherard (1953) indicated that the correlation between fineness of soil and the susceptibility to sliding was strong enough to outweigh the influence of all other factors, including steepness of slopes, construction methods, and reservoir activity. The USBR recommends shallow slopes (3H: 1V to 4H: 1V for upstream slopes, and 2.5H: 1V for downstream slopes) for homogeneous or modified-homogeneous small dams constructed of fine grained soils.

2.4.3 Compressibility of fill materials

Compressibility of embankments depends on the soil properties and the placement conditions. Under lower consolidation stresses, such as in the upper sections of an embankment, compressibility appears to correspond with placement moisture. Fill placed at relatively low average water contents shows low initial strain and moderately increasing or constant compressibility under higher pressures. Some materials may exhibit collapse settlement on wetting if placed at low initial moisture. Embankments may exhibit high initial strains when constructed at water contents near standard Proctor optimum, as determined by ASTM D698

(ASTM, 1998). The potential for collapse settlement, or high initial settlement can be predicted on the basis of laboratory testing prior to construction. Gradation and plasticity of embankment soils are considered to be more important than placement moisture conditions under high consolidation pressures, such as in the lower portions of a high dam (Sherard *et al.,* 1963).

2.4.4 Hydraulic conductivity of silt and clay soil

Clean gravels have high hydraulic conductivities, ranging on the order of 1 to 100 cm/s. The hydraulic conductivity, k, of clean sands or clean gravel-sand mixtures can be reasonably estimated based on semi-empirical correlations with effective grain size, D_{10} . The Hazen equation for example, can be used to estimate k in cm/s using the relationship

$$k = C(D_{10})^2 \tag{2.1}$$

Where, D_{10} = grain size in mm corresponding to 10 percent passing on the gradation curve for the soil. This equation is considered valid for materials having D_{10} sizes between 0.1 and 3.0 mm. The constant C varies between 0.4 and 1.2, with an average value of 1 (Holtz & Kovacs, 1981).

The USBR measured hydraulic conductivities of well-graded sand and gravel mixtures in the range of about 1 to 5 X 10^{-2} cm/s, for mixtures containing 20 to 65 percent gravel sized particles at relative densities ranging from 50 to 70 percent. At higher or lower gravel percentages, hydraulic conductivities were found to be substantially higher (USBR, 1990).

Shear strength is directly proportional and hydraulic conductivity is inversely proportional to relative density of granular materials. Relative density is defined as

$$D_r = \frac{e_{\max e}}{e_{max}\ e_{min}} \tag{2.3}$$

where,

e_{max} = maximum void ratio

e_{min} = minimum void ratio

e = void ratios.

The maximum and minimum void ratio are defined and evaluated by standardized laboratory testing procedures (ASTM Methods; ASTM D 4253 and ASTM D 4254, respectively). Typically, relative densities are specified to be on the order of 75 percent or greater in structural fill. High relative density correlates with high strength.

2.4.5 Coarse-grained soils (sand and gravel)

Coarse grained soils are used in structural fill zones, or shells, and in special filter and drain zones within embankment dams. Coarse grained soils are also used in core zones, especially when the fines content is greater than 20%. The general physical characteristics and properties of coarse grained soils are discussed in the next section as special concerns regarding the permeability, erosion potential, and liquefaction potential of coarse grained soils.

2.4.6 General characteristics of coarse-grained soil and properties

Coarse grained soils are defined in the Unified Soil Classification System as those materials having more than 50 percent by dry weight of particles retained on the U.S. Standard No. 200 sieve, or 0.075 mm. Coarse-grained soils include gravels and sands, which are differentiated by size. Sands are defined as soils finer than the No. 4 sieve (4.76 mm) and coarser than the No. 200 sieve. Gravels are coarser than the No. 4 sieve and finer than 3 inches (76.2 mm). This size division does not correspond with a distinct change in engineering behaviour, although in general, gravels are more pervious and exhibit greater shear strengths than sands.

The important properties of interest in embankment dam engineering, namely shear strength, compressibility, and permeability are determined by the gradation, grain size and shape, relative density, and durability of the coarse grained soil. Compressibility is generally of less concern, as these soils are essentially incompressible when compacted to a dense state.

2.5 Rock as Fill Material for Earth Dam

Rock-fill in current practice includes angular rock fragments as produced by quarry or occurring as deposits, rounded or sub angular fragments such as coarse gravel, cobbles, and boulders occurring in alluvial deposits. It is generally considered that the difference between clean rock-fill and dirty rock-fill is that in clean rock-fill the rock content is sufficient to have rock-to-rock contact with the strength of the rock controlling the shear strength rather than the soils or fines. For many rock materials, which occurs at rock content of 60 to 70 percent. Dirty rock-fill with a hydraulic conductivity less than 1×10^{-3} cm/sec may be considered as earth-fill because the possibility of developing construction pore pressures.

The physical engineering properties of rock-fill are difficult to evaluate and requires special testing procedures, particularly to determine the strength and permeability because of the large particle size

2.5.1 Specific gravity of fill materials

Specific gravity is the ratio of the mass of soil solids to the mass of an equal volume of water. It is an important index property of soils that is closely linked with mineralogy or chemical composition (Oyediran & Durojaoye, 2011). It is very important as far as the qualitative behaviour of the soil is concerned (Raj 2012), to examine the mineral classification of soil as it gives an idea about suitability or otherwise. It is also used in the calculation of void ratio, porosity, degree of saturation and other soil parameters (Prakash & Jain, 2002). Typical values of specific gravity are given in Table 2.1

Table 2.1: Typical values of specific gravity (Bowles, 2012)

Types of soil	Specific gravity
Sand	2.65-2.67
Silt sand	2.67-2.70
Inorganic clay	2.70-2.80
Soil with mica or iron	2.75-3.0
Organic soil	1.00-2.60

Roy and Dass (2014) reported that increase in specific gravity can increase the shear strength parameters (cohesion and angle of shearing resistance). Roy (2016) observed that increase in specific gravity also increases the California bearing ratio.

2.5.2 Density index of fill materials

The degree of compaction of fine-grained soils is measured in relation to maximum dry density for a certain compactive effort. But in the case of coarse grained soils, a different sort of index is used for compaction. Depending upon the shape, size, and gradation of soil grains, coarse-grained soils can remain in two extreme states of compaction, namely in the loosest and densest states. Any intermediate state of compaction can be compared to these two extreme states using an index called relative density or density index as expressed in equation (2.4). The soil characteristics based on relative density are shown in Table 2 .2.

$$D_r = \frac{e_{\max e}}{e_{max}\ e_{min}} \tag{2.4}$$

where,

e_{max} = maximum void ratio

e_{min} = minimum void ratio

e = void ratios.

Table 2.2: Characteristics of soils based on relative density

Relative density (%)	Soil compactness	Angle of shearing resistance(º)
0-5	Very loose	<28
15-35	Loose	28-30
35-65	Medium	30-36

65-85	Dense	36-41
85-100	Very dense	>41

Density index is expressed in percent and is defined as the ratio of the difference between the void ratio of a cohesion less soil in the loosest state and any given void ratio to the difference between its void ratios in the loosest and the densest states (BS 1377, 1990). It is a measure of the degree of compactness and the stability of soil stratum (Raj, 2012). In reality, it expresses the ratio of actual decrease in volume of voids in a sandy soil to the maximum possible decrease in volume of voids that is how far the sand under investigation can capable to the further densification beyond its natural state.

2.5.3 Particle size analysis of fill materials

The percentage of different sizes of soil particles coarser than 75µ is determined by sieve analysis whereas less than 75µ are determined by hydrometer analysis. Based on the particle size analysis, particle size distribution curves are plotted. The particle size distribution curve represents the distribution of particles of different sizes in the soil mass (Mallo & Umbugadu, 2012). The main principle is to mix few selected soils in proportion that a desired grain size distribution is obtained for the design mix. Hence for proportioning the selected soils, the grain size distribution of each soil is required to be known (Prakash & Jain, 2002).

The data obtained from grain size distribution curves are used in the design of filters for earth dams and to determine suitability of soil for road construction and airfields. Raj (2012) reported that the particle size of sands and silts has some practical value in design of filters and in the assessment of permeability, capillarity and frost susceptibility. Very important and useful information may be obtained from grain size curve such as (i) the total percentage of larger or finer particles than a given size and (ii) the uniformity or the range in grain-size distribution.

Bowles (2012) reported that particle-size is one of the suitability criteria of soils for roads, airfield, levee, dam, and other embankment construction. Information obtained from particle-size analysis can be used to predict soil-water movement, although permeability tests are more generally used.

The susceptibility to frost action in soil, an extremely important consideration in colder climates, can be predicted from the particle-size analysis. Very fine soil particles are easily carried in suspension by percolating soil water, and under drainage, systems are rapidly filled with sediments unless a filter made of appropriately graded granular materials properly surrounds them. The proper gradation of this filter material can be predicted from the particle-size analysis. Particle-size of the filter materials must be larger than the soil being protected so that the filter pores could permit passage of water but collect the smaller soil particles from suspension.

Dafalla (2013) reported that the sand shape whether rounded, sub rounded, or angular will affect the shearing strength of soil. Angular grains provide more interlock and increased shear resistance. The gradation and size of the sand affect the shear resistance. Well-graded materials provide more grain to grain area contact than poorly graded materials. Porosity and spaces available for clay within the sand is an important while considering the mixtures of clays and sands.

2.5.4 Compaction of fill materials

Soil compaction is one of the ground improvement techniques. It is a process in which by expending compactive energy on soil, the soil grains are more closely rearranged. Compaction increases the shear strength of soil and reduces its compressibility and permeability.
Murthy (2002) explained that when an earth dam is properly compacted, the shear strength of the material is increased and dam becomes more stable. Since the soil becomes dense, its permeability gets decreased. The decrease in the permeability of the dam decreases the seepage loss of the water stored. The settlement of the dam also decreases due to the increase in the density of the materials.

According to Prakash and Jain (2002), compaction of soils increases the density, shear strength, bearing capacity but reduces their void ratio, porosity, permeability and settlements. The results are useful in the stability of field problems like earthen dams, embankments, roads and airfields. The moisture content at which the soils are compacted in the field is controlled by the value of optimum moisture content determined by the laboratory proctor compaction test. The compaction energy applied in the field is also controlled by the maximum dry density determined in the laboratory.

Cylindrical metal mould, having internal diameter 4” (10.16 cm) or 6” (15.24 cm), the internal effective height of 4.6” (11.7 cm) and the mould should have detachable base plate and collar of 2 inches (5.08 cm), rammer, weighing 4.5 kg having fall of 12 inches (30.5 cm), with a flat circular face of 2” diameter, sensitive balance, sensitivity ranging from 0.1 gram to 1 gram. Thermostatically controlled oven (105°C +/- 110°C)**,** steel straight edge, moisture containers, Sieve No.4, tray and scoop, graduated cylinder, mixing tools (spoon, trowel, spatula).

2.5.5 Permeability of fill materials

The amount, distribution and movement of water in soil have an important role on the properties and behavior of soil. The principles of fluid flow as groundwater conditions are frequently encountered on construction projects site. Water pressure is always measured relative to atmospheric pressure and water table is the level at which the pressure is atmospheric. Soil mass is divided into two zones with respect to the water table: (i) below the water table (a saturated zone with 100% degree of saturation) and (ii) just above the water table called the capillary zone with degree of saturation ≤ 100% (Raj, 2012) .

According to Raj (2012), data from field permeability tests are needed in the design of various civil engineering works such as cut-off trench design of earth dams to ascertain the pumping capacity for dewatering excavations and to obtain aquifer constants. The permeability of soils has a decisive effect on the stability of foundations, seepage loss through embankments of reservoirs, drainage of subgrades, excavation of open cuts in water bearing sand, and rate of flow of water into wells (Murthy, 2002).

Prakash and Jain (2002) explained that water flowing through soil exerts considerable seepage forces, which have direct effect on the safety of hydraulic structures. The rate of settlement of compressible clay layer under load depends on its permeability. The quantity of stored water escaping through and beneath an earthen dam depends on the permeability of the embankment and the foundation respectively. The rate of drainage of water through wells and excavated foundation pits depends on the coefficient of permeability of the soils. Shear strength of soils also depends indirectly on its permeability, because dissipation of pore pressure is controlled by its permeability. According to United State Bureau of Reclamation, soils are classified as (i) Impervious: k (coefficient of permeability) less than 10^{-6} cm/sec (ii) Semi pervious: k between 10^{-6} to 10^{-4} cm/sec (iii) Pervious: k greater than 10^{-4} cm/sec.

2.5.6 Shear strength of fill materials

The shear resistance of soil is the result of friction and the interlocking of particles and possibly bonding at the particle contacts. The shear strength parameters of soils are defined as cohesion and the friction angle. The shear strength of soil depends on the effective stress, drainage conditions, density of the particles, rate of strain, and direction of the strain. Thus, the shearing strength is affected by the consistency of the materials, mineralogy, grain size distribution, shape of the particles, initial void ratio and features such as layers, joints, fissures and cementation (Poulos, 1989).

The shear strength parameters of a granular soil are directly correlated to the maximum particle size, coefficient of uniformity, density, applied normal stress, gravel and fines content of the sample. It can be said that the shear strength parameters are a result of the frictional forces of the particles as they slide and interlock during shearing (Yagiz, 2001). Soil containing particles with high angularity tend to resist displacement and hence possess higher shearing strength compared to those with less angular particles.

Prakash (2002) and Raj (2012) explained that the capability of a soil to support loading from a structure, or overburden, or to sustain a slope in equilibrium is governed by its shear strength. The shear strength of soil is of paramount importance for foundation design, earth and rock fill dam design, highway and airfield design, stability of slopes and cuts, and lateral earth pressure problems. It is highly complex because of various factors involved in it such as the heterogeneity nature of the soil, the water table location, the drainage facility, the type and nature of construction, the stress history, time, chemical action, or environmental conditions.

Prakash and Jain (2002), confining pressures play the significant role in changing the behavior of soils in deep foundations. Similarly, in high-rise earth dams, the confining pressures are of very high magnitude. Triaxial test is the only test to simulate these confining pressures. For short-term stability of foundations, dams and slopes, shear strength parameters for unconsolidated undrained or consolidated undrained conditions are used, while for long term stability shear parameters corresponding to consolidated drained conditions give more reliable results.

Akayuli *et al*. (2013) found that the friction angle is high for a sandy soil than its cohesion and vice versa for clayey soil. Shanyoug *et al.* (2009), in their study concluded that there is a general increase in cohesion with clay content. As more clay is introduced into the sandy materials, the clay particles fill the void spaces in between the sand particles and begin to induce the sand with interlocking behavior. Hence, clayey sand soils are expected to exhibit low cohesion whereas the cohesion increases with high clay content.

Dafalla (2013) observed that the mineralogy could have a major role in the shearing strength capacity of clays. The cementation between particles can either be due to a chemical bond or physicochemical bond. Swelling and shrinkage in expansive soils are of two extreme opposite effects on the shearing strength. The shear strength is generally low for fully expanded clay while dry shrinking clay is capable of developing higher cohesion and angle of internal friction. The study indicated that choosing the appropriate mix or using appropriate quantity of clay, can help to achieve required shear strength. Very moist clay-sand mixture showed steep drop in both cohesion and angle of internal friction when the clay content is high.

According to Murthy (2002) and El-Maksoud (2006), cohesion is mainly due to the intermolecular bond between the adsorbed water surrounding each grain, especially in fine-grained soils. According to Mollahasani *et al.* (2011), the soils with high plasticity like clayey soils have higher cohesion and lower angle of shearing resistance. Conversely, as the soil grain size increases like sands, the soil cohesion decreases.

Duncan *et al.* (2014) reported that the stability of soil strength and slope stability depend on the shear strength parameter of clayey soil due to the high compactive effort when applied to such soil. Also Fell *et al.* (2015) indicate that for a stable embankment slope, the soil most exhibit low angle of friction and relative high cohesion.

2.5.7 Consolidation of fill materials

When a soil layer is subjected to compressive stress due to construction activities, it undergoes compression. The compression is caused by rearrangement of particles, seepage of water, crushing of particles and elastic distortions. Settlement of a structure is analyzed for three reasons: appearance of structure, utility of the structure, and damage to the structure. The aesthetic view of a structure will diminished due to the presence of cracks or tilt of the structure caused by settlement. Further settlement can cause a structure to fail structurally and collapse.

Settlement is the combination of time-independent (immediate compression) and time-dependent compression (Raj, 2012).

According to Prakash and Jain (2002), the main aim of a consolidation test is to obtain soil data which are used in predicting the rate and amount of settlement of structure founded on clay primarily due to volume change of the clay. The information obtained for foundations resting on clay are: (i) total settlement of foundation under any given load, (ii) time required for total settlement due to primary consolidation, (iii) settlement for any given time and load, (iv) time required for any percentage of total settlement or consolidation, and (v) pressure due to which soil already has been consolidated/compressed.

Abeele (1985) explained that lowering of water table or dewatering is probably the best known cause of massive settlement. When submerged, soil particles are subjected to buoyancy. Upon dewatering, the buoyancy is removed and the apparent increase in pressure results in consolidation, even though there is no increase in external load. Vibrations can also have a densification effect on soils and lead to subsequent settlement. The effects can be severe when the vibration frequency matches the soil's natural frequency that is at resonance. Soils often fail and settle disastrously as a result of earthquakes. Devastating landslides are often one of the results of such occurrences. Of the three phases of soil, only the solid phase controls the resistance to compression and shear. Water, present in a moist soil is highly incompressible but as a liquid, is not capable of resisting shear loads. Air, present in unsaturated soils, will not support compression or shear loads.

Head (2010) reported that in a saturated soil, compression would be primarily caused by expulsion of water out of the soil voids. Under the influence of an externally applied load, the expulsion of water from the voids is highly dependent on the permeability of the medium. The extremely low permeability in the case of clay leads to a slow void contraction. The compression of saturated, low permeability layers under a static pressure is known as *consolidation*. The consolidation rate depends on the compressibility of the soil (rate of decrease in volume with stress) and soil permeability, which in turn, is dependent on the viscosity of the liquid. An increase in temperature increases the consolidation rate but does not affect total amount of consolidation.

2.5.8 Consistency limits of fill materials

The consistency of a fine-grained soil is largely influenced by the water content of the soil. A gradual decrease in water content of a fine-grained soil slurry causes the soil to pass from the liquid state to a plastic state, from the plastic state to a semi-solid state, and finally to the solid state. The water contents at these changes of state are different for different soils. The water contents that correspond to these changes of state are called the Atterberg limits. The water contents corresponding to transition from one state to the next are known as the liquid limit, the plastic limit and the shrinkage limit (Kaniraj, 1988).

The liquid limit of a soil is the water content, expressed as percentage of the weight of the oven-dried soil, at the boundary between the liquid and plastic states of consistency of the soil (IS-2720, 1970). The soil has negligibly small shear strength, the plastic limit of a soil is the water content, expressed as a percentage of the weight of oven dried soil, at the boundary between the plastic and semi-solid states of consistency of the soil.

The plastic limit for different soils has a narrow range of numerical values. Sand has no plastic stage, but very fine sand exhibits slight plasticity. The plastic limit is an important soil property. Earth roads are easily usable at this water content. Excavation work and agricultural cultivation can be carried out with the least effort with soils at the plastic limit. Soil is said to be in the plastic range when it possesses water content in between liquid limit and plastic limit. The range of the plastic state is given by the difference between liquid limit and plastic limit and is defined as the plasticity index. The plasticity index is used in soil classification and in various correlations with other soil properties as a basic soil characteristic (Raj, 2012).

Based on the plasticity index, the soils were classified by Atterberg, shows the correlations between the plasticity index, soil type, degree of plasticity and degree of cohesiveness Table 2.3

Table 2.3: Types of soils based on plasticity index (Skemton, 1953)

Plasticity index (%)	Soil type	Degree of plasticity	Degree of cohesiveness
0	Sand	Non-plastic	Non-cohesive
<7	Silt	Low plastic	Partly cohesive
7-17	Silt clay	Medium plastic	Cohesive
>17	Clay	High plastic	Cohesive

Skempton (1953) reported that the plasticity index of a soil increases linearly with the percentage of the clay-sized fraction. Laskar and Pal (2012) reporteds that plasticity depends on grain size of soil. With the increase of sand content plasticity index of soil decreases, which might be due to decrease of inter molecular attraction force. Due to decrease of attraction force, liquid limit of the soil decreases and accordingly plasticity index decreases. But as the clay content increases inter molecular attraction force increases and liquid limit increases.

The shrinkage limit is the maximum water content expressed as a percentage of oven-dried weight at which any further reduction in water content will not cause a decrease in volume of the soil mass, the soil mass being prepared initially from remoulded soil. The finer the particles of the soil, the greater are the amount of shrinkage. Soils that contain montmorillonite clay mineral shrink more. Such soils shrink heterogeneously during summer, as a result of which cracks develop on the surface. Further, these soils imbibe more and more water during the monsoon and swell. Soils that shrink and swell are categorized as expansive soils (Raj, 2012).

According to Prakash and Jain (2002), the value of shrinkage limit is used for understanding the swelling and shrinkage properties of cohesive soils. It is used for calculating the shrinkage factors which help in the design problems of the structures made of the soils or resting on soil. It gives an idea about the suitability of the soil as a construction material in foundations, roads, embankments and dams. It helps in knowing the state of given soil.

Ersoy *et al*. (2013), consistency is an important property and is a useful measure for the processing of very fine clayey soils. Plasticity and cohesion reflect the soil consistency and workability of the soils. However, these properties of the soils play an essential role in many engineering projects, such as the construction of the clay core in an earth fill dam, the construction of a layer of low permeability covering a deposit of polluted material, the design of foundations, retaining walls and slab bridges, and determining the stability of the soil on a slope.

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Lateritic Soil Samples

The materials for the study were collected from selected borrow pits in the suburb of Minna. Niger State. The following tests were carried out to assess the engineering properties of the samples to ascertain its behaviour when use as a fill material for dam embankment construction. Laboratory test carried out on the soil samples are listed in section 3.2.

3.2 Method of Sample Collection

Disturbed sample collection method was used. It involves digging and collecting samples from 0.5m, 1.0m and 1.5m. A total of twenty-seven (27) samples were collected. Geologically, the study area lies within the Northern central basement complex of Nigeria. The area is characterized by migmatite gneiss complex, older granite and schist by Mustapha and Alhassan (2012). The location and coordinates of samples collected are shown in Table 3.1

Table 3.1: Location and coordinates of samples collected

S/N	Location	Coordinates	
		Latitude (n˚) (e˚)	**longitude**
1	Maik A	9˚ 65' 74.19"	6 ˚ 42' 89.3"
2	Maik B	9 ˚ 65' 80"	6 ˚ 42' 00"
3	Maik C	9 ˚ 65' 81.12"	6 ˚ 42' 10.41"
4	Pyata A	9 ˚ 68' 57.44"	6 ˚ 52' 74.76"
5	Pyata B	9 ˚ 68' 53.91"	6 ˚ 53' 08"
6	Pyata C	9 ˚ 72' 02"	6 ˚ 54' 28"
7	L/Gwari A	9 ˚ 53' 77.04"	6 ˚ 51' 58.59"
8	L/Gwari B	9 ˚ 53' 64.59"	6 ˚ 51' 55.29"
9	L/Gwari C	9 ˚ 37' 34.09"	6 ˚ 30' 59.00"

3.2.1 Standard test method and sieve analysis apparatus

Standard sieve set, washing sieve, No. 200 (75-µm), mechanical sieve shaker, balances, drying oven, sieving containers, specimen containers and sieve brushes.

3.2.2 Standard test method for sieve analysis procedure

In accordance to BS, 1377 (2016), the sets of sieves were arranged according to the sieve openings that is from the sieve size of maximum opening to one with least, sieve No.4 (4.75mm), No.8 (2.36mm), No.16 (1.18mm), No.30 (600µm), No. 50 (300µm), No.100 (150µm) and No. 200 (75µm) Insert an additional sieve with opening size intermediate between the sieve that may be overloaded and the sieve immediately above that sieve in the original set of sieves. The percentage passing each sieve size is recorded to the nearest 1 %.

$$\text{Percentage Retained} = \frac{\text{weight of sample on each sieve}}{\text{Total weight of sample}} \times 100 \qquad (3.1)$$

$$\text{Percentage Passing} = 100 - \text{cumulative weight retained} \qquad (3.2)$$

3.3 Compaction Test

3.3.1 Sample preparation

Samples obtained from the borrow pits were transported to Geotechnical Laboratory, Civil Engineering Department, Federal University of Technology, Minna. The processing of the samples began with gradual air-drying of the samples to the desired moisture, usually around 10% or more less than the anticipated moisture. For cohesive soils, was expedited by breaking down clumps and spreading the sample out on open trays. Once the soil is crumble enough, the breakdown can continue more thoroughly. For most standard and modified Proctor variations, this means reducing the finer materials to pass through either a 4.75mm (#4) or 9.5mm (3/8 in) sieve. Coarser mates were set aside for particle size determinations.

Three specimens were prepared for the compaction points with increasing moisture contents and optimum water content. Weight of the specimens to be used will be about 2.3kg each for 4in (102mm) mould and 5.9kg for 6in (152mm) mould to ensure enough compacted volume to properly fill the molds. Water is added incrementally to increase the individual moisture contents by about 2% for each specimen and mixed thoroughly. The prepared specimens were set aside in closed containers for a given time for proper moisture conditioning.

3.3.2 Modified proctor compaction test procedure

Modified proctor compactor, compacts the specimen into the pre-weighed empty Proctor mould in three layers (lifts) according to BS 1377 (1990). Special manual rammers of 4.5 kg rammers with 450 mm drop height was used for compaction as shown in Figure 3.1. The collar is removed and any excess is carefully trimmed with a straightedge tool so the compacted soil is flush with the top of the mold. Small voids can be manually filled with excess sample. The mould with sample is then weighed and recorded and the soil is extruded from the mould. A sample of the specimen is obtained to determine exact moisture content by oven drying, and the process is repeated for subsequent samples as follow;

i. Sufficient quantity of representative soil was air dried and pulverized with a rubber mallet, the soil was then sieved through sieve No: 4 to separate the coarser material.
ii. Water was then added to 300g of soil, to bring its water content to about 5% below the estimated optimum moisture content (for coarse-grained soil 4% initial water content and for fine-grained soil 10% initial water content is preferable). It was then mix it thoroughly.
iii. The mould was then clean, then the diameter, height and weight were measure without the collar.
iv. The collar was then fitted and the soil was compacted in the layers of three evenly distributed blows to each layer.

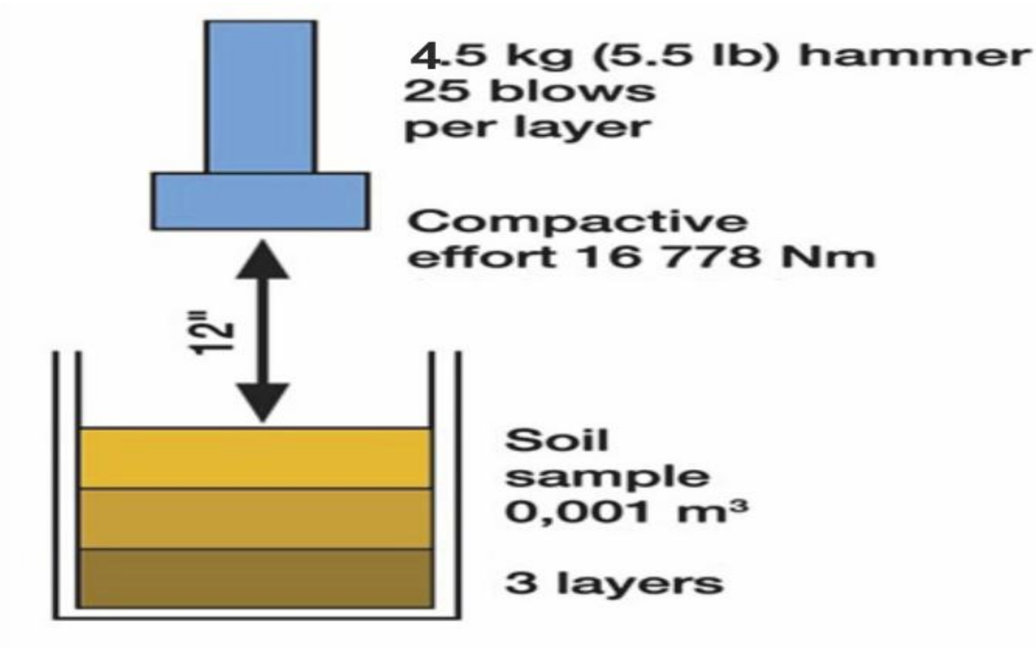


Figure 3.1: Typical mold and hammer (The Constructor, 2017)

v. The collar was removed to trim the compacted soil even with the top of the mold with a straight steel edge. Clean the mould & base plate & weigh again.

vi. The soil from the mold was split and about 100 grams sample was taken for water content determination.

vii. The soil was lamps soil was broken and mix it with remaining soil in the tray. Water will be added to increase the water content by 3% and the compaction procedure for each increment of water was repeated until the mass of the compacted soil decreases.

viii. Water content was calculated for each trail & corresponding dry density.

ix. Compaction curve was plotted against water-content as abscissa and dry density as ordinate.

x. Note the water content against the peak of the curve as optimum moisture content for the corresponding dry density as maximum dry density.

3.4 Hydraulic Conductivity (Permeability)

The hydraulic conductivity is the parameter used to assess the infiltration rate of water through soil. The hydraulic conductivity is a measure of the rate at which moisture passes through a soil as first postulated by Darcy's law. The maximum value of hydraulic conductivity, k_{sat}, for a given soil occurs when the soil is completely saturated, and this maximum value is usually used in designs. When a soil is completely saturated, the pore pressures are positive and the hydraulic conductivity is constant, assuming steady flow, constant temperature, and no changes in the water or soil chemistry. When soils are unsaturated, the hydraulic conductivity is less than the saturated hydraulic conductivity and can vary by multiple orders of magnitude as shown in plate I and II.

3.4.1 Apparatus used in conducting hydraulic conductivity tests

Permeameter with its accessories, Soil specimen, Measuring jar, Meter scale, Stopwatch and Container to collect excess water.

3.4.2 Hydraulic conductivity test procedure

i. It was ensured that the valves are closed and the tube connecting the permeameter and the standpipe was tight and also a container was place to collect the excess water.
ii. The top of the standpipe was filled to the zero mark.
iii. Saturation of the soil sample was done by opening the valve for some time till when the water level drops, the pipe was filled up to a zero mark level again.
iv. Measurement from the bottom of the apparatus to the level of zero mark was taken and recorded using measuring tape and recorded as h_0.

v. The valve was open for an interval of five minute to allow a discharge through the soil sample was closed and measurement of new height of water was taken as h_1 and recorded.

vi. The above procedure was repeated at every five minutes time interval for 60min.

vii. Using the recorded readings, graph of $In\left(\frac{h_0}{h_1}\right)$ against $(t_1 - t_0)$ was plotted.

viii. The co efficient of permeability k was determine from the gradient of the graph.

Plate I: Permeability test equipment (the Constructor 2017)

Plate II: Moulded sample set for permeability test

Plate III: Permeability apparatus (The constructor, 2017)

Plate IV: Assembled permeability apparatus

3.5 Atterberg Limit

3.5.1 Liquid limit test

The Atterberg Limit Tests on the soil sample were carried out in accordance with BS 1377, (1990). The liquid limit tests (LL) of the lateritic soil sample were determine using cone penetration apparatus.

3.5.2 Liquid limit test apparatus

Oven, Weighing balance, Sieve 425µm, Cone penetrometer and Spatula. The cone penetrometer is shown in Plate V

Plate V: Cone penetrometer (The constructor, 2017)

3.5.3 Sample preparation for liquid limit test

The sample was air dried at 60ºC temperature and sieve using sieve size 425µm. 150g of the 425µm sieved soil retained is used to carry out the test.

3.5.4 Liquid limit test procedure

i. 150 gm of air dried soil from thoroughly mixed portion of material passing 425µm BS sieve is obtained.
ii. Distilled water was added to mix the soil thus obtained in a mixing disc to form a uniform paste.

iii. Then the wet soil paste was transferred to the cylindrical cup of cone penetrometer apparatus, ensuring that no air is trapped in this process.

iv. Finally the wet soil was leveled up to the top of the cup and placed on the base of the cone penetrometer apparatus.

v. The penetrometer was so adjusted that the cone point just touches the surface of the soil paste in the cup and the initial ready was taken.

vi. The vertical clamp was then released allowing the cone to penetrate into soil paste under its own weight for 5 seconds. After 5 seconds the penetration of the cone was noted to the nearest millimeter.

vii. The test was repeated at least to have five sets of values of penetration in the range of 14 to 28 mm.

viii. The exact moisture content of each trial was determined

3.6 Plastic Limit Test

3.6.1 Sample preparation for plastic limit test

The sample was air dried at 60ºC temperature and sieved using sieve size 425µm. 150g of the 425µm sieved soil retained was used to carry out the test.

3.6.2 Procedure for plastic limit test using hand rolling method

The plastic limit of each soil sample is determined using hand rolling method according to BS 1377, (1990). 30 gm of soil passing through 425µm sieve is mixed with distilled water and left for a suitable time. A ball is formed with about 5 gm of soil paste and rolled into a thread of 3 mm diameter on a glass plate with the fingers of one hand. This procedure of mixing and rolling is repeated till the soil starts crumbling at a diameter of 3 mm. The rate of rolling should be between 80 to 90 strokes per minute to form a 3mm dia. The water content of the crumbled portion of the threads is determined. The test is repeated at least thrice to get the average water content. This average water content is known as the plastic limit.

3.7 Triaxial Test of Lateritic samples

3.7.1 Triaxial test apparatus

Triaxial machine with cells (cylinder with the sizes 76m × 38mm and 100mm × 50mm), Equipment for loading, Equipment to measure load and deformation (dial gauge) and Rubber rings .

3.7.2 Triaxial test procedure

An air-dried soil sample was sieved using 5.0mm and 600g was weighed, the moisture content obtained from the compaction test carried out was used for water addition. The 600g was pour on a metal tray and the value of water obtained from the compaction as added and mixed thoroughly. The triaxial mode of height 76mm and diameter 38mm with the wet sample and give 10 blows each for 3 layers and trimmed to the level of the mould. The soil sample was extruded using sample extruder and rap into a polythene bag to prevent moisture been lost. Three triaxial samples are prepared and used for pressure at 50 kN/m^2, 100kN/m^2 and 150 kN/m^2 each.

The soil sample is sealed with metal caps at the two edge then cover with membrane to prevent water contact. The prepared sample is placed in the compression machine and covered with transparent glass. Water is pumped into the set up sample and the cell is filled. The air pressure is then increased to raise the hydrostatic pressure to 50 kN/m^2. The pressure gauge must be watched to the exact 50 kN/m^2. The handle wheel of the screw jack is rotated until the underside of the hemisphere seating of the proving ring, through which the loading is applied just touch the cell piston.

The piston is moved down by handle until it is just in touch with the pressure plate on top the sample and the proving ring seating is again brought into contact for the beginning of the test. The machine is set into motion to give a rate of strain at an interval of 25mm. At particular interval of strain, dial gauge reading and the corresponding proving ring readings are taking and the corresponding load is determined using proving ring constant. The soil sheared or failed when the dial gauge indicated opposite direction movement and the machine is stopped. The same procedure is followed for 100 kN/m^2 and 150 kN/m^2.

$$\delta_1 = \delta_2 - \delta_3 \tag{3.4}$$

where

δ_1= Principal Stress

δ_2= Compressive Strength, δ_3= Cell Pressure

CHAPTER FOUR

4.0 DISCUSSION OF RESULTS

4.1 Index Properties of Test Samples

Index properties of lateritic samples yielded the results being discussed in this section. There was wide range of values in the index properties of test samples. An average value of 19.32% range of 18.21-20.40 % from Maikunkele borrow pit, an average of 15.70% with range from 13.59-16.45% from Pyata borrow pit and an average of 17.97% with range of 15.70-22.43% from Lapai Gwari borrow pit was obtained for natural moisture content. Details of these test results are on Table 4.1

Table 4.1: Natural moisture content of lateritic soil samples

Location	**Natural moisture (%)**	**Average (%)**
Maikunkele		19.32
A	18.21	
B	19.36	
C	20.40	
Pyata		15.33
A	13.59	
B	16.45	
C	15.99	
Lapai Gwari		17.97
A	22.43	
B	15.70	
C	15.79	

4.2 Atterberg Limits (Consistency Limits)

Liquid Limits (LL) values from consistency tests ranges between 41.51%- 58.85% with mean value of 51.50% and PI ranges from 7.74%- 19.08% with an average mean of 15.59% for Maikunkele borrow pit falls in medium plastic soil which can be used as core for an earth dam. Pyata borrow pit has LL values ranges from 43.71%- 56.61% with mean value of 50.89% with Plasticity index (PI) values ranged from non-plastic (NP) to 16.37% with mean average of 11.96%. Lapai Gwari LL values ranges from 38.69%-49.83% and mean average of Plasticity Index values (PI) were between 10.98%-19.00% with average value of 14.47% and is a medium plastic soil. The summary of the results is shown in Table 4.2.

4.3 Particle Size Distributions (Dry Method) Results

The result of particle size distributions shows that the samples from Maikukele borrow pits are the percentage passing sieve 2.00mm is 82.43%-98.07%, sieve 0.425mm is 53.35%-92.20% and sieve 0.075mm is 45.43%-74.97% with LL ranges from 41.51% - 58.85% and PI 7.74% - 19.08% which falls in range of a medium plastic soil. Pyata borrow pits samples has LL ranges from 43.71% - 56.61% and PI of 9.66% - 16.61% except for sample Pyata A3 of 1.5m depth has LL 53.49% and PI non-plastic. Lapai Gwari borrow pits has LL ranges from 48.44% - 38.69% and PI ranges from 10.98%- 19.08%. The results is shown in Table 4.2.

Table 4.2: Index properties test results

Location	Depth (m)	2.00 (mm)	0.425 (mm)	0.075 (mm)	LL (%)	PI (%)	*AASHITO	USCS
Maik A1	0.5	84.4	74.13	61.13	57.32	15.25	A-7-5	MH
Maik A2	1.0	82.43	73.37	45.17	45.72	17.34	A-7-6	OL
Maik A3	1.5	98.00	89.50	74.97	52.82	19.08	A-7-5	MH
Maik B1	0.5	84.47	71.23	53.83	58.85	15.58	A-7-6	MH
Maik B2	1.0	93.70	85.70	68.13	55.55	13.78	A-7-5	MH
Maik B3	1.5	78.49	53.35	50.20	41.51	16.42	A-7-6	OL
Maik C1	0.5	84.40	74.13	61.13	58.04	18.64	A-7-5	MH
Maik C2	1.0	97.97	92.20	71.33	47.99	16.49	A-7-5	OL
Maik C3	1.5	98.07	83.57	45.43	45.69	7.74	A-5	OL
Pyata A1	0.5	98.50	79.43	66.60	50.00	9.66	A-5	OL

Pyata A2	1.0	98.07	81.60	65.47	43.71	12.80	A-7-5	OL
Pyata A3	1.5	82.36	71.32	60.28	53.49	NP	A-3	OL
Pyata B1	0.5	97.00	92.40	62.13	48.48	13.75	A-7-5	OL
Pyata B2	1.0	88.73	75.80	67.03	51.37	12.56	A-7-5	MH
Pyata B3	1.5	99.30	94.60	62.33	54.06	16.01	A-7-6	MH
Pyata C1	0.5	97.93	87.90	53.73	54.74	16.61	A-7-5	MH
Pyata C2	1.0	88.47	81.20	60.00	56.61	9.92	A-5	OL
Pyata C3	1.5	96.17	92.17	74.57	4 5.57	16.37	A-7-6	OL
L/Gwari A1	0.5	69.50	64.34	59.20	48.26	18.74	A-7-6	OL
L/Gwari A2	1.0	79.33	69.40	57.13	44.25	15.95	A-7-6	OL
L/Gwari A3	1.5	97.30	84.47	52.60	47.18	10.98	A-7-5	OL
L/Gwari B1	0.5	94.03	81.33	64.23	48.44	11.00	A-7-5	OL
L/Gwari B2	1.0	91.03	74.53	66.03	49.83	19.08	A-7-5	OL
L/Gwari B3	1.5	94.77	79.53	57.63	38.69	12.59	A-6	ML
L/Gwari C1	0.5	59.60	51.10	45.30	43.39	16.59	A-7-6	OL
L/Gwari C2	1.0	98.73	83.57	65.80	47.40	19.00	A-7-6	OL
L/Gwari C3	1.5	77.37	67.73	37.20	46.56	16.24	A-7-6	OL

4.4 Compaction Test Results

The compaction characteristics referred to as the moisture density relationship; maximum dry density (MDD) and optimum moisture content (OMC).The range of MDD values were between 16597g/cm^3 – 18097 g/cm^3 with mean value of 17262g/cm^3 for Maikunkele borrow pit and OMC ranges from 12.260%-17.580% with mean OMC of 15.892%, the MDD ranges from 1.6123%-1.9384% with mean value of 1.7668% and the OMC ranges from 11.810%- 18.050% with mean value of 15.1317% for Pyata borrow pit and MDD ranges from 1.7293%-1.9907% with mean value of 1.8471% and the OMC ranges from 9.820%-21.180% the mean value is 14.4167% from Lapai Gwari borrow pit as shown in Table 4.3

4.5 Hydraulic Conductivity Test Results

The results shows that the coefficient of permeability ranges from 10^{-6} -10^{-9} cm/s which is an indication that the soil is of silt to clay. Maikunkele borrow pit has a range of k from 10^{-6} – 10^{-9}cm/s, with mean value of 10^{-8} cm/s. According to Bear 1972, the soil is impervious, that is an indication that the soil is silt and clay, which is a very good embankment fill material to serve as core due to its impermeable nature. Pyata borrow pit has the range of k from 10^{-6} – 10^{-8} cm/s, it is also imperious it shows that the soil is fine sand and silt with little clay content which can only be used as core for the embankment fill. Lapai Gwari borrow pit has a range of k from 10^{-5}- 10^{-8} cm/s that is from semi-pervious to impervious soil.

Casagrande 1940, reported that the lower the hydraulic conductivity the lower the movement of water through the soil, which means from the above result, the soil sample from Maikunkele has higher ability to retain water followed by sample from Pyata while Lapai Gwari has the least ability to retain water as shown on Table 4.4

Table 4.3: Compaction test results

Location	Depth (m)	Maximum dry density (g/cm^3)	Optimum moisture content (%)
Maikunkele A1	0.5	1.7530	17.580
Maikunkele A2	1.0	1.6986	15.610
Maikunkele A3	1.5	1.7258	15.690
Maikunkele B1	0.5	1.6802	15.230
Maikunkele B2	1.0	1.6597	18.210
Maikunkele B3	1.5	1.8097	12.260
Maikunkele C1	0.5	1.7018	16.930
Maikunkele C2	1.0	1.7349	17.540

Maikunkele C3	1.5	1.7723	13.975
Pyata A1	0.5	1.7584	18.050
Pyata A2	1.0	1.7297	16.550
Pyata A3	1.5	1.7874	14.940
Pyata B1	0.5	1.6869	15.690
Pyata B2	1.0	1.6123	18.980
Pyata B3	1.5	1.6778	12.790
Pyata C1	0.5	1.9378	13.470
Pyata C2	1.0	1.9384	11.810
Pyata C3	1.5	1.7727	13.450
Lapai Gwari A1	0.5	1.8260	11.210
Lapai Gwari A2	1.0	1.8793	10.770
Lapai Gwari A3	1.5	1.7254	21.180
Lapai Gwari B1	0.5	1.8026	14.620
Lapai Gwari B2	1.0	1.8797	9.820
Lapai Gwari B3	1.5	1.7293	13.920
Lapai Gwari C1	0.5	1.9907	10.330
Lapai Gwari C2	1.0	1.8601	12.130
Lapai Gwari C3	1.5	1.9304	12.320

Table 4.4: Hydraulic conductivity of test samples

Location	Depth (m)	Value of K(cm/s)
Maikunkele A1	0.5	1.5909×10^{-7}
Maikunkele A2	1.0	1.2113×10^{-8}
Maikunkele A3	1.5	7.494×10^{-7}
Maikunkele B1	0.5	3.1580×10^{-6}
Maikunkele B2	1.0	1.5606×10^{-6}
Maikunkele B3	1.5	1.4349×10^{-8}
Maikunkele C1	0.5	2.3802×10^{-9}

Maikunkele C2	1.0	$1.5203x10^{-8}$
Maikunkele C3	1.5	$2.3920x10^{-6}$
Pyata A1	0.5	$2.7790x10^{-7}$
P-yata A2	1.0	$6.9480x10^{-8}$
Pyata A3	1.5	$9.4349x10^{-8}$
Pyata B1	0.5	$8.1060x10^{-6}$
Pyata B2	1.0	$1.4825x10^{-8}$
Pyata B3	1.5	$1.3021x10^{-6}$
Pyata C1	0.5	$2.1210x10^{-6}$
Pyata C2	1.0	$1.8230x10^{-8}$
Pyata C3	1.5	$1.5100x10^{-7}$
Lapai Gwari A1	0.5	$2.7790x10^{-6}$
Lapai Gwari A2	1.0	$1.3890x10^{-5}$
Lapai Gwari A3	1.5	$2.9480x10^{-6}$
Lapai Gwari B1	0.5	$1.3202x10^{-7}$
Lapai Gwari B2	1.0	$2.29902x10^{-7}$
Lapai Gwari B3	1.5	$3.0364x10^{-8}$
Lapai Gwari C1	0.5	$1.6213x10^{-7}$
Lapai Gwari C2	1.0	$2.0265x10^{-7}$
Lapai Gwari C3	1.5	$2.3161x10^{-7}$

4.6 Shear Strength Test Result

Lateritic samples (A-3 to A-5) obtained from 1.0m depth, which contain fairly granular materials has angle of internal friction (ϕ) is 22.73º and cohesion (c) is 94.46 kN/m^2. Samples from 1.5m depth (A-5, A-6 and A-7) contain mostly silt and clayey materials has angle of internal friction (ϕ) 12.27º and 18.36º, and cohesion (c) of 148.67 kN/m^2 and 60.98 kN/m^2 respectively. Furthermore A-7-5 and A-7-6 soil obtained also contain silt and clayey materials mostly with angle of internal friction (ϕ) 23.87º and 25.78º, and cohesion (c) of 67.37 kN/m^2 and 125.49 kN/m^2.

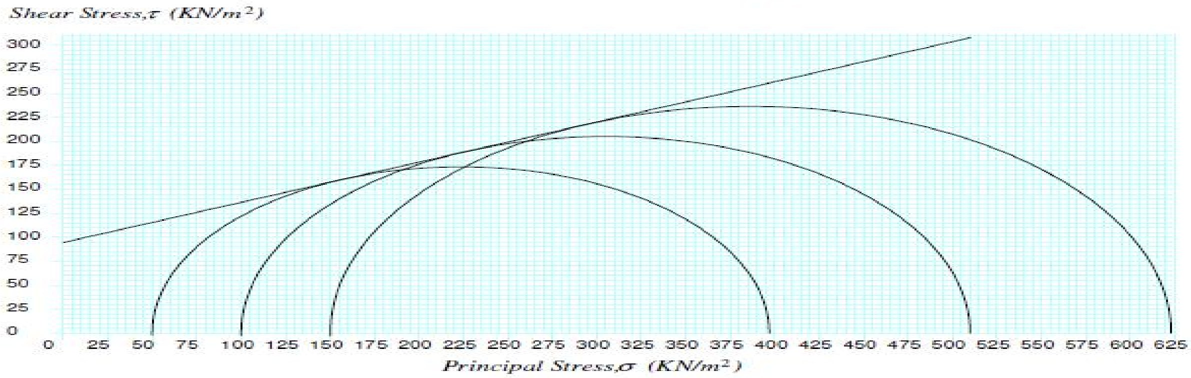
Most of the lateritic soil, which possessed good characteristic as fill material, were those collected at 1.0m depth from the earth surface. Geotechdata information (2013) and United State Bureau of Reclamation (2006) shows that from the above result, the soil ranges from inorganic silt to inorganic clay of high plasticity (OL to MH) according to Unified Soil Classification System (USCS). Specific values of 22^0 as minimum and 32^0 as maximum for OL soil and 23^0 as minimum and 33^0 as maximum for MH soil with

Fell *et al*. (2015) explained that for a stable embankment slope, the most exhibit low angle of friction and relative high cohesion.

The results of trial-axial test and Mohr-circle diagram for the representative samples is shown in Figure 4.1 – 4.5

Test no:	Cell Pressure, σ_3 (KN/m^2)	Compressive Strength, σ_2 (KN/m^2)	Principal Stress, σ_1 (KN/m^2)	Bulk Density	Moisture Content
1	50	347,00	397	17.53	14.94
2	100	410,00	510	17.53	14.94
3	150	473,00	623	17.53	14.94

ϕ = 22,73 c = 94,46

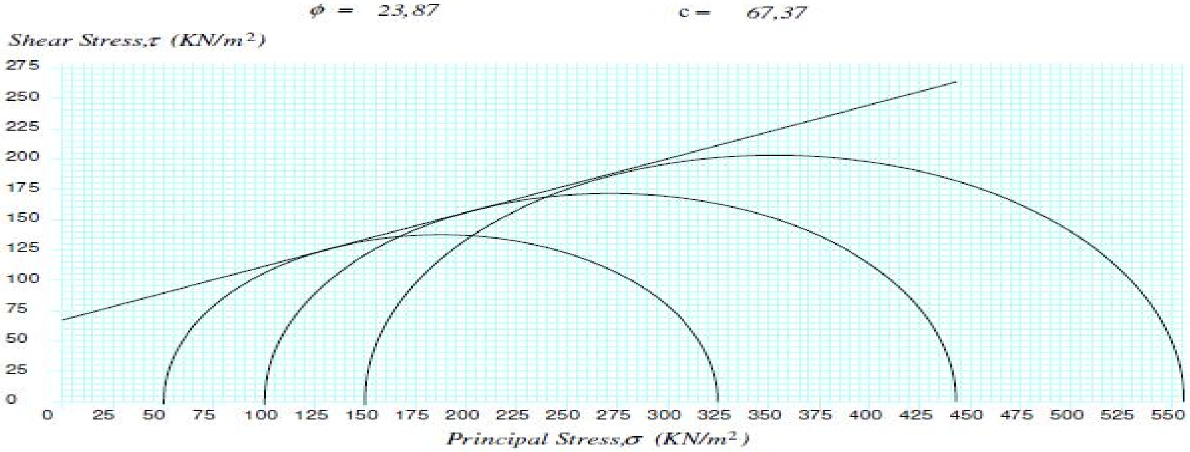


Test executed by : Engr. Ramatu Ibrahim

Figure 4.1: Mohr circle diagram for sample MK B-2-1

Test no:	Cell Pressure, σ_3 (KN/m^2)	Compressive Strength, σ_2 (KN/m^2)	Principal Stress, σ_1 (KN/m^2)	Bulk Density	Moisture Content
1	50	275,00	325	17.68	14.62
2	100	343,00	443	17.68	14.62
3	150	406,00	556	17.68	14.62

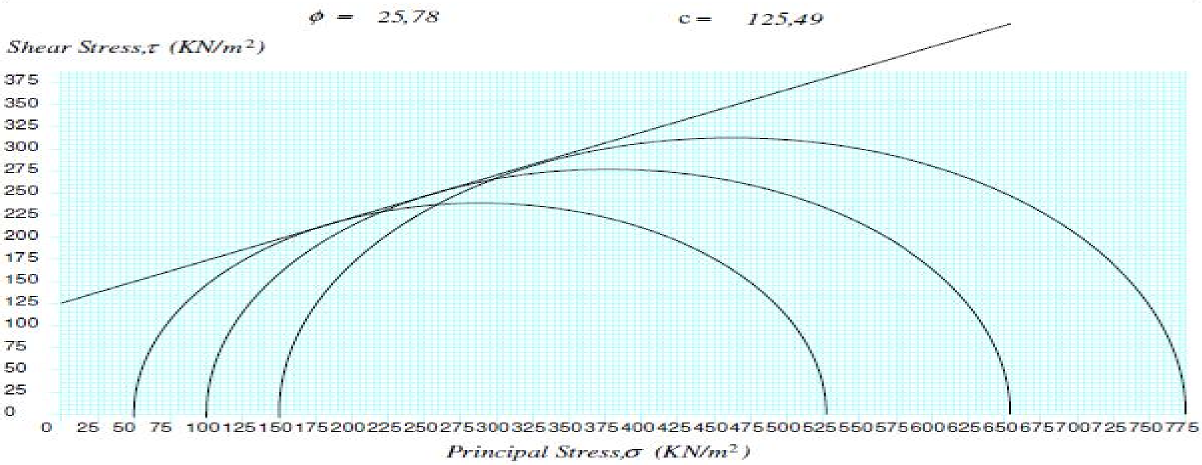
ϕ = 23,87 c = 67,37



Test executed by : Engr. Ramatu Ibrahim

Figure 4.2: Mohr circle diagram for sample MK B-2-2

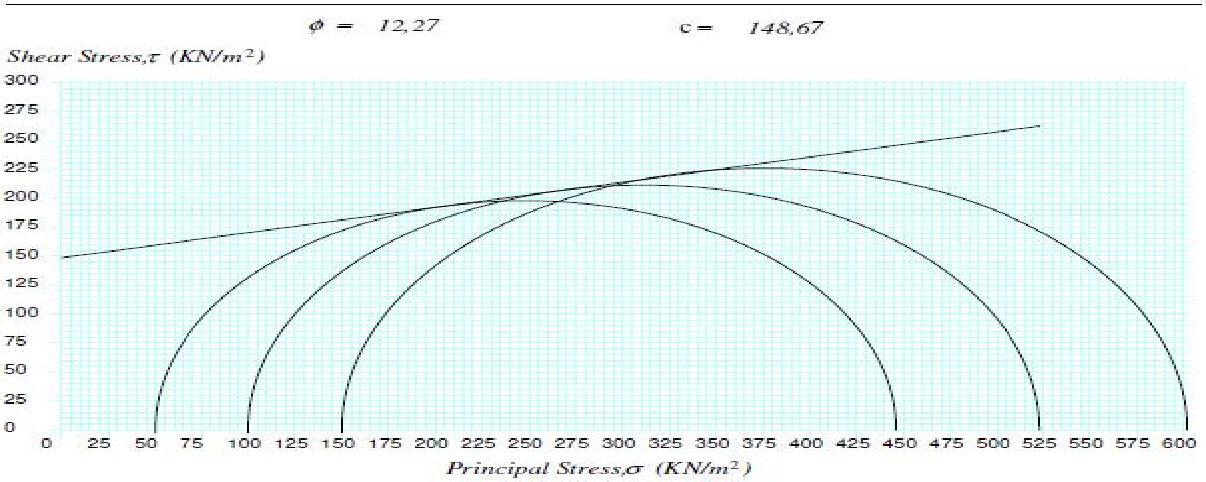
Test no:	Cell Pressure, σ_3 (KN/m^2)	Compressive Strength, σ_2 (KN/m^2)	Principal Stress, σ_1 (KN/m^2)	Bulk Density	Moisture Content
1	50	477,00	527	17.75	12.26
2	100	554,00	654	17.75	12.26
3	150	625,00	775	17.75	12.26



Test executed by : Engr. Ramatu Ibrahim

Figure 4.3: Mohr circle diagram for sample PY-2-1

Test no:	Cell Pressure, σ_3 (KN/m^2)	Compressive Strength, σ_2 (KN/m^2)	Principal Stress, σ_1 (KN/m^2)	Bulk Density	Moisture Content
1	50	396,00	446	19.02	11.81
2	100	423,00	523	19.02	11.81
3	150	452,00	602	19.02	11.81

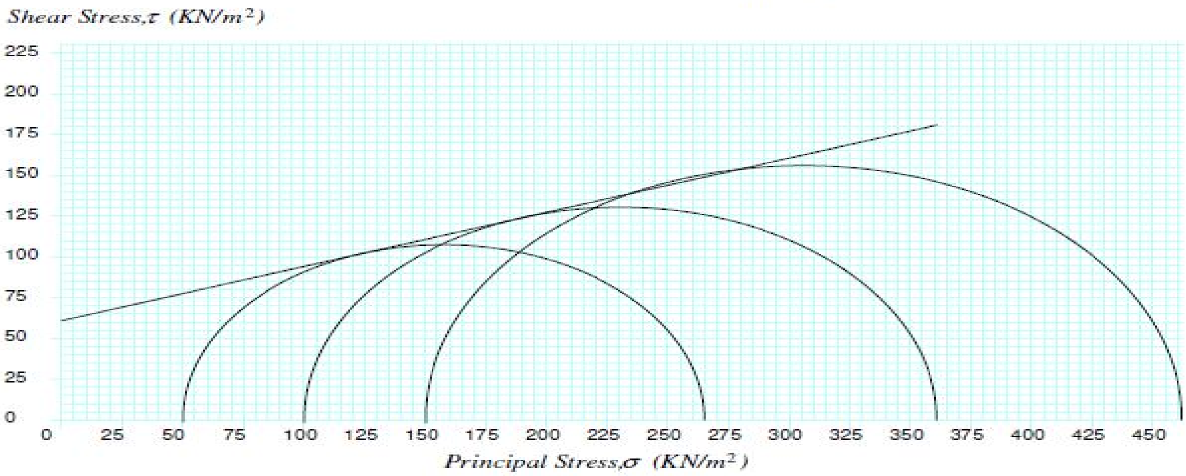


Test executed by : Engr. Ramatu Ibrahim

Figure 4.4: Mohr circle diagram for sample PY-2-2

Test no:	Cell Pressure, σ_3 (KN/m^2)	Compressive Strength, σ_2 (KN/m^2)	Principal Stress, σ_1 (KN/m^2)	Bulk Density	Moisture Content
1	50	215,00	265	16.96	13.92
2	100	261,00	361	16.96	13.92
3	150	312,00	462	16.96	13.92

$\phi =$ 18,36 c = 60,98



Test executed by : Engr. Ramatu Ibrahim

Figure 4.5: Mohr circle diagram for sample LG-2-1

Figure 4.1 is the Mohr circle diagram for sample collected at Maikunkele at a depth of 0.5m with shear stress ϕ of 22.73 kN/m^2 and cohesion c of 94.49˚, figure 4.2 is the Mohr circle diagram for sample collected from Maikukele at a depth of 1.0m having shear stress ϕ of 23.87 kN/m^2 and cohesion c of 67.37˚, figure 4.3 is the Mohr circle diagram for sample collected from Pyata at the depth of 0.5m with shear stress of ϕ 22.78 kN/m^2 and cohesion c of 125.49˚, figure 4.4 is the Mohr circle diagram for sample collected at 1.0m depth with shear stress ϕ of 12.27 kN/m^2 and cohesion c of 148.67˚ and figure 4.5 is the Mohr circle diagram of sample collected from Lapai Gwari at a depth of with shear stress ϕ of 18.36 kN/m^2 and cohesion c of 60.98˚

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

From the assessment of geotechnical properties of selected lateritic soil as fill materials for earth construction, the following conclusions were drawn:

About 96% of the test samples are good fill materials for earth dam construction for maximum functionality. Their classification range between A-6, A-7 and A-5, or CH - MH soils according to AASHTO and USCS classification systems respectively.

The compaction characteristics, MDD ranged between 1650g/cm^3 to 1990 g/cm^3, with the OMC range of 9.8 – 21.2%. The work further revealed average values of Cohesion (*C*) and angle of internal friction (φ) as 397.7 kN/m^2, 99.49 kN/m^2 and 20.6° respectively.

The hydraulic conductivity of test samples ranged from 10^{-5} cm/s - 10^{-9} cm/s, which indicated and grouped the test lateritic soil as silty clay to clay. The soil is impervious and very good embankment fill material to serve as dam core and embankment fill due to its impermeable nature.

Lateritic samples from borrow pits in Maikunkele, Pyata and Lapai Gwari suburbs of Minna, Niger state, or from other places with exact or nearly exact geotechnical properties, could be used as fill materials for construction of earth dam.

5.2 Recommendations

From the assessment of geotechnical properties selected lateritic soil as fill materials for earth construction, the following conclusions are given;

1. Most of the lateritic soil, which possessed good characteristic as fill material, were those collected at 1.0m depth from the earth surface. Therefore, Lateritic soil around 1.0m depth are recommended for use as fill material.

2. From the Strength and hydraulic conductivity test results, some of the test lateritic materials will may require a form of stabilization and enhancement if the required strength is slightly higher.

3. Proper test should be carried out on lateritic soil intended to be used as fill material for earth dam construction.

5.3 Contribution to Knowledge

The work established that test lateritic samples in Minna environ ranged between gap-graded and poorly graded. It revealed MDD of 1,6597g/cm^3 to 1,8097g/cm^3, and the OMC of 9.8% to 21.2%. The hydraulic conductivity (*k*) ranged from 10^{-5} cm/s to 10^{-9} cm/s. The work also revealed average values of confined compressive strength (*CCS*), Cohesion (C), angle of internal friction (ϕ) as 397.7 kNs/m^2, 99.49 kN/m^2 and 20.6^0 respectively. It further established the soil test samples as impervious and such useful as fill materials for application as core, shell and blanket for slope protection in zonation of embankment and earth dam.

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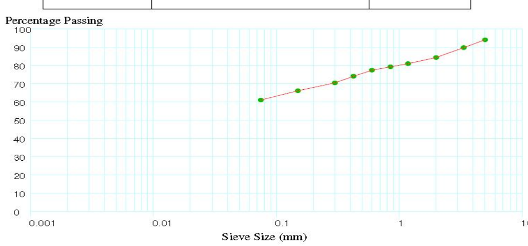
APPENDIX A

Sieve analysis charts

GRAIN SIZE ANALYSIS

Project :
Test Location : MAIKUNKELE - A1 - 0.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

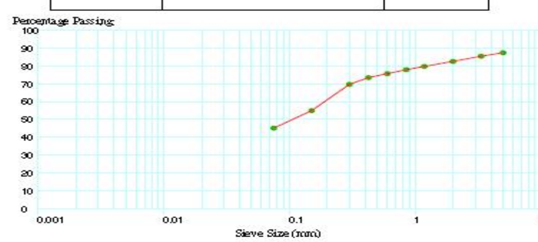
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	17.7	5.90	94.10
3.350	12.9	4.30	89.80
2.000	16.2	5.40	84.40
1.180	10.2	3.40	81.00
0.850	5.1	1.70	79.30
0.600	5.6	1.87	77.43
0.425	9.9	3.30	74.13
0.300	10.8	3.60	70.53
0.150	13	4.33	66.20
0.075	15.2	5.07	61.13



GRAIN SIZE ANALYSIS

Project :
Test Location : MAIKUNKELE - A2 - 1.0m
Sample no. : Initial Sample mass : 300 g Date :
Sample Description :

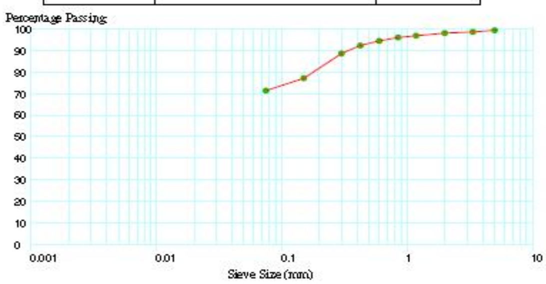
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	38.2	12.73	87.27
3.350	5.8	1.93	85.33
2.000	8.7	2.90	82.43
1.180	8.4	2.80	79.63
0.850	5.5	1.83	77.80
0.600	6.6	2.20	75.60
0.425	6.7	2.23	73.37
0.300	11.3	3.77	69.60
0.150	44	14.67	54.93
0.075	29.3	9.77	45.17



GRAIN SIZE ANALYSIS

Project :	
Test Location : MAIKUNKELE - A2 - 1.0m	
Sample no. :	**Initial Sample mass :** 300 g **Date :**
Sample Description :	

Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	2.3	0.77	99.23
3.350	2.2	0.73	98.50
2.000	1.6	0.53	97.97
1.180	3.7	1.23	96.73
0.850	2.4	0.80	95.93
0.600	4.8	1.60	94.33
0.425	6.4	2.13	92.20
0.300	11.1	3.70	88.50
0.150	34.4	11.47	77.03
0.075	17.1	5.70	71.33



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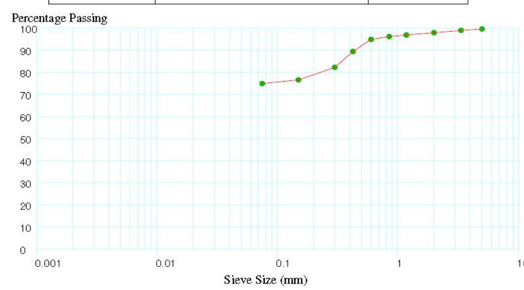
GRAIN SIZE ANALYSIS

Project :	
Test Location : MAIKUNKELE - A3 - 1.5m	
Sample no. :	**Initial Sample mass :** 300 g **Date :**
Sample Description :	

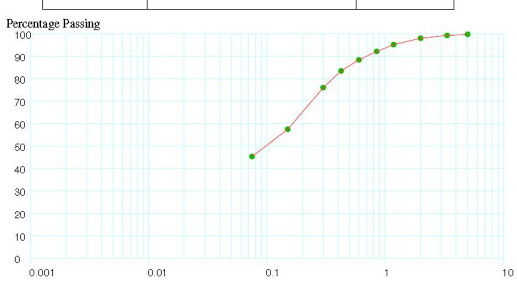
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	1.1	0.37	99.63
3.350	1.9	0.63	99.00
2.000	3	1.00	98.00
1.180	3.2	1.07	96.93
0.850	2.1	0.70	96.23
0.600	3.9	1.30	94.93
0.425	16.3	5.43	89.50
0.300	21.5	7.17	82.33
0.150	17.1	5.70	76.63
0.075	5	1.67	74.97



GRAIN SIZE ANALYSIS

Project :	
Test Location : MAIKUNKELE - B3- 1.5m	
Sample no. :	**Initial Sample mass :** 300 g **Date :**
Sample Description :	

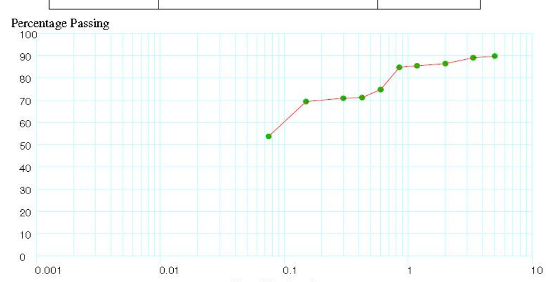
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	0.5	0.17	99.83
3.350	1.5	0.50	99.33
2.000	3.8	1.27	98.07
1.180	8.4	2.80	95.27
0.850	9.1	3.03	92.23
0.600	11.4	3.80	88.43
0.425	14.6	4.87	83.57
0.300	22.4	7.47	76.10
0.150	55.7	18.57	57.53
0.075	36.3	12.10	45.43



GRAIN SIZE ANALYSIS

Project :	
Test Location : MAIKUNKELE - B1- 0.5m	
Sample no. :	**Initial Sample mass :** 300 g **Date :**
Sample Description :	

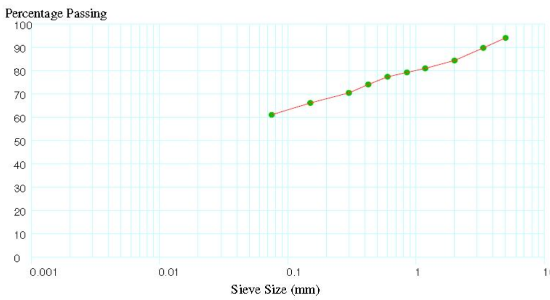
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	30.5	10.17	89.83
3.350	2.2	0.73	89.10
2.000	7.9	2.63	86.47
1.180	3	1.00	85.47
0.850	2	0.67	84.80
0.600	29.9	9.97	74.83
0.425	10.8	3.60	71.23
0.300	0.9	0.30	70.93
0.150	4.5	1.50	69.43
0.075	46.8	15.60	53.83



GRAIN SIZE ANALYSIS

Project :
Test Location : MAIKUNKELE - C1- 0.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

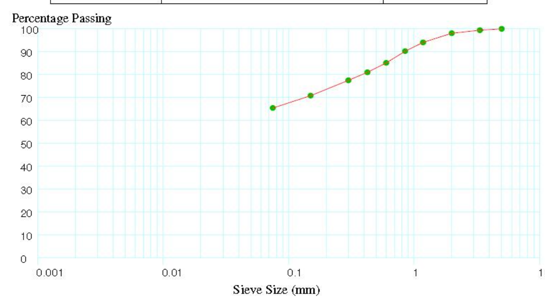
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	17.7	5.90	94.10
3.350	12.9	4.30	89.80
2.000	16.2	5.40	84.40
1.180	10.2	3.40	81.00
0.850	5.1	1.70	79.30
0.600	5.6	1.87	77.43
0.425	9.9	3.30	74.13
0.300	10.8	3.60	70.53
0.150	13	4.33	66.20
0.075	15.2	5.07	61.13



GRAIN SIZE ANALYSIS

Project :
Test Location : PYATA - A2 - 1.0m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

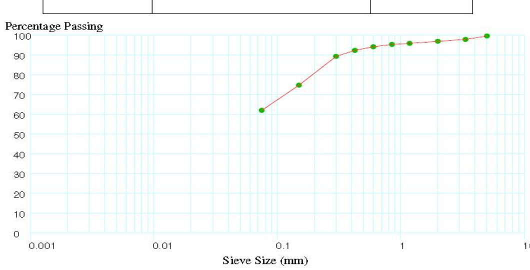
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	0.1	0.03	99.97
3.350	1.7	0.57	99.40
2.000	4	1.33	98.07
1.180	12	4.00	94.07
0.850	11.4	3.80	90.27
0.600	15.4	5.13	85.13
0.425	12.4	4.13	81.00
0.300	10.5	3.50	77.50
0.150	19.9	6.63	70.87
0.075	16.2	5.40	65.47



GRAIN SIZE ANALYSIS

Project :
Test Location : PYATA - B2 - 0.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

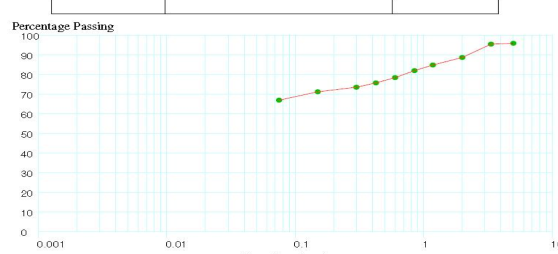
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	1	0.33	99.67
3.350	5.2	1.73	97.93
2.000	2.8	0.93	97.00
1.180	3.2	1.07	95.93
0.850	1.6	0.53	95.40
0.600	3.5	1.17	94.23
0.425	5.5	1.83	92.40
0.300	9.1	3.03	89.37
0.150	43.6	14.53	74.83
0.075	38.1	12.70	62.13



GRAIN SIZE ANALYSIS

Project :
Test Location : PYATA - B2 - 1.0m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

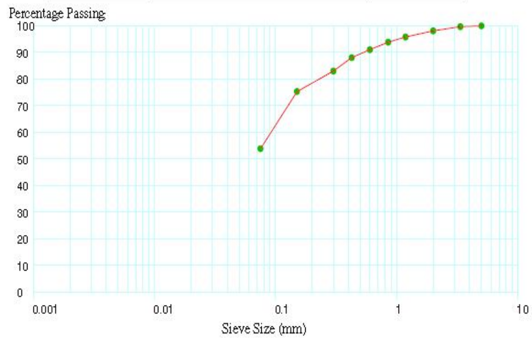
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	12.1	4.03	95.97
3.350	1.4	0.47	95.50
2.000	20.3	6.77	88.73
1.180	11.5	3.83	84.90
0.850	8.5	2.83	82.07
0.600	10.7	3.57	78.50
0.425	8.1	2.70	75.80
0.300	6.6	2.20	73.60
0.150	6.9	2.30	71.30
0.075	12.8	4.27	67.03



GRAIN SIZE ANALYSIS

Project :
Test Location : PYATA - C1 - 0.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

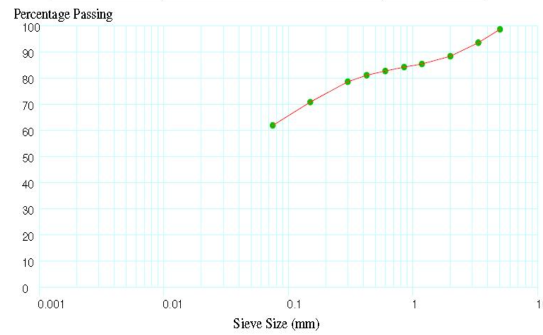
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	0.6	0.20	99.80
3.350	0.9	0.30	99.50
2.000	4.7	1.57	97.93
1.180	6.9	2.30	95.63
0.850	5.8	1.93	93.70
0.600	8.5	2.83	90.87
0.425	8.9	2.97	87.90
0.300	15.1	5.03	82.87
0.150	23.1	7.70	75.17
0.075	64.3	21.43	53.73



GRAIN SIZE ANALYSIS

Project :
Test Location : PYATA - C2 - 1.0m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

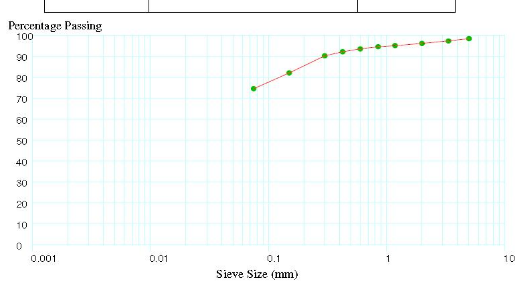
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	3.7	1.23	98.77
3.350	15.4	5.13	93.63
2.000	15.5	5.17	88.47
1.180	8.9	2.97	85.50
0.850	3.6	1.20	84.30
0.600	4.5	1.50	82.80
0.425	4.8	1.60	81.20
0.300	7.5	2.50	78.70
0.150	23.3	7.77	70.93
0.075	26.8	8.93	62.00



GRAIN SIZE ANALYSIS

Project :
Test Location : PYATA - C3 - 1.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

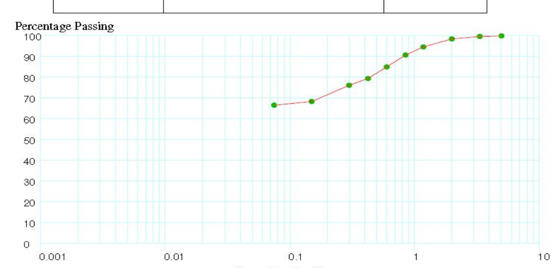
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	4.7	1.57	98.43
3.350	3.3	1.10	97.33
2.000	3.5	1.17	96.17
1.180	3.2	1.07	95.10
0.850	1.6	0.53	94.57
0.600	3	1.00	93.57
0.425	4.2	1.40	92.17
0.300	5.8	1.93	90.23
0.150	24.4	8.13	82.10
0.075	22.6	7.53	74.57



GRAIN SIZE ANALYSIS

Project :
Test Location : PYATA - C2 - 0.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

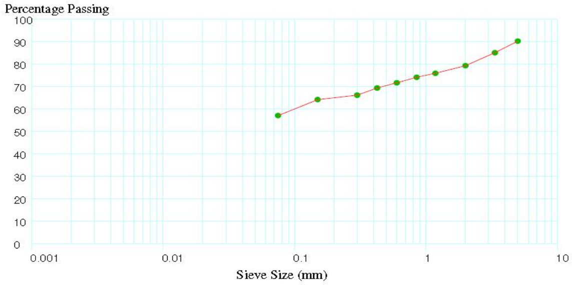
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	0.2	0.07	99.93
3.350	0.8	0.27	99.67
2.000	3.5	1.17	98.50
1.180	11.6	3.87	94.63
0.850	11.8	3.93	90.70
0.600	17.1	5.70	85.00
0.425	16.7	5.57	79.43
0.300	9.8	3.27	76.17
0.150	23.4	7.80	68.37
0.075	5.4	1.80	66.57



GRAIN SIZE ANALYSIS

Project :
Test Location : LAPAI GWARI - A2 - 1.0m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

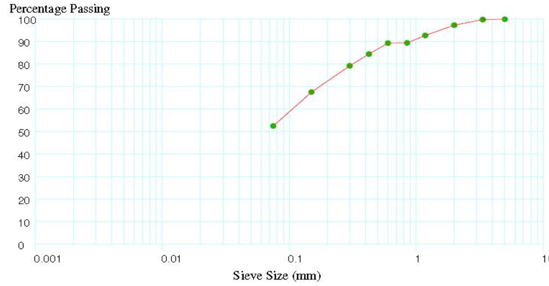
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	29.2	9.73	90.27
3.350	15.5	5.17	85.10
2.000	17.3	5.77	79.33
1.180	10.2	3.40	75.93
0.850	5.3	1.77	74.17
0.600	7.4	2.47	71.70
0.425	6.9	2.30	69.40
0.300	9.6	3.20	66.20
0.150	5.9	1.97	64.23
0.075	21.3	7.10	57.13



GRAIN SIZE ANALYSIS

Project :
Test Location : LAPAI GWARI - A3- 1.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

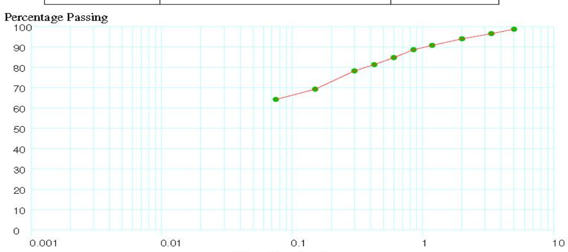
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	0.3	0.10	99.90
3.350	0.4	0.13	99.77
2.000	7.4	2.47	97.30
1.180	13.8	4.60	92.70
0.850	9.8	3.27	89.43
0.600	0.3	0.10	89.33
0.425	14.6	4.87	84.47
0.300	15.7	5.23	79.23
0.150	34.8	11.60	67.63
0.075	45.1	15.03	52.60



GRAIN SIZE ANALYSIS

Project :
Test Location : LAPAI GWARI - B1- 0.5m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

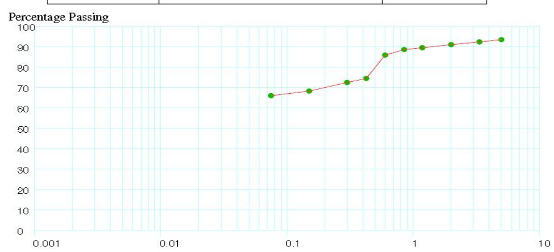
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	3.8	1.27	98.73
3.350	6.5	2.17	96.57
2.000	7.6	2.53	94.03
1.180	9.7	3.23	90.80
0.850	6.4	2.13	88.67
0.600	11.6	3.87	84.80
0.425	10.4	3.47	81.33
0.300	9.1	3.03	78.30
0.150	27	9.00	69.30
0.075	15.2	5.07	64.23



GRAIN SIZE ANALYSIS

Project :
Test Location : LAPAI GWARI - B2- 1.0m
Sample no. : **Initial Sample mass :** 300 g **Date :**
Sample Description :

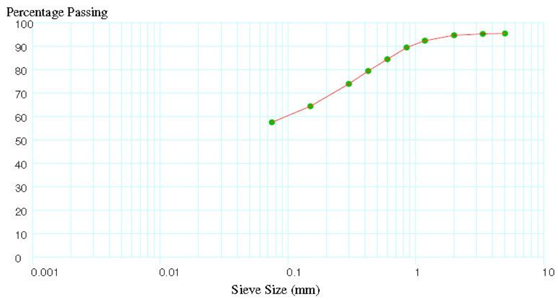
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	19.5	6.50	93.50
3.350	3.3	1.10	92.40
2.000	4.1	1.37	91.03
1.180	4.5	1.50	89.53
0.850	2.7	0.90	88.63
0.600	8.1	2.70	85.93
0.425	34.2	11.40	74.53
0.300	6.1	2.03	72.50
0.150	12.5	4.17	68.33
0.075	6.9	2.30	66.03



GRAIN SIZE ANALYSIS

Project :		
Test Location : LAPAI GWARI - B3- 1.5m		
Sample no. :	Initial Sample mass : 300 g	Date :
Sample Description :		

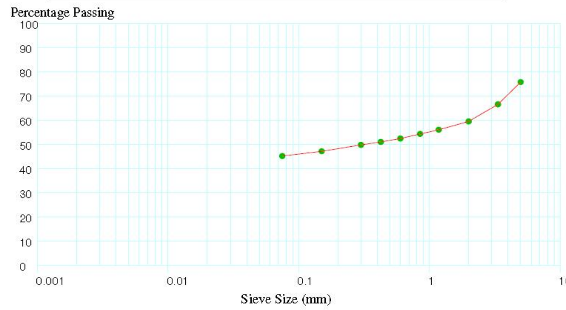
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	13.4	4.47	95.53
3.350	0.6	0.20	95.33
2.000	1.7	0.57	94.77
1.180	7	2.33	92.43
0.850	8.7	2.90	89.53
0.600	15	5.00	84.53
0.425	15	5.00	79.53
0.300	16.6	5.53	74.00
0.150	28.5	9.50	64.50
0.075	20.6	6.87	57.63



GRAIN SIZE ANALYSIS

Project :		
Test Location : LAPAI GWARI - C1- 0.5m		
Sample no. :	Initial Sample mass : 300 g	Date :
Sample Description :		

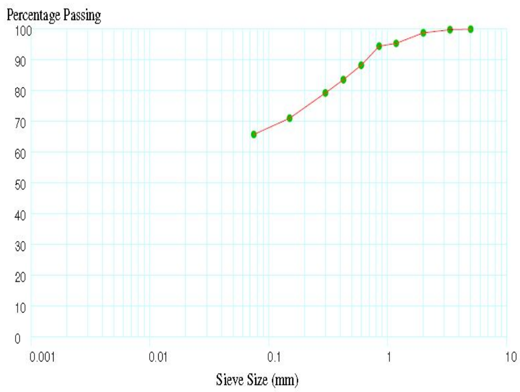
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	72.5	24.17	75.83
3.350	27.6	9.20	66.63
2.000	21.1	7.03	59.60
1.180	10.3	3.43	56.17
0.850	5.2	1.73	54.43
0.600	5.7	1.90	52.53
0.425	4.3	1.43	51.10
0.300	3.7	1.23	49.87
0.150	7.8	2.60	47.27
0.075	5.9	1.97	45.30



GRAIN SIZE ANALYSIS

Project :	
Test Location : LAPAI GWARI - C2- 1m	
Sample no. :	**Initial Sample mass :** 300 g **Date :**
Sample Description :	

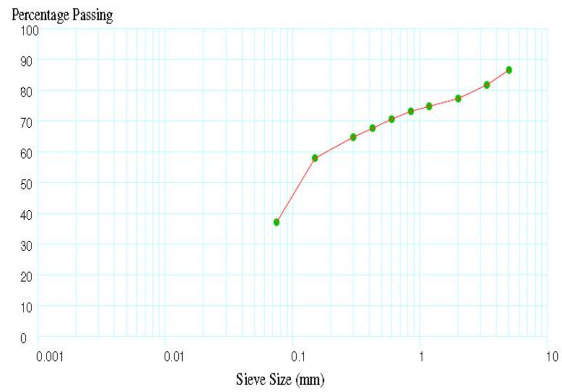
Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	0.2	0.07	99.93
3.350	0.5	0.17	99.77
2.000	3.1	1.03	98.73
1.180	10.2	3.40	95.33
0.850	2.7	0.90	94.43
0.600	18.6	6.20	88.23
0.425	14	4.67	83.57
0.300	12.9	4.30	79.27
0.150	24.6	8.20	71.07
0.075	15.8	5.27	65.80



GRAIN SIZE ANALYSIS

Project :	
Test Location : LAPAI GWARI - C3- 1.5m	
Sample no. :	**Initial Sample mass :** 300 g **Date :**
Sample Description :	

Sieve size (mm)	Mass retained (g)	% Retained	% Passing
5.000	40.1	13.37	86.63
3.350	14.6	4.87	81.77
2.000	13.2	4.40	77.37
1.180	7.6	2.53	74.83
0.850	4.9	1.63	73.20
0.600	7.6	2.53	70.67
0.425	8.8	2.93	67.73
0.300	8.7	2.90	64.83
0.150	20.4	6.80	58.03
0.075	62.5	20.83	37.20



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APPENDIX B

Liquid limit determination

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

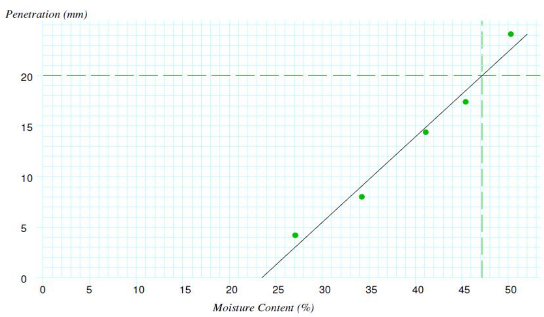
Project :

Test Location : LAPAI GWARI - A1

Sample no. : **Depth of sample :** 1.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	4.2	8.0	14.4	17.4	24.1		
Can Weight	37.39	37.25	35.70	36.31	36.21	36.48	36.29
Weight of Can + Wet Soil	51.93	54.34	55.81	56.74	61.29	41.19	40.95
Weight of Can + Dry Soil	48.83	49.98	49.95	50.36	52.90	39.95	39.70
Weight of Moisture	3.10	4.36	5.86	6.38	8.39	1.24	1.25
Weight of Dry Soil	11.44	12.73	14.25	14.05	16.69	3.47	3.41
Moisture Content	27.10	34.25	41.12	45.41	50.27	35.73	36.66
Liquid Limit	**47.18 %**		Average Plastic Limit :			**36.20 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

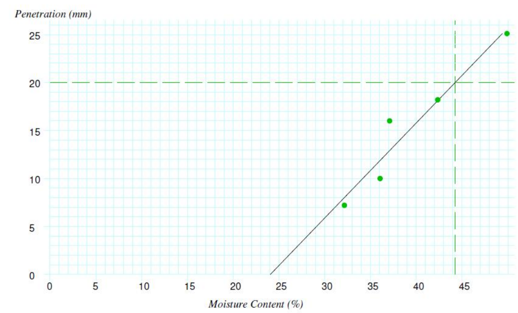
Project :

Test Location : LAPAI GWARI - A2

Sample no. : **Depth of sample :** 1.0m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	7.2	10.0	16.0	18.2	25.1		
Can Weight	37.52	36.40	38.17	36.08	32.88	37.58	37.46
Weight of Can + Wet Soil	50.92	49.91	57.28	57.46	65.65	43.71	42.66
Weight of Can + Dry Soil	47.66	46.33	52.11	51.10	54.74	42.35	41.52
Weight of Moisture	3.26	3.58	5.17	6.36	10.91	1.36	1.14
Weight of Dry Soil	10.14	9.93	13.94	15.02	21.86	4.77	4.06
Moisture Content	32.15	36.05	37.09	42.34	49.91	28.51	28.08
Liquid Limit	**44.25 %**		Average Plastic Limit :			**28.30 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

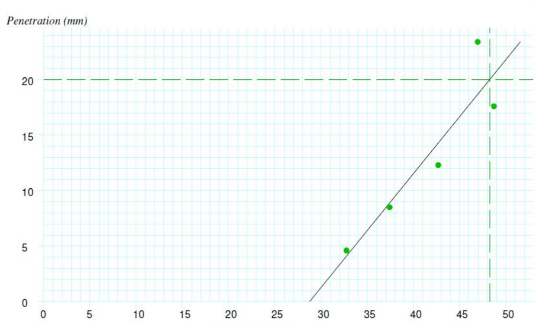
Project :

Test Location : LAPAI GWARI - A1

Sample no. : **Depth of sample :** 0.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	4.6	8.5	12.3	17.6	23.4		
Can Weight	36.80	37.92	37.60	36.20	37.22	35.27	36.49
Weight of Can + Wet Soil	50.13	55.18	54.48	61.02	66.20	41.18	41.26
Weight of Can + Dry Soil	46.84	50.48	49.43	52.89	56.94	39.80	40.20
Weight of Moisture	3.29	4.70	5.05	8.13	9.26	1.38	1.06
Weight of Dry Soil	10.04	12.56	11.83	16.69	19.72	4.53	3.71
Moisture Content	32.77	37.42	42.69	48.71	46.96	30.46	28.57
Liquid Limit	**48.26 %**		Average Plastic Limit :			**29.52 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

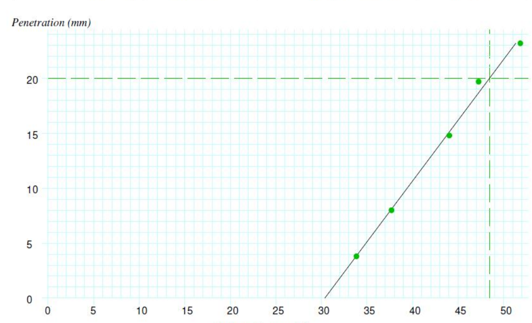
Project :

Test Location : LAPAI GWARI - B1

Sample no. : **Depth of sample :** 0.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	3.8	8.0	14.8	19.7	23.2		
Can Weight	36.72	37.70	37.62	36.39	36.87	38.15	37.36
Weight of Can + Wet Soil	49.93	50.96	56.82	59.98	62.59	43.09	42.26
Weight of Can + Dry Soil	46.59	47.33	50.95	52.41	53.81	41.76	40.91
Weight of Moisture	3.34	3.63	5.87	7.57	8.78	1.33	1.35
Weight of Dry Soil	9.87	9.63	13.33	16.02	16.94	3.61	3.55
Moisture Content	33.84	37.69	44.04	47.25	51.83	36.84	38.03
Liquid Limit	**48.44 %**		Average Plastic Limit :			**37.44 %**	

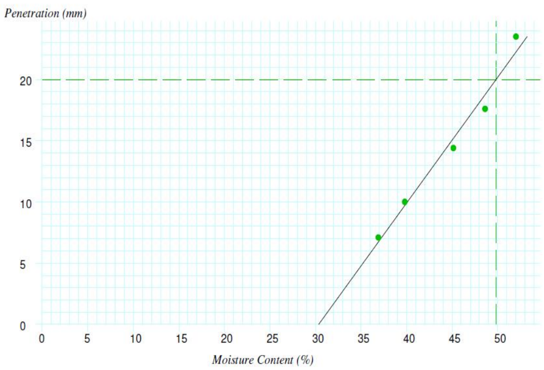


Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : LAPAI GWARI -B2		
Sample no. :	**Depth of sample :** 1.0m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	7.1	10.0	14.4	17.6	23.5		
Can Weight	35.86	36.26	36.45	32.85	37.29	35.65	36.24
Weight of Can + Wet Soil	46.99	48.87	53.78	54.34	66.84	41.32	42.05
Weight of Can + Dry Soil	43.99	45.28	48.39	47.31	56.73	39.98	40.69
Weight of Moisture	3.00	3.59	5.39	7.03	10.11	1.34	1.36
Weight of Dry Soil	8.13	9.02	11.94	14.46	19.44	4.33	4.45
Moisture Content	36.90	39.80	45.14	48.62	52.01	30.95	30.56
Liquid Limit	**49.83 %**		Average Plastic Limit :			**30.75 %**	

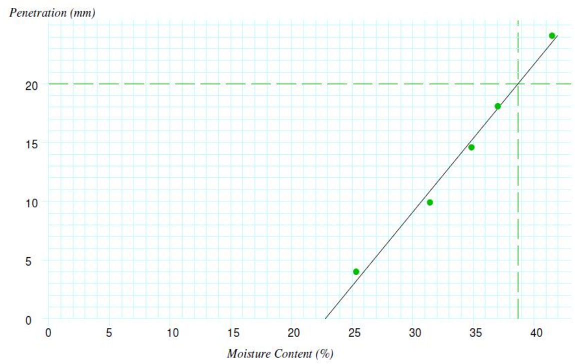


Test executed by: ____________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : LAPAI GWARI -B3		
Sample no. :	**Depth of sample :** 1.5m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	4.0	9.9	14.6	18.1	24.1		
Can Weight	35.31	34.48	34.01	37.52	34.64	36.90	34.09
Weight of Can + Wet Soil	44.41	54.26	50.84	58.43	62.53	41.87	39.80
Weight of Can + Dry Soil	42.57	49.53	46.49	52.78	54.35	40.84	38.62
Weight of Moisture	1.84	4.73	4.35	5.65	8.18	1.03	1.18
Weight of Dry Soil	7.26	15.05	12.48	15.26	19.71	3.94	4.53
Moisture Content	25.34	31.43	34.86	37.02	41.50	26.14	26.05
Liquid Limit	**38.69 %**		Average Plastic Limit :			**26.10 %**	

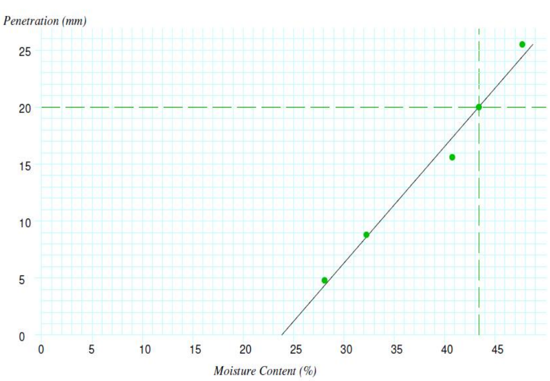


Test executed by: ____________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : LAPAI GWARI - C1		
Sample no. :	**Depth of sample :** 0.5m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	4.8	8.8	15.6	20.0	25.5		
Can Weight	37.75	37.35	36.92	37.81	37.66	38.33	36.37
Weight of Can + Wet Soil	46.87	49.49	53.22	60.01	67.69	43.46	42.92
Weight of Can + Dry Soil	44.87	46.53	48.50	53.29	57.99	42.38	41.53
Weight of Moisture	2.00	2.96	4.72	6.72	9.70	1.08	1.39
Weight of Dry Soil	7.12	9.18	11.58	15.48	20.33	4.05	5.16
Moisture Content	28.09	32.24	40.76	43.41	47.71	26.67	26.94
Liquid Limit	**43.39 %**		Average Plastic Limit :			**26.80 %**	

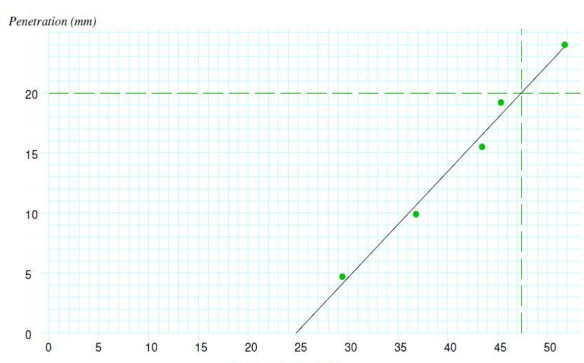


Test executed by: ____________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : LAPAI GWARI -C2		
Sample no. :	**Depth of sample :** 1.0m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	4.7	9.9	15.5	19.2	24.0		
Can Weight	37.82	36.63	36.63	37.24	37.10	37.16	35.61
Weight of Can + Wet Soil	47.89	49.30	48.68	53.33	64.99	42.82	40.98
Weight of Can + Dry Soil	45.60	45.89	45.03	48.31	55.48	41.56	39.80
Weight of Moisture	2.29	3.41	3.65	5.02	9.51	1.26	1.18
Weight of Dry Soil	7.78	9.26	8.40	11.07	18.38	4.40	4.19
Moisture Content	29.43	36.83	43.45	45.35	51.74	28.64	28.16
Liquid Limit	**47.40 %**		Average Plastic Limit :			**28.40 %**	



Test executed by: ____________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

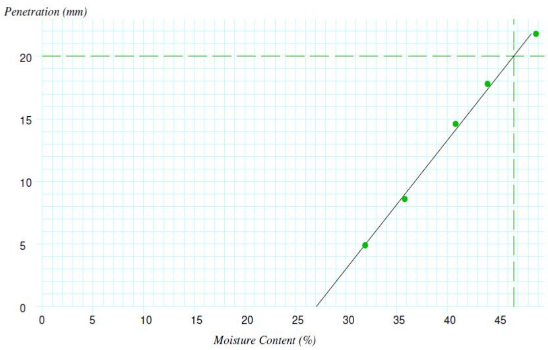
Project :

Test Location : LAPAI GWARI - C3

Sample no. : **Depth of sample :** 1.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	4.9	8.6	14.6	17.8	21.8		
Can Weight	36.46	36.78	36.53	34.09	37.41	33.65	33.84
Weight of Can + Wet Soil	49.36	48.69	53.05	50.85	62.97	39.27	39.18
Weight of Can + Dry Soil	46.24	45.55	48.26	45.73	54.59	37.96	37.94
Weight of Moisture	3.12	3.14	4.79	5.12	8.38	1.31	1.24
Weight of Dry Soil	9.78	8.77	11.73	11.64	17.18	4.31	4.10
Moisture Content	31.90	35.80	40.84	43.99	48.78	30.39	30.24
Liquid Limit	**46.56 %**			Average Plastic Limit :		**30.32 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

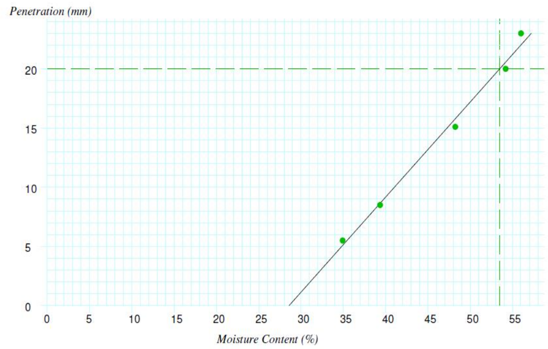
Project :

Test Location : PYATA-A3

Sample no. : **Depth of sample :** 1.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	5.5	8.5	15.1	20.0	23.0		
Can Weight	36.86	36.41	35.65	36.93	37.22		
Weight of Can + Wet Soil	54.90	55.43	59.28	57.10	59.03		
Weight of Can + Dry Soil	50.23	50.06	51.59	50.01	51.20		
Weight of Moisture	4.67	5.37	7.69	7.09	7.83		
Weight of Dry Soil	13.37	13.65	15.94	13.08	13.98		
Moisture Content	34.93	39.34	48.24	54.20	56.01	0.00	0.00
Liquid Limit	**53.49 %**			Average Plastic Limit :		**0.00 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

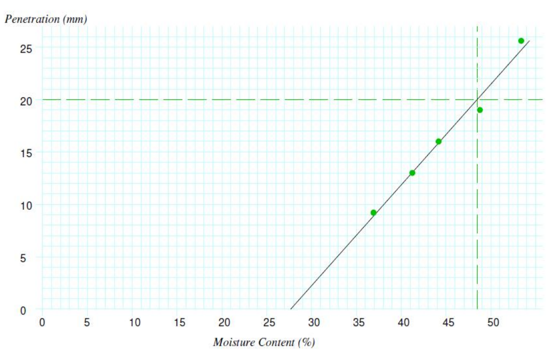
Project :

Test Location : PYATA - B1

Sample no. : **Depth of sample :** 0.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	9.2	13.0	16.0	19.0	25.6		
Can Weight	37.33	37.82	37.47	35.68	37.19	35.55	38.14
Weight of Can + Wet Soil	51.87	51.93	49.84	47.30	58.02	40.73	43.08
Weight of Can + Dry Soil	47.95	47.81	46.05	43.49	50.77	39.35	41.85
Weight of Moisture	3.92	4.12	3.79	3.81	7.25	1.38	1.23
Weight of Dry Soil	10.62	9.99	8.58	7.81	13.58	3.80	3.71
Moisture Content	36.91	41.24	44.17	48.78	53.39	36.32	33.15
Liquid Limit	**48.48 %**			Average Plastic Limit :		**34.73 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

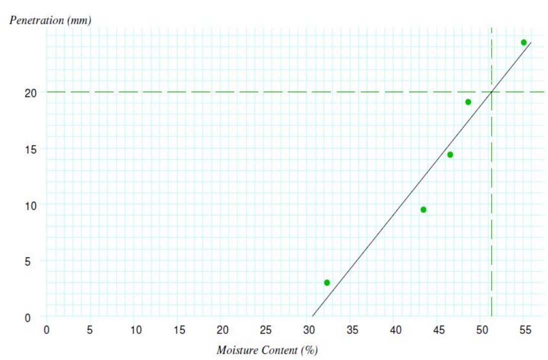
Project :

Test Location : PYATA - B2

Sample no. : **Depth of sample :** 1.0m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	3.0	9.5	14.4	19.1	24.4		
Can Weight	36.05	36.18	35.29	37.95	36.82	33.86	38.29
Weight of Can + Wet Soil	53.89	56.90	55.43	58.66	63.03	38.16	42.11
Weight of Can + Dry Soil	49.53	50.62	49.03	51.88	53.72	36.96	41.04
Weight of Moisture	4.36	6.28	6.40	6.78	9.31	1.20	1.07
Weight of Dry Soil	13.48	14.44	13.74	13.93	16.90	3.10	2.75
Moisture Content	32.34	43.49	46.58	48.67	55.09	38.71	38.91
Liquid Limit	**51.37 %**			Average Plastic Limit :		**38.81 %**	

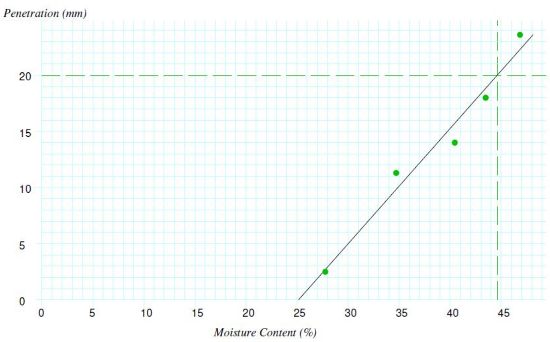


Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : PYATA - C1		
Sample no. :	**Depth of sample :** 0.5m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	2.5	11.3	14.0	18.0	23.6		
Can Weight	35.29	36.11	38.23	36.30	36.48	37.70	35.51
Weight of Can + Wet Soil	52.45	55.63	56.98	57.09	59.24	43.39	41.02
Weight of Can + Dry Soil	48.72	50.60	51.58	50.79	51.98	41.97	39.63
Weight of Moisture	3.73	5.03	5.40	6.30	7.26	1.42	1.39
Weight of Dry Soil	13.43	14.49	13.35	14.49	15.50	4.27	4.12
Moisture Content	27.77	34.71	40.45	43.48	46.84	33.26	33.74
Liquid Limit	**44.61 %**		Average Plastic Limit :			**33.50 %**	

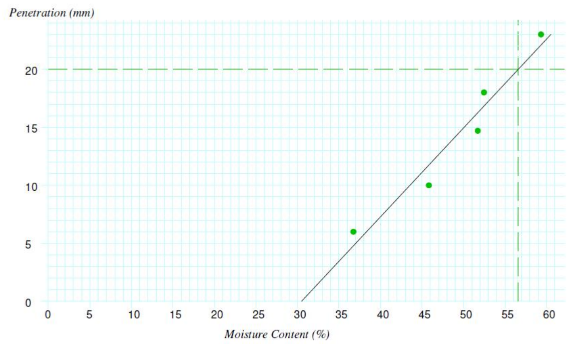


Test executed by: ________________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : PYATA - C2		
Sample no. :	**Depth of sample :** 1.0m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	6.0	10.0	14.7	18.0	23.0		
Can Weight	35.59	36.45	36.41	35.43	37.51	36.64	37.31
Weight of Can + Wet Soil	52.82	57.07	55.39	56.82	55.42	39.98	40.82
Weight of Can + Dry Soil	48.19	50.59	48.92	49.46	48.75	38.91	39.71
Weight of Moisture	4.63	6.48	6.47	7.36	6.67	1.07	1.11
Weight of Dry Soil	12.60	14.14	12.51	14.03	11.24	2.27	2.40
Moisture Content	36.75	45.83	51.72	52.46	59.34	47.14	46.25
Liquid Limit	**56.61 %**		Average Plastic Limit :			**46.69 %**	

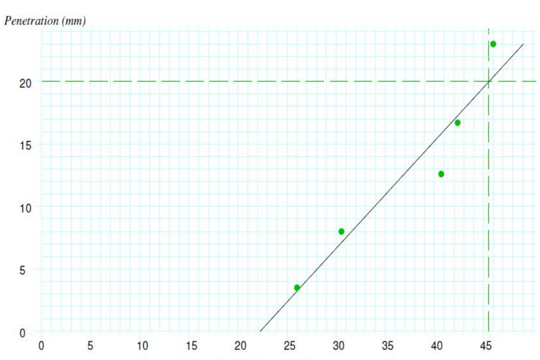


Test executed by: ________________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : PYATA - C3		
Sample no. :	**Depth of sample :** 1.5m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	3.5	8.0	12.6	16.7	23.0		
Can Weight	37.42	37.95	37.17	37.36	36.68	36.44	37.87
Weight of Can + Wet Soil	52.78	58.17	54.04	55.84	59.85	41.22	42.56
Weight of Can + Dry Soil	49.61	53.44	49.16	50.34	52.55	40.14	41.50
Weight of Moisture	3.17	4.73	4.88	5.50	7.30	1.08	1.06
Weight of Dry Soil	12.19	15.49	11.99	12.98	15.87	3.70	3.63
Moisture Content	26.00	30.54	40.70	42.37	46.00	29.19	29.20
Liquid Limit	**45.57 %**		Average Plastic Limit :			**29.20 %**	

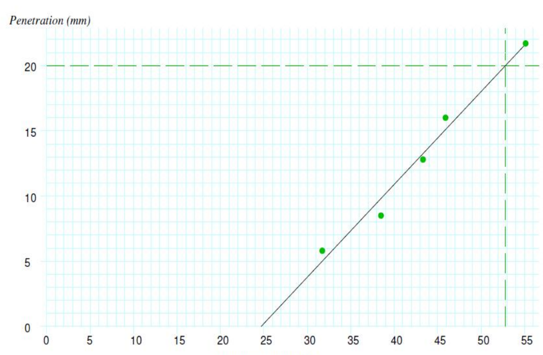


Test executed by: ________________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : MAIKUNKELE - A3		
Sample no. :	**Depth of sample :** 1.5m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	5.8	8.5	12.8	16.0	21.7		
Can Weight	36.52	35.67	36.12	35.70	38.31	38.20	36.44
Weight of Can + Wet Soil	44.99	46.86	47.50	48.57	59.44	43.85	41.12
Weight of Can + Dry Soil	42.95	43.75	44.06	44.52	51.93	42.40	39.96
Weight of Moisture	2.04	3.11	3.44	4.05	7.51	1.45	1.16
Weight of Dry Soil	6.43	8.08	7.94	8.82	13.62	4.20	3.52
Moisture Content	31.73	38.49	43.32	45.92	55.14	34.52	32.95
Liquid Limit	**52.82 %**		Average Plastic Limit :			**33.74 %**	



Test executed by: ________________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

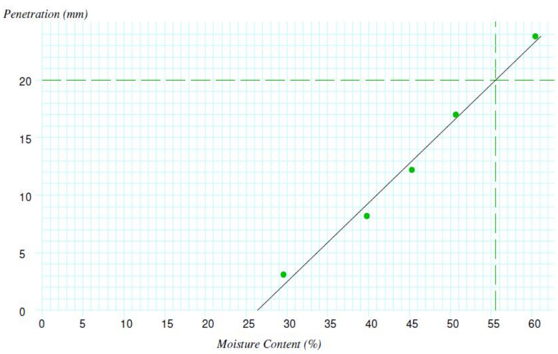
Project :

Test Location : MAIKUNKELE - B2

Sample no. : **Depth of sample :** 1.0m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	3.1	8.2	12.2	17.0	23.8		
Can Weight	34.15	37.83	37.48	37.52	33.64	37.57	36.55
Weight of Can + Wet Soil	44.89	52.66	52.24	59.08	62.24	41.53	41.41
Weight of Can + Dry Soil	42.44	48.44	47.64	51.83	51.47	40.37	39.97
Weight of Moisture	2.45	4.22	4.60	7.25	10.77	1.16	1.44
Weight of Dry Soil	8.29	10.61	10.16	14.31	17.83	2.80	3.42
Moisture Content	29.55	39.77	45.28	50.66	60.40	41.43	42.11
Liquid Limit	**55.55 %**		Average Plastic Limit :			**41.77 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

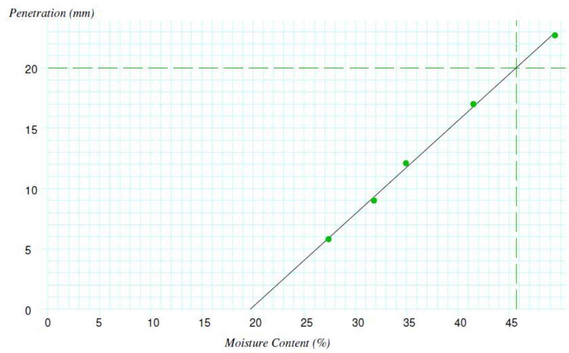
Project :

Test Location : MAIKUNKELE - B3

Sample no. : **Depth of sample :** 1.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	5.8	9.0	12.1	17.0	22.7		
Can Weight	37.16	33.04	36.51	37.36	37.82	36.48	36.78
Weight of Can + Wet Soil	45.07	41.70	44.97	50.45	59.18	39.61	41.13
Weight of Can + Dry Soil	43.37	39.61	42.78	46.61	52.11	38.73	39.96
Weight of Moisture	1.70	2.09	2.19	3.84	7.07	0.88	1.17
Weight of Dry Soil	6.21	6.57	6.27	9.25	14.29	2.25	3.18
Moisture Content	27.38	31.81	34.93	41.51	49.48	39.11	36.79
Liquid Limit	**45.69 %**		Average Plastic Limit :			**37.95 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

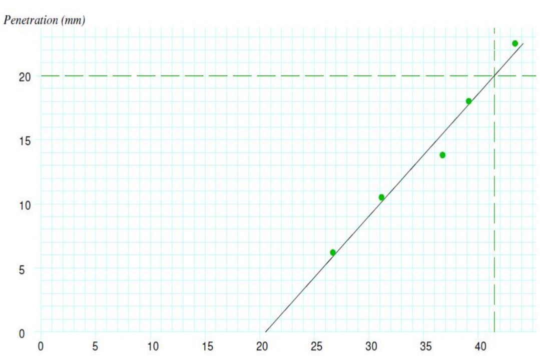
Project :

Test Location : MAIKUNKELE - B3

Sample no. : **Depth of sample :** 1.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	6.2	10.5	13.8	18.0	22.5		
Can Weight	34.37	34.04	37.82	37.60	37.80	36.70	36.70
Weight of Can + Wet Soil	45.52	48.73	53.82	55.55	60.24	40.41	40.42
Weight of Can + Dry Soil	43.17	45.24	49.52	50.50	53.45	39.66	39.68
Weight of Moisture	2.35	3.49	4.30	5.05	6.79	0.75	0.74
Weight of Dry Soil	8.80	11.20	11.70	12.90	15.65	2.96	2.98
Moisture Content	26.70	31.16	36.75	39.15	43.39	25.34	24.83
Liquid Limit	**41.51 %**		Average Plastic Limit :			**25.09 %**	



Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

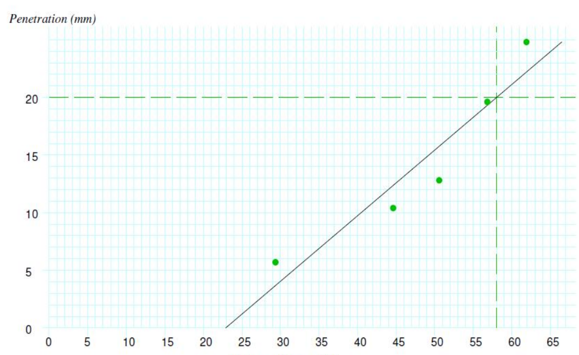
Project :

Test Location : MAIKUNKELE - C1

Sample no. : **Depth of sample :** 0.5m **Date :**

Sample Description :

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	5.7	10.4	12.8	19.6	24.8		
Can Weight	35.39	36.93	34.15	37.36	37.22	37.53	35.91
Weight of Can + Wet Soil	46.98	48.34	48.32	56.18	58.98	42.05	41.89
Weight of Can + Dry Soil	44.35	44.82	43.56	49.36	50.66	40.78	40.19
Weight of Moisture	2.63	3.52	4.76	6.82	8.32	1.27	1.70
Weight of Dry Soil	8.96	7.89	9.41	12.00	13.44	3.25	4.28
Moisture Content	29.35	44.61	50.58	56.83	61.90	39.08	39.72
Liquid Limit	**58.04 %**		Average Plastic Limit :			**39.40 %**	

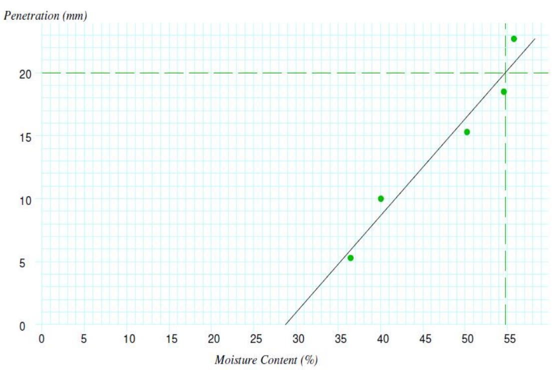


Test executed by:

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : PYATA - C1		
Sample no. :	**Depth of sample :** 0.5m	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	5.3	10.0	15.3	18.5	22.7		
Can Weight	37.44	36.77	35.60	36.28	35.32	34.46	33.91
Weight of Can + Wet Soil	52.56	53.66	56.03	58.80	58.00	40.00	40.38
Weight of Can + Dry Soil	48.52	48.83	49.20	50.85	49.88	38.50	38.56
Weight of Moisture	4.04	4.83	6.83	7.95	8.12	1.50	1.82
Weight of Dry Soil	11.08	12.06	13.60	14.57	14.56	4.04	4.65
Moisture Content	36.46	40.05	50.22	54.56	55.77	37.13	39.14
Liquid Limit	**54.74 %**		Average Plastic Limit :			**38.13 %**	

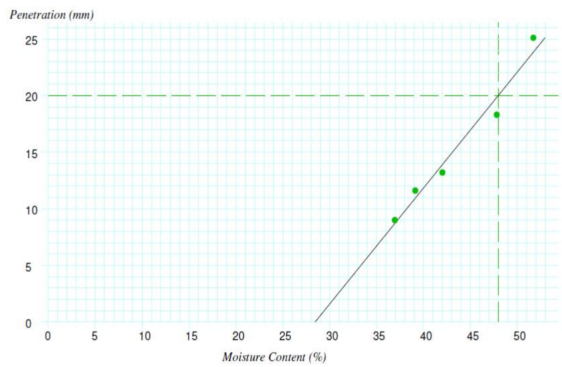


Test executed by: ____________________

LIQUID LIMIT DETERMINATION (CONE PENETROMETER METHOD)

Project :		
Test Location : MAIKUNKELE 2		
Sample no. : A	**Depth of sample :**	**Date :**
Sample Description :		

	LIQUID LIMIT					PLASTIC LIMIT	
Can Number	1	2	3	4	5	1	2
Penetration	9.0	11.6	13.2	18.3	25.1		
Can Weight	23.2	24.5	23.1	22.7	23.0	23.0	23.1
Weight of Can + Wet Soil	29.5	30.9	32.9	29.5	31.8	29.9	27.4
Weight of Can + Dry Soil	27.8	29.1	30.0	27.3	28.8	28.2	26.4
Weight of Moisture	1.70	1.80	2.90	2.20	3.00	1.70	1.00
Weight of Dry Soil	4.60	4.60	6.90	4.60	5.80	5.20	3.30
Moisture Content	36.96	39.13	42.03	47.83	51.72	32.69	30.30
Liquid Limit	**47.99 %**		Average Plastic Limit :			**31.50 %**	



Test executed by: ____________________

APPENDIX C

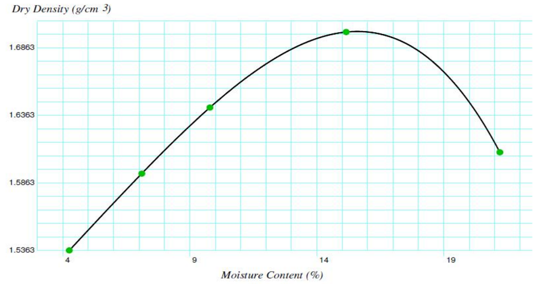
Compaction charts

COMPACTION TEST

Project :
Test Location : AB2-1.0m
Sample no. : **Volume of Mold :** 944 cm3 **Date :**
Sample Description :

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3400		3499		3590		3734		3729	
Weight of Wet Soil (g)	1,512.00		1,611.00		1,702.00		1,846.00		1,841.00	
Wet Density (g/cm3)	1.60		1.71		1.80		1.96		1.95	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	36.36	36.74	36.02	37.41	37.34	36.59	36.87	32.86	36.59	36.36
Weight of Can + Wet Soil (g)	82.67	65.48	77.50	81.43	89.58	88.06	81.10	93.31	92.86	89.83
Weight of Can + Dry Soil (g)	80.79	64.30	74.66	78.60	84.84	83.55	75.37	85.23	83.03	80.46
Weight of Water (g)	1.88	1.18	2.84	2.83	4.74	4.51	5.73	8.08	9.83	9.37
Weight of Dry Soil (g)	44.43	27.56	38.64	41.19	47.50	46.96	38.50	52.37	46.44	44.10
Moisture Content (g)	4.23	4.28	7.35	6.87	9.98	9.60	14.88	15.43	21.17	21.25
Ave. Moisture Content (g)	4.26		7.11		9.79		15.16		21.21	
Dry Density (g/cm3)	1.5363		1.5933		1.6422		1.6981		1.6090	

Dry Density (g/cm 3)



MDD : 1.6986 g/cm3
OMC : 15.60 %

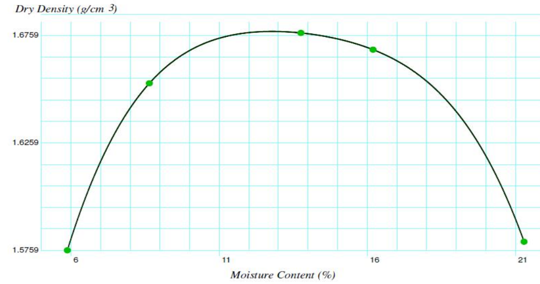
Test executed by:

COMPACTION TEST

Project :
Test Location : PYATA B3- 1.5m
Sample no. : **Volume of Mold :** 944 cm3 **Date :**
Sample Description :

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3463		3584		3689		3719		3697	
Weight of Wet Soil (g)	1,575.00		1,696.00		1,801.00		1,831.00		1,809.00	
Wet Density (g/cm3)	1.67		1.80		1.91		1.94		1.92	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	36.46	36.20	37.52	37.21	37.37	36.79	37.46	35.58	35.51	33.84
Weight of Can + Wet Soil (g)	89.48	95.51	88.00	88.76	93.33	93.35	96.33	94.75	93.33	91.78
Weight of Can + Dry Soil (g)	86.36	92.42	84.04	84.60	86.56	86.51	88.33	86.30	83.47	81.32
Weight of Water (g)	3.12	3.09	3.96	4.16	6.77	6.84	8.00	8.45	9.86	10.46
Weight of Dry Soil (g)	49.90	56.22	46.52	47.39	49.19	49.72	50.87	50.72	47.96	47.48
Moisture Content (g)	6.25	5.50	8.51	8.78	13.76	13.76	15.73	16.66	20.56	22.03
Ave. Moisture Content (g)	5.87		8.65		13.76		16.19		21.29	
Dry Density (g/cm3)	1.5759		1.6536		1.6771		1.6693		1.5799	

Dry Density (g/cm 3)



MDD : 1.6778 g/cm3
OMC : 12.79 %

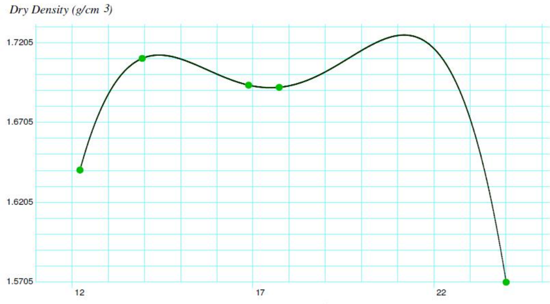
Test executed by:

COMPACTION TEST

Project :
Test Location : LANPAI GWARI - B3- 1.5m
Sample no. : **Volume of Mold :** 944 cm3 **Date :**
Sample Description :

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3626		3728		3757		3769		3726	
Weight of Wet Soil (g)	1,738.00		1,840.00		1,869.00		1,881.00		1,838.00	
Wet Density (g/cm3)	1.84		1.95		1.98		1.99		1.95	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	36.24	38.31	37.58	36.61	35.65	35.73	37.81	38.15	38.28	38.14
Weight of Can + Wet Soil (g)	80.27	71.88	91.81	78.64	91.67	90.29	98.69	91.52	99.09	99.37
Weight of Can + Dry Soil (g)	75.11	68.51	85.02	73.62	83.59	82.40	89.37	83.62	87.46	87.38
Weight of Water (g)	5.16	3.37	6.79	5.02	8.08	7.89	9.32	7.90	11.63	11.99
Weight of Dry Soil (g)	38.87	30.20	47.44	37.01	47.94	46.67	51.56	45.47	49.18	49.24
Moisture Content (g)	13.28	11.16	14.31	13.56	16.85	16.91	18.08	17.37	23.65	24.35
Ave. Moisture Content (g)	12.22		13.94		16.88		17.73		24.00	
Dry Density (g/cm3)	1.6407		1.7107		1.6939		1.6926		1.5702	

Dry Density (g/cm 3)



MDD : 1.7254 g/cm3
OMC : 21.18 %

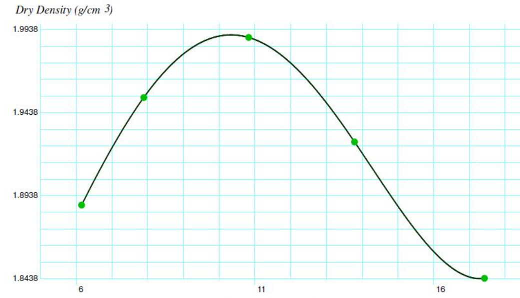
Test executed by:

COMPACTION TEST

Project :
Test Location : LAPAI GWARI- C1- 0.5m
Sample no. : **Volume of Mold :** 944 cm3 **Date :**
Sample Description :

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3780		3877		3969		3957		3932	
Weight of Wet Soil (g)	1,892.00		1,989.00		2,081.00		2,069.00		2,044.00	
Wet Density (g/cm3)	2.00		2.11		2.20		2.19		2.17	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	37.49	38.31	36.65	37.97	35.64	37.35	37.53	36.32	35.91	36.80
Weight of Can + Wet Soil (g)	84.73	88.05	70.96	80.51	88.46	95.74	86.26	95.52	91.42	98.01
Weight of Can + Dry Soil (g)	82.05	85.11	68.50	77.34	83.02	90.35	80.52	88.15	82.94	89.20
Weight of Water (g)	2.68	2.94	2.46	3.17	5.44	5.39	5.74	7.37	8.48	8.81
Weight of Dry Soil (g)	44.56	46.80	31.85	39.37	47.38	53.00	42.99	51.83	47.03	52.40
Moisture Content (g)	6.01	6.28	7.72	8.05	11.48	10.17	13.35	14.22	18.03	16.81
Ave. Moisture Content (g)	6.15		7.89		10.83		13.79		17.42	
Dry Density (g/cm3)	1.8882		1.9529		1.9891		1.9262		1.8440	

Dry Density (g/cm 3)



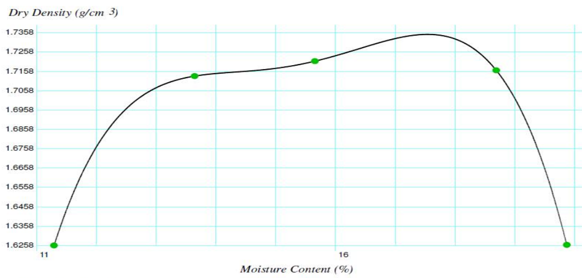
MDD : 1.9907 g/cm3
OMC : 10.33 %

Test executed by:

COMPACTION TEST

Project :	
Test Location :	MAIKUNKELE - C2 - 1.0m
Sample no. :	**Volume of Mold :** 944 cm^3 **Date :**
Sample Description :	

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3596		3726		3767		3811		3728	
Weight of Wet Soil (g)	1,708.00		1,838.00		1,879.00		1,923.00		1,840.00	
Wet Density (g/cm3)	1.81		1.95		1.99		2.04		1.95	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	36.21	38.33	37.59	36.62	36.65	35.73	37.82	38.17	38.30	38.16
Weight of Can + Wet Soil (g)	82.14	71.89	92.83	79.65	92.68	90.28	98.68	92.53	98.11	96.31
Weight of Can + Dry Soil (g)	76.18	69.50	86.04	74.61	84.58	83.41	88.38	84.63	88.50	86.38
Weight of Water (g)	5.96	2.39	6.79	5.04	8.10	6.87	10.30	7.90	9.61	9.93
Weight of Dry Soil (g)	39.97	31.17	48.45	37.99	47.93	47.68	50.56	46.46	50.20	48.22
Moisture Content (g)	14.91	7.67	14.01	13.27	16.90	14.41	20.37	17.00	19.14	20.59
Ave. Moisture Content (g)	11.29		13.64		15.65		18.69		19.87	
Dry Density (g/cm3)	1.6258		1.7133		1.7211		1.7163		1.6261	



MDD : 1.7349 g/cm3

OMC : 17.54 %

Test executed by: ____________________

COMPACTION TEST

Project :	
Test Location :	PYATA - C1 - 0.5m
Sample no. :	**Volume of Mold :** 944 cm^3 **Date :**
Sample Description :	

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3778		3880		3965		3956		3942	
Weight of Wet Soil (g)	1,890.00		1,992.00		2,077.00		2,068.00		2,054.00	
Wet Density (g/cm3)	2.00		2.11		2.20		2.19		2.18	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	37.31	38.34	37.14	37.11	37.09	37.51	38.04	35.58	37.18	37.11
Weight of Can + Wet Soil (g)	90.11	85.44	89.05	94.15	87.72	86.15	89.07	90.03	91.19	87.71
Weight of Can + Dry Soil (g)	86.01	81.26	83.57	88.24	81.51	80.51	82.30	82.98	83.70	80.41
Weight of Water (g)	4.10	4.18	5.48	5.91	6.21	5.64	6.77	7.05	7.49	7.30
Weight of Dry Soil (g)	48.70	42.92	46.43	51.13	44.42	43.00	44.26	47.40	46.52	43.30
Moisture Content (g)	8.42	9.74	11.80	11.56	13.98	13.12	15.30	14.87	16.10	16.86
Ave. Moisture Content (g)	9.08		11.68		13.55		15.08		16.48	
Dry Density (g/cm3)	1.8355		1.8895		1.9377		1.9035		1.8680	

MDD : 1.9378 g/cm3

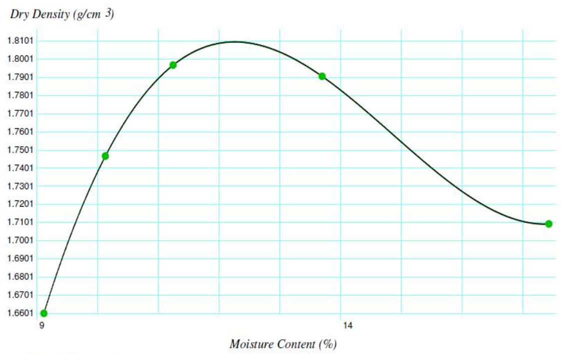
OMC : 13.47 %

Test executed by: ____________________

COMPACTION TEST

Project :		
Test Location : MAIKUNKELE - B3 - 1.5m		
Sample no. :	**Volume of Mold :** 944 cm3	**Date :**
Sample Description :		

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3598		3704		3775		3810		3783	
Weight of Wet Soil (g)	1,710.00		1,816.00		1,887.00		1,922.00		1,895.00	
Wet Density (g/cm3)	1.81		1.92		2.00		2.04		2.01	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	37.19	37.77	37.48	37.03	37.76	36.36	37.72	38.12	33.78	36.56
Weight of Can + Wet Soil (g)	96.93	83.99	89.78	84.36	95.94	81.76	96.95	86.63	81.64	85.22
Weight of Can + Dry Soil (g)	91.99	80.09	85.20	79.80	90.15	77.10	89.59	80.97	74.66	77.87
Weight of Water (g)	4.94	3.90	4.58	4.56	5.79	4.66	7.36	5.66	6.98	7.35
Weight of Dry Soil (g)	54.80	42.32	47.72	42.77	52.39	40.74	51.87	42.85	40.88	41.31
Moisture Content (g)	9.01	9.22	9.60	10.66	11.05	11.44	14.19	13.21	17.07	17.79
Ave. Moisture Content (g)	9.12		10.13		11.25		13.70		17.43	
Dry Density (g/cm3)	1.6601		1.7468		1.7969		1.7907		1.7094	



MDD : 1.8097 g/cm3

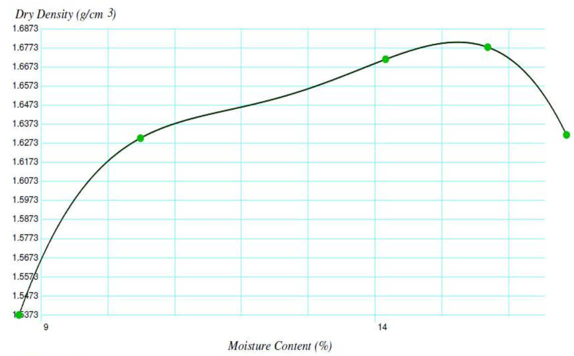
OMC : 12.26 %

Test executed by: ____________________

COMPACTION TEST

Project :		
Test Location : MAIKUNKELE - B1 - 1.0m		
Sample no. :	**Volume of Mold :** 944 cm3	**Date :**
Sample Description :		

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3465		3588		3689		3720		3688	
Weight of Wet Soil (g)	1,577.00		1,700.00		1,801.00		1,832.00		1,800.00	
Wet Density (g/cm3)	1.67		1.80		1.91		1.94		1.91	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	37.46	36.19	37.55	37.21	37.37	36.78	37.50	36.58	37.51	35.84
Weight of Can + Wet Soil (g)	89.56	95.61	88.01	90.51	93.33	95.35	95.23	93.78	90.33	91.79
Weight of Can + Dry Soil (g)	85.96	90.25	83.10	85.58	86.56	87.91	87.53	85.90	82.80	83.62
Weight of Water (g)	3.60	5.36	4.91	4.93	6.77	7.44	7.70	7.88	7.53	8.17
Weight of Dry Soil (g)	48.50	54.06	45.55	48.37	49.19	51.13	50.03	49.32	45.29	47.78
Moisture Content (g)	7.42	9.91	10.78	10.19	13.76	14.55	15.39	15.98	16.63	17.10
Ave. Moisture Content (g)	8.67		10.49		14.16		15.68		16.86	
Dry Density (g/cm3)	1.5373		1.6299		1.6712		1.6776		1.6316	



MDD : 1.6802 g/cm3

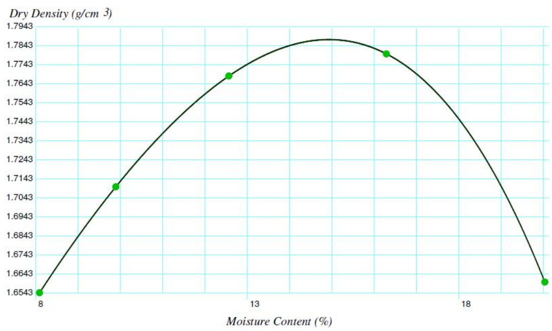
OMC : 15.23 %

Test executed by: ____________________

COMPACTION TEST

Project :	
Test Location :	PYATA - A3 - 1.5m
Sample no. :	**Volume of Mold :** 944 cm3 **Date :**
Sample Description :	

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3576		3662		3767		3842		3769	
Weight of Wet Soil (g)	1,688.00		1,774.00		1,879.00		1,954.00		1,881.00	
Wet Density (g/cm3)	1.79		1.88		1.99		2.07		1.99	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	36.60	35.65	36.47	35.52	35.59	36.37	35.82	35.22	36.17	36.12
Weight of Can + Wet Soil (g)	87.82	97.45	93.75	89.01	79.48	84.11	92.11	90.57	85.83	89.84
Weight of Can + Dry Soil (g)	84.04	92.76	88.64	84.15	74.59	78.77	84.20	82.84	77.70	80.70
Weight of Water (g)	3.78	4.69	5.11	4.86	4.89	5.34	7.91	7.73	8.13	9.14
Weight of Dry Soil (g)	47.44	57.11	52.17	48.63	39.00	42.40	48.38	47.62	41.53	44.58
Moisture Content (g)	7.97	8.21	9.79	9.99	12.54	12.59	16.35	16.23	19.58	20.50
Ave. Moisture Content (g)	8.09		9.89		12.57		16.29		20.04	
Dry Density (g/cm3)	1.6543		1.7100		1.7683		1.7799		1.6599	



MDD : 1.7874 g/cm3

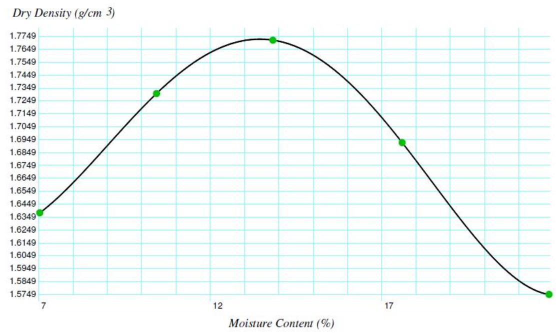
OMC : 14.94 %

Test executed by:

COMPACTION TEST

Project :	
Test Location :	PYATA - C3 - 1.5m
Sample no. :	**Volume of Mold :** 944 cm3 **Date :**
Sample Description :	

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3543		3692		3792		3767		3700	
Weight of Wet Soil (g)	1,655.00		1,804.00		1,904.00		1,879.00		1,812.00	
Wet Density (g/cm3)	1.75		1.91		2.02		1.99		1.92	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	35.66	34.49	36.49	34.13	36.42	33.97	33.82	35.70	36.20	36.21
Weight of Can + Wet Soil (g)	75.71	76.71	84.21	70.31	95.57	87.28	97.49	94.79	77.57	77.11
Weight of Can + Dry Soil (g)	73.06	73.96	79.68	66.91	88.39	80.80	87.93	85.98	70.18	69.73
Weight of Water (g)	2.65	2.75	4.53	3.40	7.18	6.48	9.56	8.81	7.39	7.38
Weight of Dry Soil (g)	37.40	39.47	43.19	32.78	51.97	46.83	54.11	50.28	33.98	33.52
Moisture Content (g)	7.09	6.97	10.49	10.37	13.82	13.84	17.67	17.52	21.75	22.02
Ave. Moisture Content (g)	7.03		10.43		13.83		17.59		21.88	
Dry Density (g/cm3)	1.6381		1.7305		1.7720		1.6926		1.5749	



MDD : 1.7727 g/cm3

OMC : 13.45 %

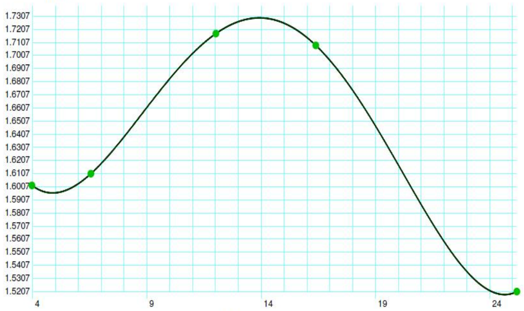
Test executed by:

COMPACTION TEST

Project :	
Test Location :	LAPAIN GWARI - B3 - 1.5m
Sample no. :	**Volume of Mold :** 944 cm^3 **Date :**
Sample Description :	

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3460		3508		3704		3765		3685	
Weight of Wet Soil (g)	1,572.00		1,620.00		1,816.00		1,877.00		1,797.00	
Wet Density (g/cm3)	1.67		1.72		1.92		1.99		1.90	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	37.49	37.36	37.64	37.91	36.79	37.25	37.69	38.02	36.68	35.60
Weight of Can + Wet Soil (g)	80.55	91.66	89.29	82.01	99.15	98.33	88.46	97.68	95.93	99.83
Weight of Can + Dry Soil (g)	78.88	89.62	86.22	79.21	92.51	91.73	81.56	88.99	83.90	87.03
Weight of Water (g)	1.67	2.04	3.07	2.80	6.64	6.60	6.90	8.69	12.03	12.80
Weight of Dry Soil (g)	41.39	52.26	48.58	41.30	55.72	54.48	43.87	50.97	47.22	51.43
Moisture Content (g)	4.03	3.90	6.32	6.78	11.92	12.11	15.73	17.05	25.48	24.89
Ave. Moisture Content (g)	3.97		6.55		12.02		16.39		25.18	
Dry Density (g/cm3)	1.6017		1.6106		1.7174		1.7084		1.5207	

Dry Density (g/cm 3)



Moisture Content (%)

MDD : 1.7293 g/cm3

OMC : 13.92 %

Test executed by:

COMPACTION TEST

Project :	
Test Location :	LAPAIN GWARI - B2 - 1.0m
Sample no. :	**Volume of Mold :** 944 cm^3 **Date :**
Sample Description :	

Weight of Mold (g)	1888		1888		1888		1888		1888	
Weight of Mold+Wet Soil (g)	3639		3734		3848		3846		3791	
Weight of Wet Soil (g)	1,751.00		1,846.00		1,960.00		1,958.00		1,903.00	
Wet Density (g/cm3)	1.85		1.96		2.08		2.07		2.02	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	36.16	35.56	37.48	37.34	36.75	35.31	37.86	37.66	36.05	37.81
Weight of Can + Wet Soil (g)	79.02	91.16	96.92	81.00	96.38	96.17	96.83	96.54	81.48	81.13
Weight of Can + Dry Soil (g)	77.59	89.30	93.51	78.36	90.74	90.27	89.69	89.54	73.81	73.75
Weight of Water (g)	1.43	1.86	3.41	2.64	5.64	5.90	7.14	7.00	7.67	7.38
Weight of Dry Soil (g)	41.43	53.74	56.03	41.02	53.99	54.96	51.83	51.88	37.76	35.94
Moisture Content (g)	3.45	3.46	6.09	6.44	10.45	10.74	13.78	13.49	20.31	20.53
Ave. Moisture Content (g)	3.46		6.26		10.59		13.63		20.42	
Dry Density (g/cm3)	1.7929		1.8403		1.8774		1.8253		1.6740	

Dry Density (g/cm 3)

Moisture Content (%)

MDD : 1.8797 g/cm3

OMC : 9.82 %

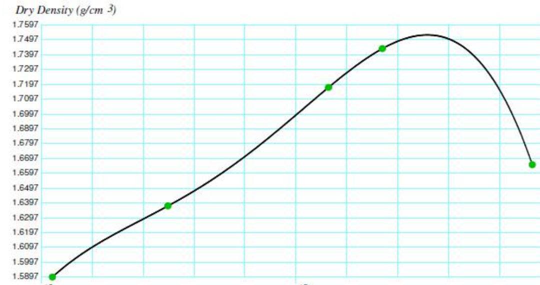
Test executed by:

COMPACTION TEST

Project :	
Test Location :	MAIKUNKELE - A1 - 0.5m
Sample no. :	**Volume of Mold :** 944 cm^3 **Date :**
Sample Description :	

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3542		3627		3763		3809		3769	
Weight of Wet Soil (g)	1,654.00		1,739.00		1,875.00		1,921.00		1,881.00	
Wet Density (g/cm3)	1.75		1.84		1.99		2.03		1.99	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	36.93	37.31	37.60	37.75	38.01	37.10	37.48	38.26	35.34	37.15
Weight of Can + Wet Soil (g)	94.10	97.04	93.71	94.07	95.46	94.63	94.16	96.88	90.08	90.80
Weight of Can + Dry Soil (g)	89.54	90.75	87.38	87.92	87.78	86.76	85.87	88.68	81.24	81.85
Weight of Water (g)	4.56	6.29	6.33	6.15	7.68	7.87	8.29	8.20	8.84	8.95
Weight of Dry Soil (g)	52.61	53.44	49.78	50.17	49.77	49.66	48.39	50.42	45.90	44.70
Moisture Content (g)	8.67	11.77	12.72	12.26	15.43	15.85	17.13	16.26	19.26	20.02
Ave. Moisture Content (g)	10.22		12.49		15.64		16.70		19.64	
Dry Density (g/cm3)	1.5897		1.6377		1.7176		1.7438		1.6655	

Dry Density (g/cm 3)



Moisture Content (%)

MDD : 1.7530 g/cm3

OMC : 17.58 %

Test executed by:

COMPACTION TEST

Project :	
Test Location : LAPAIN GWARI - A2 - 1.0m	
Sample no. :	**Volume of Mold :** 944 cm^3 **Date :**
Sample Description :	

Weight of Mold (g)	**1888**		**1888**		**1888**		**1888**		**1888**	
Weight of Mold+Wet Soil (g)	3612		3774		3868		3874		3800	
Weight of Wet Soil (g)	1,724.00		1,886.00		1,980.00		1,986.00		1,912.00	
Wet Density (g/cm3)	1.83		2.00		2.10		2.10		2.03	
Can Number	1	2	3	4	5	6	7	8	9	10
Weight of Can (g)	37.78	38.34	36.24	37.63	36.44	36.54	35.61	37.39	37.34	35.33
Weight of Can + Wet Soil (g)	77.40	76.05	86.37	85.53	87.10	87.80	99.49	97.51	90.19	81.21
Weight of Can + Dry Soil (g)	75.31	73.92	82.79	81.68	81.67	82.39	91.30	90.55	81.24	74.02
Weight of Water (g)	2.09	2.13	3.58	3.85	5.43	5.41	8.19	6.96	8.95	7.19
Weight of Dry Soil (g)	37.53	35.58	46.55	44.05	45.23	45.85	55.69	53.16	43.90	38.69
Moisture Content (g)	5.57	5.99	7.69	8.74	12.01	11.80	14.71	13.09	20.39	18.58
Ave. Moisture Content (g)	5.78		8.22		11.90		13.90		19.49	
Dry Density (g/cm3)	1.7265		1.8462		1.8744		1.8471		1.6951	

MDD : 1.8793 g/cm3

OMC : 10.77 %

Test executed by: