EFFECT OF OIL PALM FROND ASH ON THE GEOTECHNICAL PROPERTIES OF A LIME STABILIZED LATERITIC SOIL

By

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ABSTRACT

Agro-based and bio-wastes generally referred to as solid wastes are good stabilization materials. Different solid wastes have been used by various researchers combining two or more at different times to treat soil and improve its engineering properties since the long term performance of any construction project depends on the soundness of the underlying soils. This paper has discussed the effect of the application of lime at varying percentages of 2%, 4%, 6% and 8%, oil palm frond ash also at varying percentages of 2%, 4%, 6% and 8%, then lime and ash at varying percentages of 2%, 4%, 6% and 8% to lateritic soil gotten from three pits along Ikole - Omuo road. Preliminary tests such as natural moisture content, specific gravity, and sieve analysis revealed the soils to be in the A-7 group of AASHTO classification which is fair to poor for construction hence, the need for stabilization. Atterberg limit tests, standard proctor compaction test, unconfined compression test and California bearing ratio tests were conducted on the soil samples before the application of additives and after their application. The results were compared, conclusions drawn and relevant recommendations made.

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DEDICATION

I dedicate this research to God Almighty for only Him could have made it possible.

CERTIFICATION

This is to certify that this Project was written and compiled by **ORIOLA MARY OLUBUNMI** with matriculation number **CVE/13/1067** under my supervision and is approved for its contribution to knowledge and literary presentation. All sources of information are specifically acknowledged by means of references, in partial fulfillment of the requirements for the award of Bachelor of Engineering (B.Eng.) degree in Civil Engineering, Federal University Oye Ekiti.

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~~Engr.~~ Tochukwu Onuorah

(Supervisor)

Date: 26/03/2019

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Dr. (Mrs.) O. Ndububa

(Head of Department)

Date: 26/03/2019

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LIST OF ABBREVIATIONS

CBR - California Bearing Ratio

SDAL - Saw Dust Ash-Lime

UCS - Unconfined Compressive Strength

CCA - Corn Cob Ash

MDD - Maximum Dry Density

OMC - Optimum Moisture Content

RAP - Reclaimed Asphalt Pavement

PKSA - Palm Kernel Shell Ash

BLA - Bamboo Leaf Ash

IWS - Industrial Waste Sand

AAFA - Alkali Activated Fly Ash

ESR - Expansive Soil-Rubber

OPFA - Oil Palm Frond Ash

AASHTO - American Association of State Highway and Transportation Officials

ASTM - American Society for Testing and Materials

OPC - Ordinary Portland Cement

CHAPTER ONE

INTRODUCTION

1.1 General Background

All soils originate directly or indirectly from rocks and these are classified according to their mode of formation. By a combination of physical and chemical processes rock masses are reduced to particles ranging in size down to 0.001 mm. Soils result when collections of these particles are re-deposited and are compressed and consolidated by further re-depositions above. The nature of the subsequent soil depends not only on its parent rocks, but also on the processes and conditions of disintegration, transport and deposition – and on time. Soils consist of grains (mineral grains, rock fragments, etc.) with water and air in the voids between grains. The water and air contents are readily changed by changes in conditions and location: soils can be perfectly dry (have no water content) or be fully saturated (have no air content) or be partly saturated (with both air and water present). Although the size and shape of the solid (granular) content rarely changes at a given point, they can vary considerably from point to point.

The term "soil" means different things to different people: To a geologist, it represents the products of past surface processes. To a pedologist, it represents currently occurring physical and chemical processes. To an engineer, it is a material that can be built on (e.g. foundations to buildings, bridges), built in (e.g. tunnels, culverts, basements e.t.c.), built with (e.g. roads, runways, embankments, dams, etc.), and supported (e.g. retaining walls, quays, etc.).

For any land-based structure, the foundation is very important and has to be strong to support the entire structure. In order for the foundation to be strong, the soil around it plays a very critical role. So, to work with soils, we need to have proper knowledge about their properties and factors which affect their behavior. The process of soil stabilization helps to achieve the required properties in a soil needed for the construction work (Kumar *et* al., 2016).

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From the beginning of construction work, the necessity of enhancing soil properties has come to the light. Ancient civilizations of the Chinese, Romans and Incas utilized various methods to improve soil strength etc., some of these methods were so effective that their buildings and roads still exist (Kumar *et* al., 2016).

The functionality and sustainability of a transportation infrastructure project depends mainly on the characteristics of the underlying soils. Often times, it has always been discovered that the residual soil does not always meet the engineering properties of the proposed design. Thus, a soil improvement method was introduced termed soil stabilization (Ankit *et* al., 2013).

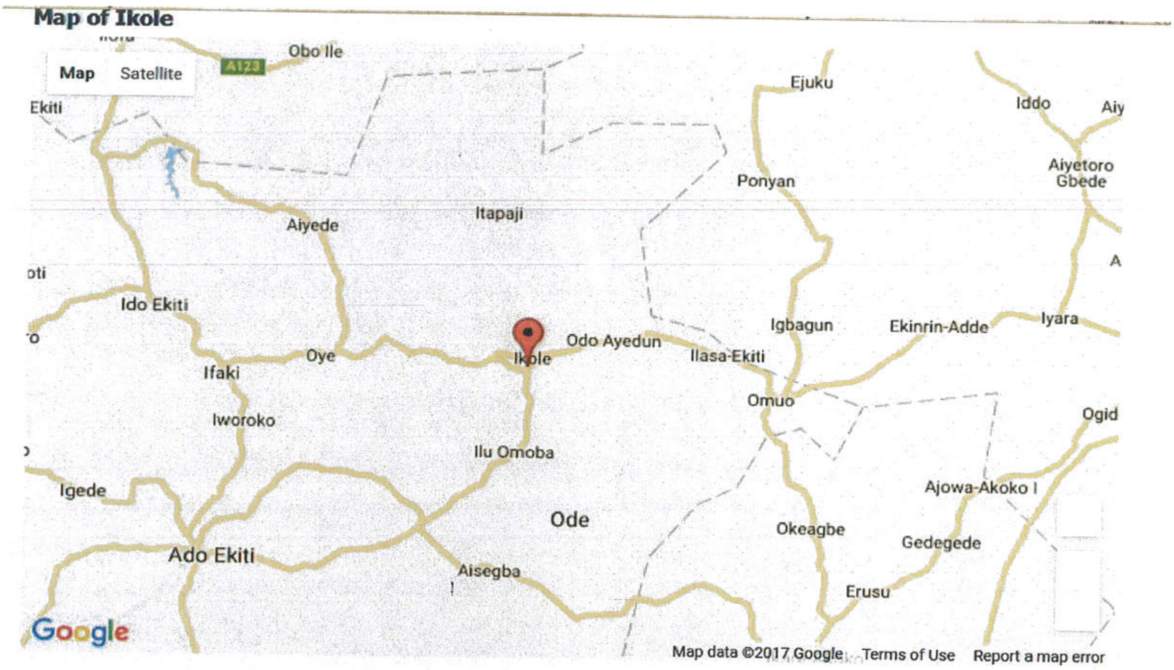
Soil stabilization is the process of improving the engineering properties of the soil, and thus making it more stable. It is used to increase the permeability and compressibility of the soil mass in the earth structure, in addition to an increase in its shear strength. Soil stabilization also increases the bearing capacity of foundation soils. Soil stabilization can be achieved through various methods such as mechanical stabilization (addition or removal of soil), cement stabilization (mixing pulverized soil and cement with water), lime stabilization (addition of lime), chemical stabilization (addition of chemicals), bituminuous stabilization (addition of bitumen as emulsions), thermal stabilization (heating or cooling the soil), electrical stabilization, stabilization by grout, stabilization by geotextiles and fabrics and reinforced earth (Arora, 2005).

In the construction of highways, pavement design plays an important role and each layer of the pavement must possess some structural properties to withstand the anticipated load upon it without deformation. As the property of each soil layer is increased, its functionality is also increased. For example, a pavement constructed over a high quality, stiff sub-grade may not need the additional features offered by a sub-base course.

The success of every project can be determined with a proper feasibility study and a well-considered cost-benefit analysis. Since highway project usually require huge funds for its execution, a lot of engineer tend to compromise standards and reduce the specified quality in order to meet a specific budget and time frame. It has also been discovered that the cost of improving the soil to suit the desired engineering properties of the construction is a good phase to look into to challenge the incompetency. In recent times, the challenge of construction and maintenance cost has also set in and researchers have focused on solid wastes likes of groundnut husk ash, oil palm frond ash, rice husk ash, banana leaves ash, egg shell ash etc. because of its availability, easy accessibility and pozzolanic (cementitious) characteristics. Also, in some areas, the availability of some of these materials has been a challenge and the need to seek for other materials to create a wide range of substitutes becomes absolutely necessary.

1.2 Description of Study Area

Geographically, Ikole Local Government is entirely within the tropics. It is located between longitude 5°30'52.17" East of Greenwich and latitude 7°47'53.76" North of the Equator (distancetos.com). Its neighbours are Kwara State to the North, Kogi State to the North east, Ekiti East to the East, Gboyin Local Government in the South and Oye Local government in the West as shown in fig 1.1. The headquarters of the local government, Ikole Ekiti which shall be considered in this research work is about 48.7 kilometres from Ado – Ekiti,the Ekiti State capital and 27.2 km from Omuo (a town located at the end of the Ikole – Omuo road stretch with longitude 5°43'28.98" East of Greenwich and latitude 7°45'46.85" North of the Equator, sourced from distancetos.com). The local government is mainly on the upland zone rising to about 250 metres above the sea level.



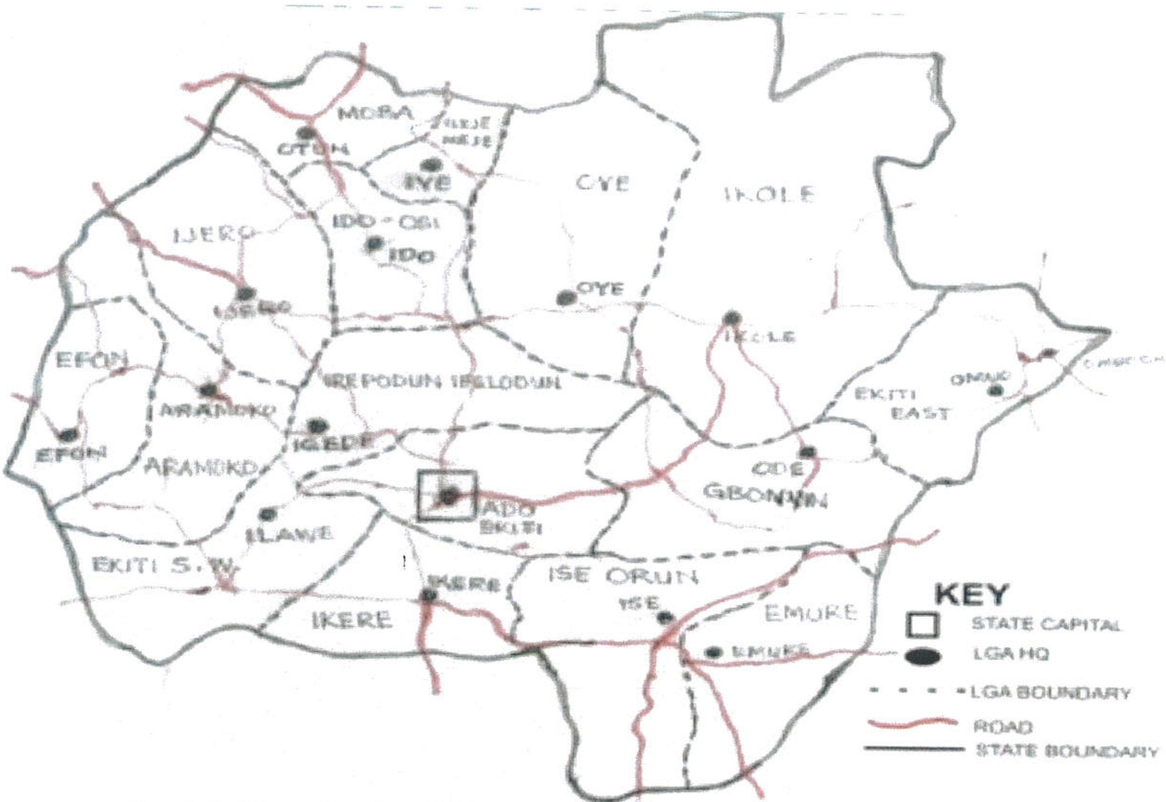


Fig. 1.1: Maps showing Ikole area and its surrounding local government

1.3 Problem Statement

Lateritic soils are generally used for road construction in Nigeria. Lateritic soils in its natural state generally have low bearing capacity and low strength due to high clay content. The strength and stability of lateritic soil containing large amounts of clay cannot be guaranteed under load in the presence of moisture (Alhassan M., 2008). The use of lateritic soils consisting of high plastic clay content results in cracks in and damage to pavement, roadways, foundations or any civil engineering construction.

The need to improve the strength and durability of lateritic soil in recent times has become imperative; this has geared researchers towards using stabilizing materials that can be sourced locally at a very low cost (Bello *et* al., 2015). These local materials can be classified as either agricultural or industrial wastes. In cases where sourcing for durable soil may prove economically unwise, the viable option is to stabilize the available soil to meet the specified requirements of construction (Nnochiri and Aderinlewo, 2016).

1.4 Justification

According to Nnochiri and Aderinlewo (2016), the chemical composition of oil palm frond when burnt at a temperature produces ash whose chemical properties are similar to that of Ordinary Portland Cement.

In addition, the research result as reported by Ankit et al., (2013), shows that the engineering properties of the soil are generally improved when laterite is stabilized with lime.

On this basis, this research desires to check the effect of oil palm frond ash and lime on the geotechnical properties of the lateritic soil from three pits along Ikole – Omuo road.

1.5 Aim and Objectives

The aim of this project is to investigate the effect of the oil palm frond ash on the geotechnical properties of a lime stabilized lateritic soil. The specific objectives of this project are:

i. To study the geotechnical properties of the lateritic soil sample

ii. To study the geotechnical properties of lime stabilized lateritic soil

iii. To study the geotechnical properties of oil palm frond ash stabilized lateritic soil

iv. To study the geotechnical properties of oil palm frond ash on a lime stabilized lateritic soil

1.6 Scope of the Study

This research is limited to checking the overall effect of oil palm frond ash on the geotechnical properties of a lime stabilized lateritic soil. The geotechnical properties of soil samples obtained along Ikole - Omuo road was firstly investigated. Afterwards, the soil samples were stabilized with lime at varied proportions and the Engineering properties re-assessed. Thereafter, oil palm frond ash was introduced at varied proportions and the properties equally reassessed. Finally, lime and oil palm frond ash was introduced also at varied proportions and the resulting effects compared with the properties prior to their addition. This was determined through various laboratory engineering tests such as natural moisture content, specific gravity, atterberg limit tests, sieve analysis, standard proctor compaction test, unconfined compression test and California bearing ratio test on the soil samples.

CHAPTER TWO

LITERATURE REVIEW

2.1 Background

Majorly, For the purposes of road construction, it is seen that various categories of soil are brought in the construction site to act as a sub grade or a sub base and these is the soil carrying any load that will eventually be imposed on the soil after the construction processes are over therefore it must suitably meet the requirements according to specific codes and practices to last the minimum number of years. In many places, one type of soil is usually predominant. This predominant group may not be resistant to lateral deformation when some loads are imposed. The resistant to deformation is dependent on the bearing capacity of the soil. The shear strength plays a great role and derives its value from cohesion and frictional resistance and forces. A stable soil is one which can withstand failure under small deformations and varying weather conditions. From the foregoing, stabilization of soil may then be defined as a soil improvement technique in which the resistance of the soil to the various types of deformations and forces is increased.

2.2 Soil Stabilization

Soil stabilization is a general term for any physical, chemical, biological, or combined method of changing a natural soil to meet an engineering purpose. Improvements include increasing the weight bearing capabilities, tensile strength, and overall performance of in-situ sub-soils, sands, and other waste materials in order to strengthen road surfaces. There are generally various methods of stabilizing soils carried out by researchers and may be grouped but not limited to as follows:

1. Mechanical stabilization method.
2. Cementatious stabilization method.
3. Bituminous stabilization method.
4. Chemical stabilization method.

5. Specialized methods.

1. **Mechanical stabilization method**

It is a technique in which the improvement in the property of the soil is achieved by the addition of another soil of suitable grading. The stabilizing soil may be the same type of soil or a different one, in each case the stabilizing soil is of suitable grading and in most cases, the stabilizing soil is usually brought from another site. It can be said to be the most important and most used method of stabilization of soil in the tropical and poorer areas of the world. It is also used in the selection of base and sub-base materials in road construction and in selecting surfacing materials in earth and agricultural roads

2. **Cemetatious stabilization method**

From research, Soils mixed with small quantities of cementing materials have increased strength due to cementing action of the materials. The cementatious materials are 2-6% of Portland cement; 2-10% of Lime (hydrated or quick lime) reduces the thickness of the water film surrounding the soil particle, giving strength as a result of flocculation; lime with fly-ash which modifies the clay mineral sand and imparts strength; 2-3% of sodium silicae which results in increase in the strength of the mixture.

3. **Bituminous stabilization method**

Research has it that he mixture of bitumen with soil particles may bind the particles or waterproof the solid particles or both effects may occur. As a result of the high cost of bitumen, most times the soil for the bitumen stabilization is penetrated with about 1% of lime.

4. **Chemical stabilization method**

It is generally the use of various chemicals for stabilization of soil in small quantities. By far the most widely used are the resins, they are waterproofing agents preventing water entry particularly in climates in which the soil is nearly always wet. For example, calcium chloride added to soils decreases the rate of evaporation by lowering the vapour pressure of water and increasing its surface tension.

5. **Specialized methods**

They include chemical solidification of soils, thermal stabilization, electrical stabilization etc. these are used in cases where other simpler and cheaper methods like mechanical, cementatious stabilizations etc., do not produce the required result.

2.3 Engineering Index of Soil

2.3.1 Natural Moisture Content

The natural water content also called the natural moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage.

In almost all soil tests, natural moisture content of the soil is to be determined. The knowledge of the natural moisture content is essential in all studies of soil mechanics. To sight a few, natural moisture content is used in determining the bearing capacity and settlement. The natural moisture content will give an idea of the state of soil in the field.

2.3.2 Particle Size Analysis

2.3.2.1 Unified Soil Classification System

Unified soil classification system is adopted by ASTM D-2487-98 and IS: 1498-1970 for classification and identification of soils for general engineering purpose.

According to Unified soil classification system, Soils are broadly classified into three divisions:

i. **Coarse grained soils:** In these soils, 50% or more of the total material by weight is larger than 75micron IS sieve size.

ii. **Fine grained soils:** In these soils, 50% or more of the total material by weight is smaller than 75micron IS sieve size.

iii. **Highly organic soils and other miscellaneous soil materials:** These soils contain large percentage of fibrous organic matter, such as peat, and the particles of decomposed vegetation.

In addition, certain soils containing shells, cinders and other non-soil materials insufficient quantities are also grouped in this division:

1. **Coarse grained soils:**

Coarse grained soils are further divided into two sub-divisions:

a) Gravels (G): In these soils more than 50% of the coarse fraction (+75micron) is larger than 4.75mm sieve size. This sub-division includes gravels and gravelly soil and is designated by symbol G.

b) Sands (S): In these soils, more than 50% of the coarse fraction is smaller than 4.75mm IS sieve size. This sub-division includes sands and sandy soils.

Each of the above sub-divisions is further divided into four groups depending upon grading and inclusion of other materials.

W: WellGraded

C: Claybinder

P: Poorlygraded

M: Containing fine materials not covered in other groups.

These symbols used in combination to designate the type of grained soils. For example, GC means clayey gravels.

2. **Fine grained soils:**

Fine grained soils are further divided into three sub-divisions:

a) Inorganic silts and very fine sands: M

b) Inorganic clays: C

c) Organic silts and clays and organic matter: O

The fine grained soils are further divided into the following groups on the basis of the following arbitrarily selected values of liquid limit which is a good index of compressibility:

i. Silts and clays of low compressibility:

Having a liquid limit less than 35 and represented by symbol L.

ii. Silts and clays of medium compressibility:

Having a liquid limit greater than 35 and less than 50 and represented by symbol I.

iii. Silts and clays of high compressibility:

Having a liquid limit greater than 50 and represented by a symbol H.

Combination of these symbols indicates the type of fine grained soil. For example, ML means inorganic silt with low to medium compressibility.

2.3.2.2 AASHTO Classification

The AASHTO Soil Classification System was developed by the American Association of State Highway and Transportation Officials and is used as a guide for the classification of soils and soil-aggregate mixtures for highway construction purposes. The classification system was first developed by Hogentogler and Terzaghi in 1929, but has been revised several times since.

General Classification	Granular Materials (35% or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35% Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing											
2.00 mm (No. 10)	50 max.	—	—	—	—	—	—	—	—	—	—
0.425 mm (No. 40)	30 max.	50 max.	51 min.	—	—	—	—	—	—	—	—
0.075 mm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit	—		—	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N.P.	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.[a]
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

[a] Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

From *Standard Specification for Transportation Materials and Methods of Sampling and Testing*. Copyright 1990 by the American Association of State Highway and Transportation Officials, Washington, D.C. Used by permission.

Fig. 2.1: AASHTO Soil Classification System and Symbol Chart

2.3.3 Specific Gravity

Specific gravity G is defined as the ratio of the weight of an equal volume of distilled waters at that temperature both weights taken in air. The knowledge of specific gravity is needed in calculation of soil properties like void ratio, degree of saturation etc.

The specific gravity of the soil particles lies within the range of 2.65 to 2.85. Soils containing organic matter and porous particles may have specific gravity values below 2.0. Soils having heavy substances may have values above 3.0.

2.3.4 Atterberg Limits

Amuu and Akinyele (2005) says The plasticity characteristics of a soil often are a good indicator of the swell potential as indicated in Table 2.0 below. If it has been determined that a soil has potential for excessive swell, lime treatment may be appropriate. Lime will reduce well in an expansive soil to greater or lesser degrees depending on the activity of the clay minerals present.

Table 2.0: Swell potential of soils

Liquid limit	Plasticity index	Potential Swell
> 60	35	High
50 – 60	25 – 35	Marginal
< 50	< 25	Low

2.3.5 Soil Compaction

Soil compaction is a vital part of the construction process. It is used for support of structural entities such as building foundations, roadways, walkways, and earth retaining structures to name a few. For a given soil type certain properties may deem it more or less desirable to perform adequately for a particular circumstance. In general, the preselected soil should have adequate strength, be relatively incompressible so that future settlement is not significant, be stable against volume change as water content or other factors vary, be durable and safe against deterioration, and possess proper permeability. When an area is to be filled or backfilled the soil is placed in layers called lifts. The ability of the first fill layers to be properly compacted will depend on the condition of the natural material being covered. If unsuitable material is left in place and backfilled, it may compress over a long period under the weight of the earthfill, causing settlement cracks in the fill or in any structure supported by the fill. In order to determine if the natural soil will support the first fill layers, an area can be proof rolled. Proof rolling consists of utilizing a piece heavy construction equipment (typically, heavy compaction equipment or hauling equipment) to roll across the fill site and watching for deflections to be revealed. These areas will be indicated by the development of rutting, pumping, or ground weaving. To ensure adequate soil compaction is achieved, project specifications will indicate the required soil density or degree of compaction that must be achieved. These specifications are generally recommended by a geotechnical engineer in a geotechnical engineering report.

The soil type – that is, grain-size distributions, shape of the soil grains, specific gravity of soil solids, and amount and type of clay minerals, present has a great influence on the maximum dry unit weight and optimum moisture content. It also has a great influence on how the materials should be compacted in given situations. Compaction is accomplished by use of heavy equipment. In sands and gravels, the equipment usually vibrates, to cause re-orientation of the soil particles into a denser configuration. In silts and clays, a sheepsfoot roller is frequently used, to create small zones of intense shearing, which drives air out of the soil. Determination of adequate compaction is done by determining the in-situ density of the soil and comparing it to the maximum density determined by a laboratory test. The most commonly used laboratory test is called the Proctor compaction test and there are two different methods in obtaining the maximum density. They are the Standard Proctor and Modified Proctor tests; the modified Proctor is more commonly used. For small dams, the standard Proctor may still be the reference. While soil understructures and pavements needs to be compacted, it is important after construction to decompact areas to be landscaped so that vegetation can grow.

2.3.5.1 Compaction Methods

There are several means of achieving compaction of a material. Some are more appropriate for soil compaction than others, while some techniques are only suitable for particular soils or soils in particular conditions. Some are more suited to compaction of non-soil materials such as asphalt. Generally, those that can apply significant amounts of shear as well as compressive stress are most effective. The available techniques can be classified as:

1. Static - a large stress is slowly applied to the soil and then released.
2. Impact - the stress is applied by dropping a large mass onto the surface of the soil.

3. Vibrating - a stress is applied repeatedly and rapidly via a mechanically driven plate or hammer. Often combined with rolling compaction.

4. Gyrating - a static stress is applied and maintained in one direction while the soil is a subjected to a gyratory motion about the axis of static loading. Limited to laboratory applications.

5. Rolling - a heavy cylinder is rolled over the surface of the soil. Commonly used on sports pitches. Roller compactors are often fitted with vibratory devices to enhance their effectiveness.

6. Kneading - shear is applied by alternating movement in adjacent positions. An example, combined with rolling compaction, is the 'sheepsfoot' roller used in waste compaction at landfills.

2.3.5.2 Test methods in laboratory

Soil compactors are used to perform test methods which cover laboratory compaction methods used to determine the relationship between molding water content and dry unit weight of soils.

Soil placed as engineering fill is compacted to a dense state to obtain satisfactory engineering properties such as, shear strength, compressibility or permeability. In addition, foundation soils are often compacted to improve their engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and molding water content needed to achieve the required engineering properties, and for controlling construction to assure that the required compaction and water contents are achieved. Test methods such as EN13286-2, EN 13286-47, ASTMD698, ASTMD1557, AASHTOT99, AASHTOT180, AASHTOT193, BS1377:4 provide soil compaction testing procedures.

2.3.6 Unconfined Compressive Strength

Unconfined compressive strength is the strength of a rock or soil sample when crushed in one direction (uniaxial) in a triaxial test without any lateral restraint.

2.3.7 California Bearing Ratio

The California Bearing Ratio (CBR) is a penetration test for evaluation of the mechanical strength of natural ground, subgrades and base courses beneath new carriage way construction. It was developed by the California Department of Transportation before World War II.

The basic site test is performed by measuring the pressure required to penetrate soil or aggregate with a plunger of standard area. The measured pressure is then divided by the pressure required to achieve an equal penetration on a standard crushed rock material. The CBR test is described in ASTM Standards D1883-05 (for laboratory-prepared samples) and D4429 (for soils in place in field), and AASHTOT193. The CBR test is fully described in BS1377: Soils for civil engineering purposes: Part 4, Compaction related tests and in Part 9, In-situ tests.

The CBR rating was developed for measuring the load-bearing capacity of soils used for building roads. The CBR can also be used for measuring the load-bearing capacity of unimproved airstrips or for soils under paved airstrips. The harder the surface, the higher the CBR rating. A CBR of 3 equates to tilled farmland, a CBR of 4.75 equates to turf or moist clay while moist sand may have a CBR of 10. High quality crushed rock has a CBR over 80. The standard material for this test is crushed California limestone which has a value of 100, meaning that it is not unusual to see CBR values of over 100 in well-compacted areas. The harder the material, the higher the CBR value. A CBR value of 2% is usually found for clay, high-quality sub-base will have CBR values between 80% and 100% and some sands may have values around 10%.

The unit load (pressure) on the plunger for 250 mm or 500 mm of penetration is divided by the unit load of the standard material and the result is multiplied by 100.

Mathematically expressed as: CBR value = $\frac{P}{Ps} \times 100$

P = measured pressure for site soils [N/mm^2]

P_s = pressure to achieve equal penetration on standard soil [N/mm^2]

P_s @ 250mm = 13.24

P_s @ 500mm = 19.96

The bearing capacity of the standard material is considered as a reference value for this test which reveals why the CBR values are percentages, indicating the strength of a granular material in relation to the standard material.

2.4 Additives

Additives are those agro-based, bio-wastes or industrial chemicals added to the soil to enhance their index properties and engineering performance.

2.4.1 Oil Palm Frond

Oil palm fronds are gotten from Oil Palm Tree. Oil palm frond ash is gotten after Oil Palm Frond that have been dried and burnt. This ash contains certain oxides of elements that make its characteristics similar to that of Ordinary Portland Cement in their powdered state.

Table 2.1: Chemical composition of OPFA and OPC

Components (oxides)	OPFA (%)	OPC (%)
CaO	28.66	60.83
ZnO	0.89	NIL
MgO	3.97	3.02
P2O5	3.99	NIL
SiO3	5.59	NIL
Al2O3	14.79	6.47
Fe2O3	4.51	2.79
SiO2	33.67	20.05
K2O	3.41	0.51
Na2O	0.52	0.48
SO3	NIL	0.35
TiO2	NIL	0.38

2.4.2 Hydrated Lime

The Federal Institute of Industrial Research, Oshodi described hydrated lime as an organic product that has many beneficial uses. It is a caustic solid substance, white when pure, is obtained by calcining limestone and other forms of carbonates. Pure lime, also called quicklime is composed basically of calcium oxide (CaO). Upon hydration, quicklime liberates large amount of heat and forms calcium hydroxide, sold commercially as a white powder called slaked or hydrated lime. Hydrated lime has become one of the most important industrial minerals because of its chemical and physical properties as well as its commercial importance.

2.4.2.1 Reactions

According to Ankit *et* al., (2013), the different reactions lime exhibits are discussed below:

1. **Drying**

If quicklime is used, it immediately hydrates (i.e. chemically combines with water) and releases heat. Soils are dried, because water present in the soil participates in this reaction, and because the heat generated can evaporate additional moisture. The hydrated lime produced by these initial reactions will subsequently react with clay particles. These subsequent reactions will slowly produce additional drying because they reduce the soil's moisture holding capacity. If hydrated lime or hydrated lime slurry is used instead of quicklime, drying occurs only through the chemical changes in the soil that reduce its capacity to hold water and increase its stability.

2. **Modification**

After initial mixing, the calcium ions (Ca++) from hydrated lime migrate to the surface of the clay particles and displace water and other ions. The soil becomes friable and granular, making it easier to work and compact. At this stage the Plasticity Index of the soil decreases dramatically, as does its tendency to swell and shrink. The process, which is called "flocculation and agglomeration," generally occurs in a matter of hours.

3. **Stabilization**

When adequate quantities of lime and water are added, the pH of the soil quickly increases to above 10.5, which enables the clay particles to break down. Silica and alumina are released and react with calcium from the lime to form calcium-silicate-hydrates (CSH) and calcium-aluminate-hydrates (CAH). CSA and CAH are cementitious products similar to those formed in Portland cement. They form the matrix that contributes to the strength of lime-stabilized soil layers. As this matrix forms, the soil is transformed from a sandy, granular material to a hard, relatively impermeable layer with significant load bearing capacity. The process begins within hours and can continue for years in a properly designed system. The matrix formed is permanent, durable, and significantly impermeable, producing a structural layer that is both strong and flexible.

2.4.2.2 Effects

1. A reduction in the plasticity index:

The soil suddenly switches from being plastic (yielding and sticky) to being crumbly (stiff and grainy). In the latter condition it is easier to excavate, load, discharge, compact and level.

2. An improvement in the compaction properties of the soil:

The maximum dry density drops, while the optimal water content rises, so that the soil moves into a humidity range that can be easily compacted. This effect is clearly advantageous when used on soils with a high water content, A treatment with quicklime therefore makes it possible to transform a sticky plastic soil, which is difficult to compact, into a stiff, easily handled material. After compacting, the soil has excellent load-bearing properties.

3. Improvement of bearing capacity:

In most cases, two hours after treatment, the CBR (California Bearing Ratio) of a treated soil is between 4 and 10 times higher than that of an untreated soil. This reaction greatly relieves on-site transportation difficulties.

2.5 Laterite

According to Nnochiri and Aderinlewo (2016), Laterites are soil types rich in iron and aluminum that are formed in tropical areas. Most laterites are rusty-red because of the presence of iron oxides. They develop by intensive and long lasting weathering of the underlying parent rock. Tropical weathering (laterization) is a prolonged process of chemical weathering which produces a wide variety in the thickness, grade, chemistry and ore mineralogy of the resulting soils. The initial products of weathering are essentially kaolinized rocks called saprolites (Dalvi et al, 2004). Lateritic soils are products of tropical weathering with red, reddish- brown or dark brown colour, with or without nodules or concretions and generally (but not exclusively) found below hardened ferruginous crusts. Laterite formation factors include climate (precipitation, leaching, capillary rise and temperature), topography (drainage), vegetation, parent rock (iron rich rocks) and time of these primary factors. However, climate is considered to be the most important factor.

2.5.1 The Need to Stabilize Laterites

Lateritic soils are generally used for road construction in Nigeria. Lateritic soils in its natural state generally have low bearing capacity and low strength due to high clay content. The strength and stability of lateritic soil containing large amounts of clay cannot be guaranteed under load in the presence of moisture (Alhassan M., 2008). The use of lateritic soils consisting of high plastic clay content results in cracks in and damage to pavement, roadways, foundations or any civil engineering construction. The need to improve the strength and durability of lateritic soil in recent times hâs become imperative, this has geared researchers towards using stabilizing materials that can be sourced locally at a very low cost (Bello *et* al., 2015). These local materials can be classified as either agricultural or industrial wastes (Amu *et* al., 2011). In cases where sourcing for durable soil may prove economically unwise, the viable option is to stabilize the available soil to meet the specified requirements of construction.

2.6 Pavement

According to Yoder and Witczak (1975), highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade.

2.6.1 Requirements of a pavement

An ideal pavement should meet the following requirements:

i. Sufficient thickness to distribute the wheel load stresses to a safe value on the sub-grade soil,
ii. Structurally strong to withstand all types of stresses imposed upon it,
iii. Adequate coefficient of friction to prevent skidding of vehicles,
iv. Smooth surface to provide comfort to road users even at high speed,
v. Produce least noise from moving vehicles,

vi. Dust proof surface so that traffic safety is not impaired by reducing visibility,

vii. Impervious surface, so that sub-grade soil is well protected, and

viii. Long design life with low maintenance cost.

2.6.2 Types of Pavement

According to Mwangi (2013), pavements can be classified based on the structural performance into two major types: Flexible pavements and Rigid pavements. On the contrary, in rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads). In addition to these, composite pavements are also available. A thin layer of flexible pavement over rigid pavement is an ideal pavement with most desirable characteristics. However, such pavements are rarely used in new construction because of high cost and complex analysis required.

2.6.3 Flexible pavements

Flexible pavements will transmit wheel load stresses to the lower layers by grain-to-grain transfer through the points of contact in the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road).

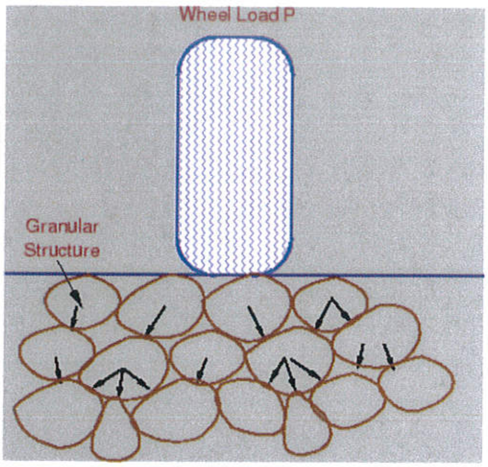


Fig. 2.2: Load transfer in granular structure

2.6.3.1 Types of Flexible Pavements

The following types of construction have been used in flexible pavement:

i. Conventional layered flexible pavement

Conventional flexible pavements are layered systems with high quality expensive materials are placed in the top where stresses are high, and low quality cheap materials are placed in lower layers.

ii. Full - depth asphalt pavement

Full - depth asphalt pavements are constructed by placing bituminous layers directly on the soil sub-grade. This is more suitable when there is high traffic and local materials are not available.

iii. Contained rock asphalt mat (CRAM)

Contained rock asphalt mats are constructed by placing dense/open graded aggregate layers in between two asphalt layers. Modified dense graded asphalt concrete is placed above the sub-grade will significantly reduce the vertical compressive strain on soil sub-grade and protect from surface water.

2.6.3.2 Typical layers of a Flexible Pavement

Typical layers of a conventional flexible pavement includes seal coat, surface course, tack coat, binder course, prime coat, base course, sub-base course, compacted sub-grade, and natural sub-grade.

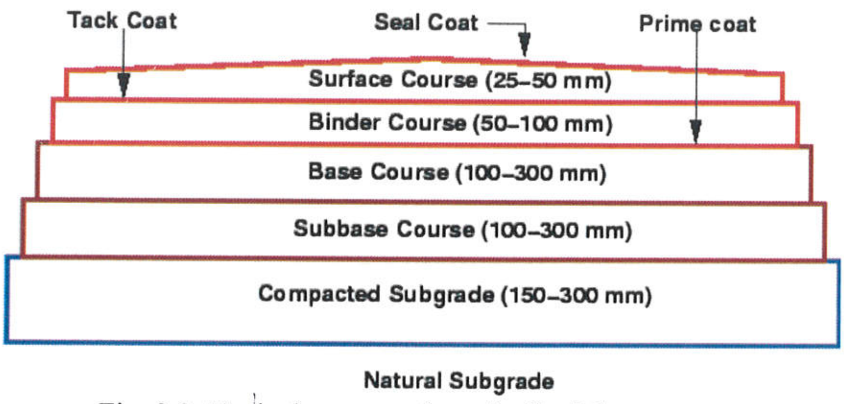


Fig. 2.3: Typical cross section of a flexible pavement

Seal Coat

Seal coat is a thin surface treatment used to water-proof the surface and to provide skid resistance.

Tack Coat

Tack coat is a very light application of asphalt, usually asphalt emulsion diluted with water. It provides proper bonding between two layer of binder course and must be thin, uniformly cover the entire surface, and set very fast.

Prime Coat

Prime coat is an application of low viscous cutback bitumen to an absorbent surface like granular bases on which binder layer is placed. It provides bonding between two layers. Unlike tack coat, prime coat penetrates into the layer below, plugs the voids, and forms a water tight surface.

Surface course

Surface course is the layer directly in contact with traffic loads and generally contains superior quality materials. They are usually constructed with dense graded asphalt concrete (AC). The Functions and requirements of this layer are:

i. It provides characteristics such as friction, smoothness, drainage, etc. Also it will prevent the entrance of excessive quantities of surface water into the underlying base; sub-base and sub-grade,
ii. It must be tough to resist the distortion under traffic and provide a smooth and skid- resistant riding surface,
iii. It must be water proof to protect the entire base and sub-grade from the weakening effect of water.

Binder course

This layer provides the bulk of the asphalt concrete structure. It's chief purpose is to distribute load to the base course The binder course generally consists of aggregates having less asphalt and doesn't require quality as high as the surface course, so replacing a part of the surface course by the binder course results in more economical design.

Base course

The base course is the layer of material immediately beneath the surface of binder course and it provides additional load distribution and contributes to the sub-surface drainage. It may be composed of crushed stone, crushed slag, and other untreated or stabilized materials.

Sub-base course

The sub-base course is the layer of material beneath the base course and the primary functions are to provide structural support, improve drainage, and reduce the intrusion of fines from the sub-grade in the pavement structure If the base course is open graded, then the sub-base course with more fines can serve as a filler between sub-grade and the base course A sub-base course is not always needed or used. For example, a pavement constructed over a high quality, stiff sub-grade may not need the additional features offered by a sub-base course. In such situations, sub-base course may not be provided.

Sub-grade

The top soil or sub-grade is a layer of natural soil prepared to receive the stresses from the layers above. It is essential that at no time soil sub-grade is overstressed. It should be compacted to the desirable density, near the optimum moisture content.

2.7 Literature Review

A number of works have been done by different researchers using different materials, specifications at different times. In relation to this project, some of their works were reviewed; their limitations stated for some of the works and from their successes, a conclusion was drawn out.

Akshaya and Subasis (2013) reviewed various literatures on expansive soils, their engineering properties and recommended various soil-improvement methods. They asserted that an expansive soil is a problematic soil for civil engineers because of its low strength and cyclic swell-shrink behaviour. Stabilization using solid wastes is one of the different methods of treatment, to improve the engineering properties and make it suitable for construction. In the review, they itemized the beneficial effects of some prominent solid wastes as obtained in laboratory studies on stabilization of expansive soil. They also stated other areas where limited studies were carried and suggested further tests that could be performed on some of the solid wastes that were omitted in the literatures reviewed.

According to a review on chemical stabilization by James and Pandian (2015), Chemical stabilization involves the use of chemical agents for initiating reactions within the soil for modification of its geotechnical properties. Cement and lime stabilization have been the most common stabilization methods adopted for soil treatment. Cement stabilization results in good compressive strengths and is preferred for cohesionless to moderately cohesive soil but loses effectiveness when the soil is highly plastic. Lime stabilization is the most preferred method for plastic clays; however, it proves to be ineffective in sulphate rich clays and performs poorly under extreme conditions. With such drawbacks, lots of researches have been undertaken to address the issues faced with each stabilization method, in particular, the use of solid wastes for soil stabilization. Solid waste reuse has gained high momentum for achieving sustainable waste management in recent times. Research has shown that the use of solid wastes as additives with and replacement for conventional stabilizers has resulted in better results than the performance of either individually. This review provided insight into some of the works done by earlier researchers on lime/cement stabilization with industrial wastes as additives and helps to form a sound platform for further research on industrial wastes as additives to conventional stabilizers. Inference obtained from comparison of individual results of stabilization is that addition of waste materials to lime/cement in soil stabilization produced better results than pure lime/cement stabilization with only a few exemptions which produced results on the contrary. The extent of improvement varies from meager to enormous depending upon the combinations. Most of the researchers have concentrated on stabilization of fine-grained expansive soils/clayey soils, which may be predominantly due to the effectiveness of chemical stabilization with fine-grained soils. Index properties like Atterberg limits, compaction characteristics, particle size distribution, and so forth have been predominantly dealt with by most of the researchers. Majority of the researchers too have adopted UCC strength test as the preferred mode of evaluation of strength of the stabilized soil due to quickness and ease of use of the test. The following limitations in their research were that Permeability and compressibility seem to be least concentrated upon among the engineering properties in stabilization. Works involving stabilization of silts and silty sands were also minimal in literature. Effectiveness of other methods of ground improvement like grouting may be the reason for less concentration of literature in the area of stabilization of coarse grained soils.

In the work of James and Pandian (2015), Industrialization has resulted in rapid improvement in the standards of living; however, it has also resulted in pollution and generation of solid wastes that have recently reached epic proportions. An effective waste management alternative is the need of the hour. Reuse of waste materials have been advocated for quite a while now and the utilization of industrial wastes in improving the properties of poor soils open up a new avenue for solid waste management.. Expansive soils have been one of the most problematic soils encountered by a Civil Engineer. A lot of techniques are available for stabilization of such poor soils including lime and cement stabilization. However, the utilization of solid wastes in soil stabilization is an area of potential and promise. And it also provides the double advantage of waste management along with soil improvement. With this as base, the paper reviewed the various industrial wastes that have been adopted in soil stabilization as a standalone stabilizer without lime or cement, in order to shed light into the prospects of increased utilization of solid wastes in soil stabilization. Based on the review of the various works in literature, Utilization of solid wastes in soil stabilization improved the geotechnical properties of the soil while also providing an opportunity for reuse of waste materials. Different industrial wastes provide different degrees of improvement and are suitable for improving soils of different types and are suited for different engineering requirements, thereby enhancing the range of stabilization that can be achieved.

2.7.1 Lime as a Stabilizing Material

Lime is a cheap stabilizer which is effective for stabilizing clayey soils with high plasticity. Lime is also used for stabilizing other soils. However, lime alone is not sufficient to have stabilized soil for a base course material.

Ankit *et* al., (2013) discovered from the chemistry of certain characteristics that Lime exhibits such as drying, stabilization and modification and passed through some tests and procedures and, it was found out to have greatly influenced the characteristics of some soils particularly fine-grained clay soils (with a minimum of 25 percent passing the #200 sieve (74mm) and a Plasticity Index greater than 10). Also, Soils containing significant amounts of organic material (greater than about 1 percent) or sulfates (greater than 0.3percent) may require additional lime. Lime as the stabilization material was first taken to the laboratory to conduct chemical tests on it for its properties and its reaction with other chemicals when added. Its immediate and mid-term effects were checked then, scarification and initial pulverization, lime spreading, preliminary mixing and watering, final mixing and pulverization, compaction and final curing processes were used in adding the lime to the soil in the order listed. They concluded that lime can be used as an excellent soil stabilizing materials for highly active soils which undergo through frequent expansion and shrinkage. Lime acts immediately and improves various property of soil such as carrying capacity of soil, resistance to shrinkage during moist conditions, reduction in plasticity index, increase in CBR value and subsequent increase in the compression resistance with the increase in time. The reaction is very quick and stabilization of soil starts within few hours.

2.7.2 Solid Wastes Ash as a Stabilizing Material

Solid wastes can also be referred to as pozzolanas, that is, possessing cementatious characteristics.

Arinze and Okpara (2015) revealed from their study that Laterite is a soil and rock type rich in iron and aluminum, and is commonly considered to have formed in hot and wet tropical areas. Nearly all laterites are of rusty red coloration, because of high iron oxide content. A lot of industrial waste such as baggasse ash, sugarcane ash, municipal solid waste, cold reclaimed asphalt (RAP), Aircanut coir, sawdust ash, ironstone, tyre ash, bio-enzyme, coconut shell/leaf asphalt ash, kernel shell and corncob ash. This showed that there are great potentials in industrial waste stabilization of laterite. Therefore a large scale stabilization with industrial waste should be embarked by various construction companies and agencies involved in road construction. Moreover, it can be used as a partial replacement for chemical stabilization in order to reduce the enormous cost of chemical stabilization.

In the work of Nnochiri and Aderinlewo, (2016) in third world countries discovered the most common and readily available material that can partially replace cement without economic implications are bio-based materials and agro-based wastes. As a result of the increase in the price of cement which also kept an increase in the construction cost of stabilized road and made it financially high and also that soils are in different forms and possess different characteristics to suit its purposes like lateritic soils which are generally known to have low bearing capacity and low strength due to the presence of high content of clay, they both did a survey around and decided to burn banana leaves to ash as the material to stabilize the soil. This analysis of the geotechnical properties of lateritic soils using ashes of banana leaves as stabilizers while also using ordinary Portland cement as the basis for comparison has been carried out in compliance with BSI (1990) and Head (1992). The chemical composition of both the banana leaves and cement was done and it shows that it has about half of the same chemical composition with cement but in different proportion. The banana leaves ash were added in varying proportion of 2%, 4%, 6%, 8% and 10% by weight of the soil. Preliminary tests were performed on the soil before application and engineering tests were performed on the soil after application. It was discovered to improve the strength property of the soil. The study also revealed that banana leaves ash satisfactorily act as cheap stabilizing agents for sub-grade purposes. Optimum CBR results can be achieved by adding 4 % banana leaves ashes by weight of soil to the natural soil sample. Strength of lateritic soil stabilized with ashes of banana leaves increased. Cement still ranks higher above the banana leaves ash used in the study for improving the CBR of the lateritic soil.

Akinwumi and Aidomojie, (2015) affirmed that portland cement has been effectively used to improve the engineering properties of some local soils for construction of stabilized pavement layers, stabilized earth buildings and support layer for the foundation of buildings. However, cement is expensive and its use is unsustainable, necessitating the search for alternative materials for its total or partial replacement. Research has it that agro-based and bio-based wastes can partially replace cement as it is always readily available. They provided experimental insights on the engineering properties of lateritic soil stabilized with cement-corncob ash (CCA) to ascertain its suitability for use as a pavement layer material. Series of specific gravity, consistency limits, compaction, California bearing ratio (CBR) and permeability tests, considering three CCA blends and four CCA contents, varying from 0 to 12%, were carried out.

From the results obtained, the following conclusions were made:

1. The chemical compositions of the cement and corncob ash indicate that they are richin lime and silica, respectively.

2. As the CCA content in the soil increased, the plasticity, swell characteristics and permeability of the lateritic soil decreased while the bearing capacity and long-term strength increased.

3. The natural soil, which only satisfy the requirements of TRL (1993) and the Nigerian General Specification (1997) for use as a sub-grade material, became improved by CCA stabilization such that it satisfies their requirements for use as a sub-base material.

4. The higher the corncob ash content in the CCA blend, the higher the plasticity, swell potential and the permeability of the soil; and the lower the bearing capacity and strength of the soil. Of the blends, 60C:40CA blend gave the best geotechnical properties.

Mixture of cement and corncob ash, which is cheaper than wholly using cement, can be used to improve soils with similar geotechnical properties to that of the soil used in this study in order to make them better suited for use as pavement layer materials.

Karthi K. S *et al.,* (2014) researched that some waste materials such Fly Ash, rice husk ash, pond ash may use to make the soil to be stable. Addition of such materials will increase the physical as well as chemical properties of the soil. They evaluated the effect of Fly Ash derived from combustion of sub-bituminous coal at electric power plants in stabilization of soft fine-grained red soils. CBR test and other strength property tests were conducted on soils and soil–Fly Ash mixtures prepared at optimum water content of 9%. Addition of Fly Ash resulted in appreciable increases in the CBR of the soil. For water contents 9% wet of optimum, CBRs of the soils are found in varying percentage such that 3,5,6 and 9. The optimum CBR value of the soil was found at 6%. Increment of CBR value is used to reduce the thickness of the pavement and increasing the bearing capacity of soil.

Akshaya K. S, (2012) proposed that Expansive soils are the soils which swell significantly when come in contact with water and shrink when the water squeezes out. Because of this alternate swell- shrink behavior of the soil, damages occur to different civil engineering structures founded on them. The severity of damages done by expansive soil has been well documented in literature worldwide (Chen, 1988; Nelson and Miller, 1992; Gourley et al., 1993). There are a number of techniques available to improve the engineering properties of expansive soil to make it suitable for construction. Stabilization using dust/powder like waste materials with and without a binder like lime, cement etc. is one of them. The expansive soil collected locally was mixed with ceramic dust from 0 to 30% at an increment of 5% and engineering tests were performed on it.

A series of laboratory tests were conducted to study the effects of waste ceramic dust on the, liquid limit, plastic limit, plasticity index, MDD, OMC, UCS, soaked CBR, shear strength parameters and swelling pressure of an expansive soil .Based on the observations and discussions, following conclusions are drawn from this study.

a. The liquid limit, plastic limit and plasticity index go on decreasing irrespective of the percentage of addition of ceramic dust.
b. The addition of 30% ceramic dust changes the soil from CH group to CL group.
c. The MDD goes on increasing and OMC goes on decreasing with increase in percentage of addition of ceramic dust.

d. The UCS goes on increasing with increase in percentage of addition of ceramic dust.
e. The soaked CBR goes on increasing with increase in percentage of addition of ceramic dust. There is 150% increase in soaked CBR value as compared to untreated soil, when 30% ceramic dust was added.
f. The cohesion value goes on decreasing and angle of internal friction goes on increasing with increase in percentage of addition of ceramic dust.
g. The swelling pressure goes on decreasing with addition of ceramic dust. There is 81.5% decrease in swelling pressure of soil as compared to untreated soil, when 30% ceramic dust was added.

From the economic analysis it is found that ceramic dust up to 30% can be utilized for strengthening the sub-grade of flexible pavement with a substantial save in cost of construction.

Edeh *et* al., (2012) noticed that in recent times, the demand for good flexible pavement materials accentuated by design guidelines that are based on the assumptions that aggregates are important ingredient of pavement structure, has increased due to increased constructional activities in the road sector and paucity of available construction materials along road alignments. To overcome this problem, the different alternative generated waste materials, including reclaimed asphalt pavements (RAP) scarified from failed highway pavement, deposited in large quantities along reconstructed road alignment, is stabilized with palm kernel shell ash (PKSA). The material is deposited in large quantities, as waste on production sites, which cause not only environmental hazard but disposal problems. Domestic and industrial wastes are generated everyday and in large quantities and the safe disposal of these waste materials are increasingly becoming a major concern around the world (ETL, 1999; Gardner, 2011; Gomes et al., 2011; Hossain et al., 2011; Wen and Wu, 2011; Osinubi and Edeh, 2011). They evaluated of the characteristics of palm kernel shell ash (PKSA) stabilized reclaimed asphalt pavement (RAP) with a view to determine its suitability for use as highway material in flexible pavements. The RAP - PKSA mixtures were subjected to different preliminary and engineering tests. An experimental approach was used to assess the suitability of RAP stabilized with non-self-cementing PKSA as highway pavement material. The improved particle size distribution of PKSA stabilized RAP contain 95.2–99.5% coarse materials with 0.5 – 4.8% fines content and falls under AASHTO classifications of A-1-a and A-1-b described as very gravelly SAND. The materials are generally non plastic. The specific gravity of 100%RAP and 100%PKSA are 1.81 and 1.31, respectively, while the values for the various RAP/PKSA mixes are in the range 1.18 - 2.42. The maximum dry density (MDD) decreased as the optimum moisture content (OMC) of the RAP/PKSA mixes increased with higher PKSA content. The highest MDD of 1.88 Mg/m3 with corresponding OMC of 24.5% is obtained for 90%RAP/10%PKSA mix proportion. The experiments are based on local waste materials of PKSA and RAP generated and deposited in large quantities resulting in environmental problems. The evaluations of the waste are limited to laboratory experiments whose results can be used as a control to field work. The strength empirical parameter of California bearing ratio is still used as a bases to characterizing road construction materials in developing countries of the world. The 90%RAP/10%PKSA mix with CBR value of 17.11 and 21.39% (soaked and unsoaked values, respectively) achieved using BSL compaction energy can be used as sub grade materials in road construction.

Robert M. Brooks, (2009) worked on the potential of RHA-fly ash blend as a swell reduction layer between the footing of a foundation and sub-grade. In order to examine the importance of the study, a cost comparison was made for the preparation of the sub-base of a highway project with and without the admixture stabilizations. Remolded expansive clay was blended with RHA and fly ash and strength tests were conducted on it. The following conclusions were made:

1. Stress strain behavior of unconfined compressive strength showed that failure stress and strains increased by 106% and 50% respectively when the fly ash content was increased from 0 to 25%.
2. When the RHA content was increased from 0 to 12%, Unconfined Compressive Stress increased by 97%.
3. When the RHA content was increased from 0 to 12%, CBR improved by 47%.
4. The optimum RHA content was found at 12% for both UCS and CBR tests.

5. The swelling potential of expansive soil decreases with increasing swell reduction layer thickness ratio.

6. The vertical movement of clay soils with cushioning material stabilizes after 3 cycles of swelling and shrinkage.

7. An RHA content of 12% and a fly ash content of 25% are recommended for strengthening the expansive subgrade soil while a fly ash content of 15% is recommended for blending into RHA to form a swell reduction layer.

Amuu and Adetuberu (2010) carried out a research to study the characterisitics of bamboo leaf ash stabilization on lateritic soil in highway construction. Bamboo leaf was fired in an open atmosphere and then heated at 600°C for 2 hours in a furnace was found with amorphous material containing amorphous silica. Preliminary tests were performed on three samples, A, B, and C for identification and classification purposes followed by the consistency limit tests. Geotechnical property tests (compaction, California bearing ratio (CBR), and triaxial) were also performed on the samples, both at the stabilized and unstabilized states by adding 2, 4, 6, 8 and 10% bamboo leaf ash (BLA) by weight of sample to the soils. The results showed that the addition of BLA improved the strengths of the samples. Optimum moisture contents reduced to 20.20, 19.60 and 9.32% at 8, 4 and 6% BLA additions in samples A, B and C respectively while MDD increased to 1400, 1676 and 1941 kg/m^3 respectively at 8, 2 and 4% BLA additions in samples A, B, and C. The unsoaked CBR values of samples A and B increased from 5.44 to 38.21% and from 11.42 to 34.99% respectively. The shear strengths of samples A and B also increased from 181.31 to 199.00kN/m^2 and from 144.81 to 155.90 kN/m^2 respectively. It was therefore concluded that bamboo leaf ash has a good potential for stabilizing lateritic soils in highway construction.

In the experimentation between Nnochiri and Aderinlewo (2016), this study assesses the geotechnical properties of lateritic soil stabilized with the ashes of oil palm fronds. These properties are then compared with those of the same soil stabilized with cement to determine how well the ashes perform since cement is considered to be the best stabilizer. Laboratory tests such as specific gravity, moisture content, Atterberg limits, particle size distribution, compaction, unconfined compressive strength (UCS) and California bearing ratio (CBR) tests were first carried out to determine the basic properties of the lateritic soil (without the stabilizers). Based on the results of these tests, the soil was classified according to AASHTO soil classification system as an A-7-5 soil which is a poor soil. Hence, the need for stabilization. Thereafter, strength tests such as California bearing ratio (CBR), unconfined compressive strength (UCS) and compaction tests were performed on the soil to which the ashes and cement were added in percentages of 2, 4, 6, 8 and 10 by weight of the lateritic soil. The compaction test showed that the highest maximum dry densities (MDD) were recorded in the case of the oil palm frond ash (OPFA) and cement at 4% (MDD = 2.02kg/m3) and 6% (MDD = 2.40kg/m3) respectively. The highest CBR values obtained were 32.6% and 87.32% at 4% OPFA content and 6% cement content respectively. The unconfined compressive strengths (UCS) of the soil were highest at 4% OPFA content (234.86kN/m2) and 6% cement content (588.32kN/m2). The chemical tests performed on the OPFA and the cement showed that the highest oxide component were SiO_2 (33.67%) and CaO (60.83%) respectively.

Kumar T. K. *et al.*, (2016) investigated that many geotechnical structures are constructed on weak and loose soil deposits with deeper foundations. Thus for safe design this formation needs improvement before construction starts. A popular technique to improve such soil condition is to use rice husk ash and sugar cane straw ash in the soil. This technique is now being used all over the world in various applications such as embankment, foundations, road pavement, bridges, buildings etc. The soil properties need to be improved by using rice husk ash and sugar cane straw ash as admixtures. For this reasons a detailed investigation of the behaviors of the soil-rice husk ash and soil-sugar cane ash therefore became essential. Sugarcane straw ash which is a voluminous by-product in the sugar mills and jagarry house when juice is extracted from the cane was however generally used as fuel to fire furnaces in the same sugar mill that yields about 8-10% ashes containing high amounts of un-burnt matter, silicon, aluminum, iron and calcium oxides. The ash, therefore, becomes an industrial waste and poses disposal problems. Sugarcane straw ash is rich in amorphous silica indicated that it has pozzolanic

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properties, which helps in stabilization of soil. The study presented that the usage of rice husk ash and sugar cane straw ash with lateritic soil improved the properties of the soil and the influence of addition of materials like rice husk, sugar cane ash. The admixtures was added at a dosage of 2%, 4%, 6% & 8% of total weight of soil and the index properties and engineering properties were determined and compared with the normal soil which is un stabilized. By comparing the soil parameters while using rice husk ash and sugar cane ash we observed that sugar cane straw ash found to be yielding good results. The properties of the lateritic soil are improved by the addition of sugar cane straw ash and rice hush ash at 4% and 6% respectively. Based on the summary of results discussed above, it was concluded that sugarcane straw ash was an effective stabilizer for improving the geotechnical properties of lateritic soil samples. The liquid limit, plastic limit, plastic index, OMC is increased by 12.22%, 7.2%,44.3%&5% respectively for sugar cane straw ash but there is a decrease in MDD by 1.9 %. The liquid limit, plastic limit, plastic index increased by 9.25%, 17.76%&49.22% respectively for sugar cane straw ash but there is a decrease in MDD by 1.98%.

Akshaya and Radhikesh (2011) presented the results of a laboratory study undertaken to investigate the effect of Marble dusts on strength and durability of an expansive soil stabilized with optimum percentage of Rice Husk ash (RHA). The optimum percentage of RHA was found out be 10% based on Unconfined Compressive Strength (UCS) tests. Marble dust was added to
RHA stabilized expansive soil up to 30%, by dry weight of the soil, at an increment of 5%. Compaction tests, UCS tests, Soaked California Bearing Ratio (CBR) tests, Swelling pressure tests and Durability tests were conducted on these samples after 7 days of curing. The UCS, and Soaked CBR of RHA stabilized expansive soil increased up to 20% addition of Marble dust. Further addition of Marble dust had negative effects on these properties. The Maximum Dry Density(MDD) and Swelling pressure of expansive soil goes on decreasing and Optimum Moisture Content (OMC) goes on increasing irrespective of the percentage of addition of Marble dust to RHA stabilized expansive soil. From the Durability test results, it was found that the addition of Marble dust had

made the RHA stabilized expansive soil durable. For best stabilization effect, the optimum proportion of Soil: Rice husk ash: Marble dust was found to be 70: 10: 20.

Ogunribido T.H. (2011) investigated the suitability and sugar cane straw ash stabilization requirement of some lateritic soil samples as road construction materials. Three samples A, B and C were collected from three different locations in Akure and stabilized with 0, 2, 4, 6, 8 and 10% of the sugar cane straw ash were subjected to tests such as natural moisture content, specific gravity, linear shrinkage, atterberg limit, shrinkage limit, compaction, California bearing ratio and unconfined compressive strength. The result of the preliminary tests with the sugar cane ash stabilizer indicates soils in the study area are poor for road construction. The suitability of the soil samples A, B and C was improved by the addition of sugar cane straw ash. The optimum percentage by weight of soil of the sugar cane straw ash for the improvement of the geotechnical properties of the lateritic soil is 4%. The soil samples A, B and C can be used as sub grade materials in road construction when there is optimum stabilization with sugar cane straw ash.

2.7.3 Solid Waste Mixed with Lime as a Stabilizing Material

Monica and Sanjeev, (2013) researched that for any construction work, the foundation is very important and it has to be strong to support the entire structure. In this view, the soil plays a critical role and in order for the foundation to be strong and durable, the soil on that particular construction site at a point in time much be checked of its behavioral characteristics. Expansive soils as part of the soil types creates problem for lightly and moderately loaded structures. By consolidating, which is a natural way of improving soil properties, under load and changing volumetrically along with seasonal moisture variation, these problems are manifested through swelling, shrinkage and unequal settlement. A study was carried out and experimental results obtained in the laboratory to check the improvement in the properties of expansive soil with low cost materials (fly ash and lime) in varying percentages mixed at different proportions revealed that the clay stabilized with low cost materials has lesser swelling potential as the increase in optimum moisture content is being observed. From the results, a clear change of the expansive soil texture took place. When lime & fly ash are mixed with the expansive soil, the Plastic limit increases by mixing lime and liquid limit decreases by mixing fly ash, which decreases plasticity index. As the amount of fly ash & lime increases there is apparent reduction in modified dry density & free swell index and increase in optimum moisture content. It was concluded that mixing lime & fly ash in specific proportion with the expansive soil is an effective way to tackle the problem of shrinkage, swelling and unequal settlement.

Musa Alhassan (2008) worked on A-7-6 lateritic soil treated at British Standard Light (BSL) compaction energy with up to 8% lime content (by dry weight of the soil) at 2% variations and each was admixed with up to 8% rice husk ash (RHA) at 2% variations. The soil-lime-RHA mixtures used for permeability test specimens were obtained by first thoroughly mixing dry predetermined quantities of pulverized soil, lime and RHA to obtain a uniform color. The required quantity of water, which is determined from the moisture-density relation for the soil-lime-RHA mixtures, was then added and the mixing continued. After compaction, the specimens and molds were placed in transparent cellophane bags, which were sealed and then cured in a highly humid environment. After the curing period had been attained, the specimens and the molds were removed from the sealed cellophane bags for permeability testing. Effects of the ash on the soil-lime mixtures were investigated with respect to unconfined compressive strength (UCS) and coefficient of permeability. The UCS of the specimens increased with increasing RHA content at specified lime contents to their maximum values at 6% RHA. The coefficient of permeability of the cured specimens decreased with increase in the ash content to their minimum values at 6% RHA content and beyond this point, the permeability slightly rises. These results indicate that no more than 6% RHA can be used to increase UCS and reduce permeability of lateritic soil.

Nnochiri E. S. *et al.,* (2017) in their study assessed the geotechnical characteristics of lateritic soil and sawdust ash lime (SDAL) mixtures. Preliminary tests were carried out on the natural soil sample for identification and classification purposes.

The sawdust was mixed with lime for stabilization in the ratio 2:1. This mixture was thereafter added to the lateritic soil in varying proportions of 2, 4, 6, 8 and 10% by weight of soil. Addition of SDAL increased values of Optimum Moisture Content (OMC) from 17.0% at 0% SDAL to 26.5% at 10% SDAL by weight of soil, also, values of Maximum Dry Density (MDD) decreased from 2040 kg/m^3 at 0% SDAL to 1415 kg/m^3 at 10% SDAL. Values of Unconfined Compressive Strength (UCS) increased from 38.58 kN/m^2 at 0% SDAL to highest value of 129.63 kN/m^2 at 6% SDAL. The values of liquid limits and plasticity index of the soil were effectively reduced with the addition of the SDAL, from 54.0% at 0% SDAL to 49.0% at 10% SDAL and from 13.7% at 0% SDAL to 12.5% at 10% SDAL respectively. It was therefore concluded that the sawdust ash lime (SDAL) mixture can serve as a cheap soil stabilizing agent for poor lateritic soil.

Muthupriya P. *et* al., (2017) found out that when wastes are disposed, they are always in large quantity and if not properly disposed is hazardous to the health. They looked for a way to proffer solution and decided to test its effect as it is readily available on a sample of clay soil with its fluctuating characteristics. They dealt with the complete analysis of the improvement of clay soil properties and its stabilization using industrial waste sand and lime. The experiment was carried out keeping 20% of lime as constant and industrial waste sand 10%, 20% and 30%. Preliminary tests was first performed on the clay soil itself and the values were noted and after the application of the stabilizing material (IWS), the same test was repeated again on the soil samples in varying proportion. There was an appreciable improvement in the optimum moisture content and maximum dry density for the soil treated with industrial waste making it more non plastic. Lime was also used with industrial waste and it was found out to improve the strength behavior of the sub base. The addition of lime and industrial waste mixes to sub base increases the unconfined compressive strength value more than that by ordinary methods. It can potentially reduce ground improvement costs by adopting this method of stabilization. The long-term performance of any construction project depends on the soundness of the underlying soils.

Akshaya K. S., (2012) described the effect of lime on some geotechnical properties of an expansive soil stabilized with optimum percentage of quarry dust. The lime added were 2 to 7 % at an increment of 1%. Atterberg's limit, compaction, consolidated undrained triaxial compression, and durability tests were conducted on these mixes. The effect of 7 and 28 days of curing were also studied on shear strength. It was concluded that addition of lime had increased the plastic limit, shrinkage limit, cohesion, angle of internal friction, optimum moisture content, decreased the liquid limit, plasticity index, maximum dry density of the soil-quarry dust mixes and made the soil-quarry dust mixes durable. Curing also had positive effects on shear parameters.

According to Sambre and Naik (2016) since long, improvement in characteristics of expansive soil by using different materials has been a challenging job for engineers. In modern days of industrialization it became imperative to use waste materials from various industries for expansive soil stabilization so as to reduce the polluting effect of waste materials from various industries and to achieve worthwhile results. The objective of the study was to evaluate the effect of fly ash, pond ash and lime derived from combustion of thermal Power plants in stabilization of expansive Soil. The research aimed at stabilizing and improving the locally available soil around Nashik district of Maharashtra state India. An experimental program was undertaken to examine the effects of fly ash, pond ash and lime on the compaction and strength performance of expansive soil. The soil samples are prepared with different proportions of fly ash (5%) and pond ash, (10%, 20%, 30%, 40%) and lime (4%, 6%, 8%, 10). A series of test were conducted including Index Properties, Consistency Limits, Modified Proctor Test, laboratory Unconfined Compression Strength Tests and $c - \phi$ properties of soil CBR Value. The probable variations in the strength of Fly ash, Pond ash and lime specimens were observed and recorded. It was concluded that Industrial waste material which is cheaply and easily available in abundant amount i.e. pond ash and fly ash can improve the soil with the help of admixture like lime. This solution for soil improvement is environmental friendly and eco-friendly pond ash can replaces the conventional earth material in some of the geotechnical constructions also.

Sarat and Partha, (2013) in their study revealed that Stabilization of Expansive soil is a matter of great concern nowadays and several methods and materials have been suggested for effective stabilization of these expansive soils. Materials formed using reactions between silica and alumina and alkali cations such as sodium or potassium are very similar, at a molecular level, with natural rocks. Alkaline activated materials have been shown to have improved mechanical characteristics at higher levels than cement. However, its application in geotechnical engineering is not reported. The study presented the stabilization of a local expansive soil using alkali activated fly ash. AAFA (Alkali Activated Fly ash) was prepared by mixing a freshly prepared 2M solution of Potassium hydroxide (KOH) solution with the fly ash, keeping the fly ash to fluid ratio as 0.35. After mixing it is kept in a thermostatically controlled oven maintained at 50° C for 2 hours and then after it is allowed to get dried at room temperature for 24 hours. After the AAFA gets dry, it is grounded and sieved through 1.18 mm IS sieve and taken for soil stabilization. Laboratory investigations were carried out on neat clayey soil samples and AAFA mixed soil samples in accordance with the BIS specifications and their results were analyzed and compared. The activated fly ash at different alkali concentrations and potassium hydroxoxide and fly ash ratios were also tested. Different geotechnical properties like Atterberg's limits, compaction and strength behavior of stabilized expansive soil were tested. It was observed that there is a considerable improvement in geotechnical properties of expansive soil with the addition of alkali activated fly ash.

2.7.4 Newly Introduced Stabilization Materials

2.7.4.1 Geotextiles

Bairagi *et* al., (20) studied that Light structures such as highways, railroads, runways, and other lifeline structures, constructed over black cotton soil may be severely damaged due to high swell-shrinkage behaviour of this soil owing to fluctuating water content. In India, black cotton soil cover as high as 20% of the total land area. Because -shrinkage behaviour it is also called expansive soil. Expansive soil are considered to be unsafe with reference to safety of the structure in serviceability aspects, and needs to be tackled in a well engineered manner, if it should be used as a foundation soil. Several ground improvement and ground stabilisation techniques are in use to control the swelling potential of such soil. The use of Jute geotextile is a new and innovative solution, in which a geotextile material is used in expansive soil to stabilize it.

The study attempted to understand the effectiveness of Jute fibres in controlling swelling behaviour of black cotton soil measured in the laboratory with and without use of random reinforced jute fibres in the black cotton soil. The influence of one parameters of random reinforced jute fibres, via, its amount, on the measured reduction of swelling behaviour is systematically studied, under controlled conditions. In the study, soil samples containing 0%, 1%, 2% to 5% of jute fibre were prepared and the shrinkage limit, optimum moisture content, maximum dry density, California bearing ratio and unconfined compressive strength were conducted as per relevant IS code of practice. The test results showed significance decrease in the expansive behaviour of the black cotton soil. The shrinkage limit increase from 8.66% to 14.68%. There was a remarkable increase in California bearing ratio and unconfined compressive strength test results. The C.B.R. value increased from 1.8% to 4.1% and unconfined compressive strength values increased from 1.09kg/cm^2 to 1.35kg/cm^2 if black cotton soil is blended with jute fibres from 0% to 5% by weight of black cotton soil.

2.7.4.2 Fibers

In the work of Rama Rizana (2015), Most buildings and other civil engineering construction projects are started as raw land. The first step to be performed is site investigation in order to know the situation of the site. It is difficult to find location that has perfect soil properties. Therefore various means of improving the soil properties was adopted among was is soil reinforcement. Reinforced soil is originally defined as a soil which is strengthened by a material able to resist tensile stresses and which interacts with the soil through friction and/or adhesion. Subsequently, the meaning of soil reinforcement was broadened, and this term is now also used for other mechanical and structural methods of soil improvement, such as compressive reinforcement by confinement and encapsulation (Hausmann, 1990). The main purpose of soil reinforcement is to increase the stability or soil strength (Bayormy *et al.*, 2007; Liu *et al.*, 2014; Abdi and Zandieh, 2014; Lajevardi *et al.*, 2014), improve bearing capacity and reduce settlements and lateral deformation. Soil reinforcement is not a new concept. The ancient ziggurats found in Iraq, which are more than 3000 years old is one of early examples of soil reinforcement application. Reed-reinforced earth levees were constructed along the Tiber River by the Romans. The modern uses of soil reinforcement appeared in the 1960s with the development of Reinforced Earth retaining walls and geotextile stabilization of haul roads and access roads (Bonaparte *et al.*, 1987). Another solution to reinforce soil is by using fiber. It has been a solution to stabilize thin soil and localized repair of failed slopes. Unlikely geosynthetics, another reinforcement method using fibers is applied by distributing the fibers randomly. Fibers which can be used either natural fibers or synthetic fibers. Hejazi *et al.*, (2012) made a simple study by reviewing more than 100 researches of soil reinforcement by using natural and synthetic fibers.

To prepare sample or specimens of fiber-reinforced soils to be tested in the laboratory, it can be mixed either manually or mechanically by using mixing machine. Whatever the mixing method used, many researches implicitly assume that the fibers will be randomly distributed in the soil mass. That orientation distribution would give the soil strength isotropy.

Effects of fiber-reinforced soil are relatively similar to geosynthetics-reinforced soil for both coarse-grained and fine-grained soils, such as increasing bearing capacity and soil strength (Gray and Ohashi, 1983; Gray and Al-Refeai, 1986; Puppala and Musenda, 2000;Prabakar and Sridhar, 2002;Yetimoglu and Salbas, 2003; Babu *et al.*, 2008, Chauhan *et al.*, 2008; Choudhary, 2010; Tang *et al.*, 2010; Al-Adili *et al*, 2011; Maheshwari *et al.*, 2011; Lirer et al., 2012; Anagnostopoulos *et al.*, 2013; Singh and Gabra, 2013; Muntohar *et al.*, 2013).

It was concluded from the work of Ahmed Maaty (2014) that Silt soil cannot satisfy the requirements of highway construction because of its low strength. A new stabilizer from waste aluminum industry is developed (aluminum chops (AC) and wires (AW)) to evaluate the effect of reinforcing the subgrade with low-cost by-product materials on its mechanical and durability characteristics. Laboratory tests, including modified proctor compaction, compressive strength, splitting tensile strength, and CBR are developed to evaluate the mechanical properties. The durability properties were investigated by studying the influence of environmental conditions such as water immersion effect on compressive strength, mass loss after freezing and thawing cycles, water absorption by capillarity and wetting-drying durability. Moreover, a practical application about the base course thickness saving and its economically viable as well as correlations between mechanical properties were investigated. The results indicated that the aluminium fiber can effectively improve the mechanical and durability characteristics of silt subgrade where the increase in aluminum chops grade leads to improve the majority properties. While aluminum wires of 2.0 cm length produces reduction in CBR and compressive strength compared to smaller length. Stabilization with aluminium fiber has a remarkable influence in reducing the base course thickness (especially at using 4% of AW1.0) and increasing the construction cost saving (especially at using 1% of AW1.0).

2.7.4.3 Scrap Tire Rubber

Carraro A.H. (2013) studied the beneficial use of scrap tire rubber mixed with expansive soils is of interest to civil engineering applications since the swell percent and the swell pressure can be potentially reduced with no deleterious effect to the shear strength of the mixture. However, for applications whose design and analysis rely upon the stiffness characteristics of the materials used (e.g. roadways and foundations), stringent stiffness requirements may be in order as well. Consequently, one of the goals of this study was to investigate the degree to which the stiffness of expansive soil-rubber (ESR) mixtures changes due to rubber addition so that the final mixture can have acceptable stiffness, shear strength and swell potential characteristics, while, at the same time, be entirely developed using sustainable materials. Additionally, conventional chemical stabilization methods typically used to stabilize expansive soils may present additional difficulties associated with the generation of expansive minerals formed as a result of the chemical stabilization process. Thus an alternative stabilization method that does not rely upon chemical stabilization would be useful, particularly due to the very specific combination of geotechnical, environmental and waste management issues encountered along the Front Range in Colorado.

While most of the fundamental background and initial development on ESR technology has already been conducted by the PI's research team, a direct emphasis to local pavement engineering applications were in order, particularly on the resilient modulus characterization of such materials produced using this novel technology. A rubber content of around 10% appears to be beneficial to both reduce the swell potential characteristics of a subgrade soil with high sulfate content from Colorado while preserving minimum levels of its elastic and resilient parameters when compacted using the Modified compaction effort at a level of relative compaction typically adopted in the design of pavement structures in Colorado.

As it is customary during the adoption of novel, alternative technologies, a trade off might exist between environmental, technical and financial requirements. The results generated by this study suggest waste materials widely available in Colorado may be used in a rational and scientific manner to mitigate some of the technical difficulties associated with the conventional design and construction of pavement structures in the area in a way that addresses both engineering and environmental needs.

Other than a specific recipe that can be applicable to the stabilization of other expansive soil deposits in Colorado, the present study provides a general framework for ESR stabilization that may be used as is or further developed and/or used in combination with other types of soil stabilization protocols. The general rationale behind this study is that it is possible to elevate engineering design to a level that takes into account environmental-friendly, technically-sound, and cost-effective alternatives to conventional design practices.

CHAPTER THREE

METHODOLOGY

3.1 Theoretical Background

Cement has commonly being the most widely used material for soil stabilization. Current research by Nnochiri and Aderinlewo, (2016) revealed that oil palm frond ash possess certain pozzolanic characteristic that makes it a good soil stabilizing agent. Due to the increase in cost of cement, the need for a good sub-grade and also the already high cost of highway construction, it becomes necessary to investigate on how to manage the challenge of constructing durable roads on difficult soil terrains at lower cost, without lowering the standards and specifications.

3.2 Materials

The materials to be used for this research include:

i. Soil sample
ii. Lime
iii. Oil palm frond ash
iv. Water
v. Laboratory test equipment

3.2.1 Soil Sample

The lateritic soil samples (disturbed) was gotten from three pits along Ikole – Omuo expressway and air-dried to reduce to minimal the moisture content.

3.2.2 Lime

The hydrated Lime was gotten from a chemical store in Ado-ekiti.

3.2.3 Oil Palm Frond Ash (OPFA)

Dried Oil palm fronds were obtained from the University farm located within the school premises. It was burnt to ashes in the open and the ash was passed through a 1mm sieve to remove unwanted particles and produce a completely smooth texture.

3.2.4 Water

It was gotten from the nearest available portable water source.

3.3 Methods

The following tests were conducted on the lateritic soil sample before and after it is stabilized with lime and OPFA:

i. Natural Moisture Content
ii. Specific gravity
iii. Sieve analysis
iv. Atterberg limit tests
v. Standard Proctor Compaction Test
vi. Unconfined Compressive Strength
vii. California bearing ratio test

3.3.1 Natural Moisture Content

The aim of the test is to determine the natural moisture content of the given soil sample.

Apparatus Required:

i. Non-corrodible air-tight container.
ii. Electric oven, maintain the temperature between 1050C to 1100C.
iii. Desiccator.
iv. Balance of sufficient sensitivity.

Procedure:

1) Clean the container with lid, dry it and weigh it (W1).
2) Take a specimen of the sample in the container and weigh with lid (W2).
3) Keep the container in the oven with lid removed. Dry the specimen to constant weight maintaining the temperature between 1050C to 1100C for a period varying with the type of soil but usually 16 to 24 hours.
4) Record the final constant weight (W3) of the container with dried soil sample.
5) Peat and other organic soils are to be dried at lower temperature (say 600) possibly for a longer period. Certain soils contain gypsum which on heating loses its water of crystallization. If it is suspected that gypsum is present in the soil sample used for moisture content determination, it shall be dried at not more than 800C and possibly for a longer time.

3.3.2 Specific Gravity

Apparatus

i. A jar bottle fitted with stopper
ii. A weighing balance

Procedure

i. The jar bottle and stopper were cleaned, dried and weighed to the nearest 0.1g (J_0)
ii. About 200g of the soil passing BS sieve 425mm was placed in the jar bottle and weighed together with the rubber stopper.
iii. Approximately 500ml of water at room temperature was added to the soil. The rubber stopper was inserted into the bottle. The jar bottle was shaken by hand until the particles are in suspension (2-5 mins)
iv. The stopper was removed and the soil allowed to settle for a few minutes and the jar bottle was filled with distilled water to the brim. The stopper was then placed on the top of the jar bottle.
v. The jar bottle and the stopper were carefully dried on the outside and the whole assembly weighed to the nearest 0.1g (W1).
vi. The jar bottle was emptied, washed thoroughly and filled completely to the brim with water.
vii. The jar bottle was then dried carefully on the outside and the whole assembly weigh to the nearest 0.1g (W_2).

3.3.3 Sieve Analysis

A sieve analysis (or gradation test) is a practice or procedure used (commonly used in civil engineering) to assess the particle size distribution (also called *gradation*) of a granular material. The size distribution is often of critical importance to the way the material performs in use. A sieve analysis can be performed on any type of non-organic or organic granular materials including sands, crushed rock, clays, granite, feldspars, coal, and soil, a wide range of manufactured powders, grain and seeds, down to a minimum size depending on the exact method. Being such a simple technique of particle sizing, it is probably the most common too.

Preparation:

In order to perform the test, a sufficient sample of the aggregate must be obtained from the source. To prepare the sample, the aggregate should be mixed thoroughly and be reduced to a suitable size for testing. The total weight of the sample is also required.

Procedure:

Fig. 3.1: Sieves used for gradation test.

Fig. 3.2: A mechanical shaker used for sieve analysis.

A gradation test is performed on a sample of aggregate in a laboratory. A typical sieve analysis involves a nested column of sieves with wire mesh cloth (screen). See the separate Mesh (scale) page for details of sieve sizing.

A representative weighed sample is poured into the top sieve which has the largest screen openings. Each lower sieve in the column has smaller openings than the one above. At the base is a round pan, called the receiver. The column is typically placed in a mechanical shaker. The shaker shakes the column, usually for some fixed amount of time. After the shaking is complete the material on each sieve is weighed. The weight of the sample of each sieve is then divided by the total weight to give a percentage retained on each sieve. The size of the average particle on each sieve is then analyzed to get a cut-off point or specific size range, which is then captured on a screen.

The results of this test are used to describe the properties of the aggregate and to see if it is appropriate for various civil engineering purposes such as selecting the appropriate aggregate for concrete mixes and asphalt mixes as well as sizing of water production well screens.

The results of this test are provided in graphical form to identify the type of gradation of the aggregate. The complete procedure for this test is outlined in the American Society for Testing and Materials (ASTM) C 136 and the American Association and State Highway and Transportation Officials (AASHTO) T 27.

A suitable sieve size for the aggregate underneath the nest of sieves to collect the aggregate that passes through the smallest. The entire nest is then agitated, and the material whose diameter is smaller than the mesh opening pass through the sieves. After the aggregate reaches the pan, the amount of material retained in each sieve is then weighed.

3.3.4 Atterberg Limit Tests

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

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

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

Shrinkage Limit

The shrinkage limit (SL) is the water content where further loss of moisture will not result in any more volume reduction. The test to determine the shrinkage limit is ASTM International D4943. The shrinkage limit is much less commonly used than the liquid and plastic limits.

Plastic Limit

The plastic limit (PL) is determined by rolling out a thread of the fine portion of a soil on a flat, non-porous surface. The procedure is defined in ASTM Standard D 4318. If the soil is at a moisture content where its behavior is plastic, this thread will retain its shape down to a very narrow diameter. The sample can then be remolded and the test repeated. As the moisture content falls due to evaporation, the thread will begin to break apart at larger diameters. The plastic limit is defined as the moisture content where the thread breaks apart at a diameter of 3.2 mm (about 1/8 inch). A soil is considered non-plastic if a thread cannot be rolled out down to 3.2 mm at any moisture possible.

Liquid Limit

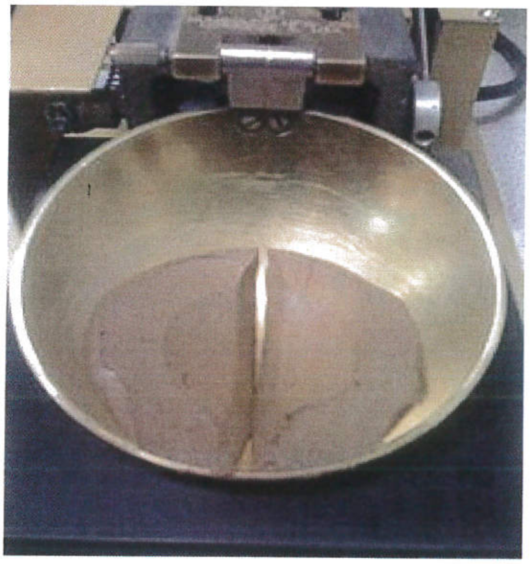

Fig. 3.3: Casagrande cup in action

The liquid limit (LL) is conceptually defined as the water content at which the behavior of a clayey soil changes from plastic to liquid. However, the transition from plastic to liquid behavior is gradual over a range of water contents, and the shear strength of the soil is not actually zero at the liquid limit. The precise definition of the liquid limit is based on standard test procedures described below.

Procedure:

The original liquid limit test of Atterberg's involved mixing a pat of clay in a round-bottomed porcelain bowl of 10–12 cm diameter. A groove was cut through the pat of clay with a spatula, and the bowl was then struck many times against the palm of one hand. Casagrande subsequently standardized the apparatus and the procedures to make the measurement more repeatable. Soil is placed into the metal cup portion of the device and a groove is made down its center with a standardized tool of 2 millimeters (0.079 in) width. The cup is repeatedly dropped 10 mm onto a hard rubber base at a rate of 120 blows per minute, during which the groove closes up gradually as a result of the impact. The number of blows for the groove to close is recorded. The moisture content at which it takes 25 drops of the cup to cause the groove to close over a distance of 12.7 millimeters (0.50 in) is defined as the liquid limit. The test is normally run at several moisture contents, and the moisture content which requires 25 blows to close the groove is interpolated from the test results. The liquid limit test is defined by ASTM standard test method D 4318. The test method also allows running the test at one moisture content where 20 to 30 blows are required to close the groove; then a correction factor is applied to obtain the liquid limit from the moisture content.

Another method for measuring the liquid limit is the fall cone test, also called the cone penetrometer test. It is based on the measurement of penetration into the soil of a standardized cone of specific mass. Although the Casagrande test is widely used across North America, the fall cone test is much more prevalent in Europe due to being less dependent on the operator in determining the Liquid Limit.

Plasticity Index

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. The PI is the difference between the liquid limit and the plastic limit (PI = LL-PL). Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of 0 (non-plastic) tend to have little or no silt or clay.

Soil descriptions based on PI:

i. (0)- Non plastic
ii. (<7) - Slightly plastic
iii. (7-17) - Medium plastic
iv. >17 - Highly plastic

Liquidity Index

The liquidity index (LI) is used for scaling the natural water content of a soil sample to the limits. It can be calculated as a ratio of difference between natural water content, plastic limit, and liquid limit: $LI = (W-PL)/(LL-PL)$ where W is the natural water content

3.3.5 Standard Proctor Compaction Test

Proctor test is carried out to determine compaction of soil to understand compaction characteristics of different soils with change in moisture content. Compaction is the process of densification of soil by reducing air voids. The degree of compaction of a given soil is measured in terms of its dry density. The dry density is maximum at the optimum water content. A curve is drawn between the water content and the dry density to obtain the maximum dry density and the optimum water content.

Apparatus:

1. Compaction mould, capacity 1000ml.

2. Rammer, mass 2.6 kg

3. Detachable base plate

4. Collar, 60mm high

5. IS sieve, 4.75mm

6. Oven

7. Desiccator

8. Weighing balance, accuracy 1g

9. Large mixing pan

10. Straight edge

11. Spatula

12. Graduated jar

13. Mixing tools, spoons, trowels, etc.

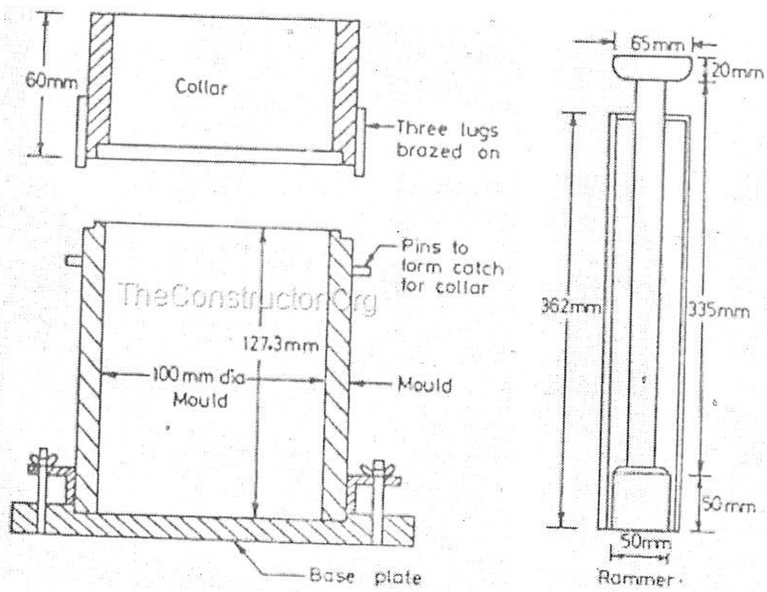

Fig. 3.4: Standard Proctor Compaction Test Apparatus

Procedure:

1. Take about 20kg of air-dried soil. Sieve it through 20mm and 4.7mm sieve.
2. Calculate the percentage retained on 20mm sieve and 4.75mm sieve, and the percentage passing 4.75mm sieve.

3. If the percentage retained on 4.75mm sieve is greater than 20, use the large mould of 150mm diameter. If it is less than 20%, the standard mould of 100mm diameter can be used. The following procedure is for the standard mould.
4. Mix the soil retained on 4.75mm sieve and that passing 4.75mm sieve in proportions determined in step (2) to obtain about 16 to 18 kg of soil specimen.
5. Clean and dry the mould and the base plate. Grease them lightly.
6. Weigh the mould with the base plate to the nearest 1 gram.
7. Take about 16 – 18 kg of soil specimen. Add water to it to bring the water content to about 4% if the soil is sandy and to about 8% if the soil is clayey.
8. Keep the soil in an air-tight container for about 18 to 20 hours for maturing. Mix the soil thoroughly. Divide the processed soil into 6 to 8 parts.
9. Attach the collar to the mould. Place the mould on a solid base.
10. Take about 2.5kg of the processed soil, and hence place it in the mould in 3 equal layers. Take about one-third the quantity first, and compact it by giving 25 blows of the rammer. The blows should be uniformly distributed over the surface of each layer.
11. The top surface of the first layer be scratched with spatula before placing the second layer. The second layer should also be compacted by 25 blows of rammer. Likewise, place the third layer and compact it.
12. The amount of the soil used should be just sufficient to fill the mould ad leaving about 5 mm above the top of the mould to be struck off when the collar is removed.
13. Remove the collar and trim off the excess soil projecting above the mould using a straight edge.
14. Clean the base plate and the mould from outside. Weigh it to the nearest gram.
15. Remove the soil from the mould. The soil may also be ejected out.
16. Take the soil samples for the water content determination from the top, middle and bottom portions. Determine the water content.
17. Add about 3% of the water to a fresh portion of the processed soil, and repeat the steps 10 to 14.

3.3.6 Unconfined Compressive Strength

The unconfined compression test is by far the most popular method of soil shear testing because it is one of the fastest and cheapest methods of measuring shear strength. The method is used primarily for saturated, cohesive soils recovered from thin-walled sampling tubes. The unconfined compression test is inappropriate for dry sands or crumbly clays because the materials would fall apart without some land of lateral confinement.

Aim:

The purpose of this laboratory is to determine the unconfined compressive strength of a cohesive soil sample. We will measure this with the unconfined compression test, which is an unconsolidated undrained (UU or Q-type) test where the lateral confining pressure is equal to zero (atmospheric pressure).

Preparation:

To perform an unconfined compression test, the sample is extruded from the sampling tube. A cylindrical sample of soil is trimmed such that the ends are reasonably smooth and the length-to-diameter ratio is on the order of two. The soil sample is placed in a loading frame on a metal plate; by turning a crank, the operator raises the level of the bottom plate.

The top of the soil sample is restrained by the top plate, which is attached to a calibrated proving ring. As the bottom plate is raised, an axial load is applied to the sample. The operator turns the crank at a specified rate so that there is constant strain rate. The load is gradually increased to shear the sample, and readings are taken periodically of the force applied to the sample and the resulting deformation. The loading is continued until the soil develops an obvious shearing plane or the deformations become excessive. The measure data are used to determine the strength of the soil specimen and the stress-strain characteristics. Finally, the sample is oven dried to determine its water content. The maximum load per unit area is defined as the unconfined compressive strength, qu.

In the unconfined compression test, we assume that no pore water is lost from the sample during set-up or during the shearing process. A saturated sample will thus remain saturated during the test with no change in the sample volume, water content, or void ratio.

More significantly, the sample is held together by an effective confining stress that results from negative pore water pressures (generated by menisci forming between particles on the sample surface). Pore pressures are not measured in an unconfined compression test; consequently, the effective stress is unknown. Hence, the undrained shear strength measured in an unconfined test is expressed in terms of the total stress.

The loading frame consists of two metal plates. The top plate is stationary and is attached to the load-measuring device. The bottom plate is raised and lowered by means of a crank on the front of the loading frame. After the soil sample has been placed between the plates, the bottom plate is gradually raised; the resistance provided by the stationary top plate applies an axial force to the sample. Although the loading frames in our laboratory are hand operated, electric motor-driven and hydraulic load frames are common. Loads are measured with a calibrated proving ring or an electronic load cell. Vertical deformations are measured with a dial gauge; the dial gauge is attached to the top plate and measures the relative movement between the top and bottom plates. We will be performing a strain-controlled test, in which the load is applied at a constant rate of strain or deformation

Procedure:

1. The first step in the procedure is to examine the loading frame. Turn the crank and learn how to read the load and deformation dial gages. Determine the calibration constant for the proving ring and the units of the deformation dial gauge.
2. We will be shearing the samples at a strain rate of 1% per minute. From the length of your soil sample, determine the deformation at 1% strain. Depending on the units of the vertical deformation dial gauge (usually 0.001 inches or 0.0001 inches), determine the number of dial divisions per 1 strain- Practice turning the

crank at his number of dial divisions/minute. It is important that the soil sample not be sheared faster than this specified rate.

3. Measure the initial height and diameter of the soil sample with calipers. It is unlikely that the sample will be a perfect right cylinder. Therefore, it will be necessary to find the average height and diameter by taking several measurements in different places along the soil sample. The measurements should be taken by more than one member of a lab team to be sure that the calipers are read correctly. If you have any questions about how to take measurements with calipers, ask the laboratory instructor for instruction.
4. Record the weight of the soil sample and determine the total (moist) unit weight.
5. Place the soil sample in the loading frame, seat the proving ring and zero the dials.
6. During this lab you will record the load applied at specified strain values. It is recommended that readings be taken at strains of 0, 0.1, 0.2, 0.5, 1,2,3,4,5,6,8, 10, 12 14, 16, 18 and 20 percent. You should prerecord the vertical deformation dial readings at these strain values.
7. Readings of force (F) are taken from the proving ring dial gauge and the stress applied to the ends of the sample (01, or major principal stress)
8. Shear the sample at a strain rate of 1% per minute. Typically, the sample fails in one of two ways. In stiffer clays, a distinct failure plane forms. For this type of failure, it is likely that the point of failure will be indicated by the measurement of a peak and then a decrease in load. If this is the case, continue to take four or five readings past the point of failure. (Caution: before you stop shearing the sample, be sure that the sample has failed.) A "barreling" failure is more typical for softer clays. In this type of failure, distinct failure plane doesn't form, rather the sample bulges in the middle
9. The unconfined compressive strength (qu) is the maximum value $\sigma 1$, which may or may not coincide with the maximum force measurement (depending on the area correction). It is also equal to the diameter of Mohr's circle.

10. When your lab team has completed the experiment, dismantle the loading frame and measure the water content of the soil sample. It is recommended that you reduce the data for this test during the lab period.

3.3.7 California Bearing Ratio Test

The California Bearing Ratio Test (CBR Test) is a penetration test developed by California State Highway Department (U.S.A.) for evaluating the bearing capacity of sub-grade soil for design of flexible pavement. It is the ratio of force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25 mm/min. to that required for the corresponding penetration of a standard material. Tests are carried out on natural or compacted soils in soaked or un-soaked conditions and the results so obtained are compared with the curves of standard test to have an idea of the soil strength of the sub-grade soil.

Apparatus:

Mould, Steel Cutting collar, Spacer Disc, Surcharge weight, Dial gauges, IS Sieves, Penetration Plunger, Loading Machine, Miscellaneous Apparatus CBR Graphs.

Procedure:

1. Normally 3 specimens each of about 7 kg must be compacted so that their compacted densities range from 95% to 100% generally with 10, 30 and 65 blows.
2. Weigh of empty mould
3. Add water to the first specimen (compact it in five layer by giving 10 blows per layer)
4. After compaction, remove the collar and level the surface.
5. Take sample for determination of moisture content.
6. Weight of mould + compacted specimen.
7. Place the mold in the soaking tank for four days (ignore this step in case of unsoaked CBR.

8. Take other samples and apply different blows and repeat the whole process.
9. After four days, measure the swell reading and find %age swell.
10. Remove the mould from the tank and allow water to drain.
11. Then place the specimen under the penetration piston and place surcharge load of 10lb.
12. Apply the load and note the penetration load values.
13. Draw the graphs between the penetration (in) and penetration load (in) and find the value of CBR.
14. Draw the graph between the percentage CBR and Dry Density, and find CBR at required degree of compaction.

CHAPTER FOUR
RESULTS AND DISCUSSION

4.1 Introduction

The careful analysis of the tests result as carried out in the laboratory are interpreted and explained.

4.2 Natural Moisture Content

The natural moisture content of the soils are respectively 9.55%, 6.80% and 10.6% indicating that pit 3 has the highest moisture content and pit 2 has the lowest moisture content of the three soils.

4.3 Specific Gravity

The values for the specific gravity of the soils are 2.52, 2.67 and 3.67 for pit 1, pit 2 and pit 3 respectively.

The specific gravity of soil particles lies within the range of 2.65 to 2.85. Soils containing organic matter and porous particles may have specific gravity values below 2.0. Soils having heavy substances may have values above 3.0.

Das (2000) state that most clay minerals have specific gravity that falls within a general range 1.6 - 2.9.

4.4 Sieve Analysis

According to Unified Soil Classification System, Fine grained has 50% or more of the total material by weight smaller than 75 micron IS sieve size. Fine grained are also further divided on the basis of arbitrarily selected values of liquid limit which is a good index of compressibility. Soils having liquid limit greater than 35% and less than 50% are silts and clays of medium compressibility.

According to AASHTO Method of classification, soils having over 36% passing through 0.075mm or No. 200 sieve, having liquid limit over 41% and plasticity index over 11% are in the A-7 group of clayey soil according to their characteristics of fraction. Soils having plasticity index equal to or less than LL minus 30 are in the A-7-5 subgroup and soils having plasticity index greater than LL minus 30 are in the A-7-6 subgroup. It is fair to poor as a subgrade. Thus, the three soils are in the A-7 group of clayey soils as represented in figures 4.1 to 4.3.

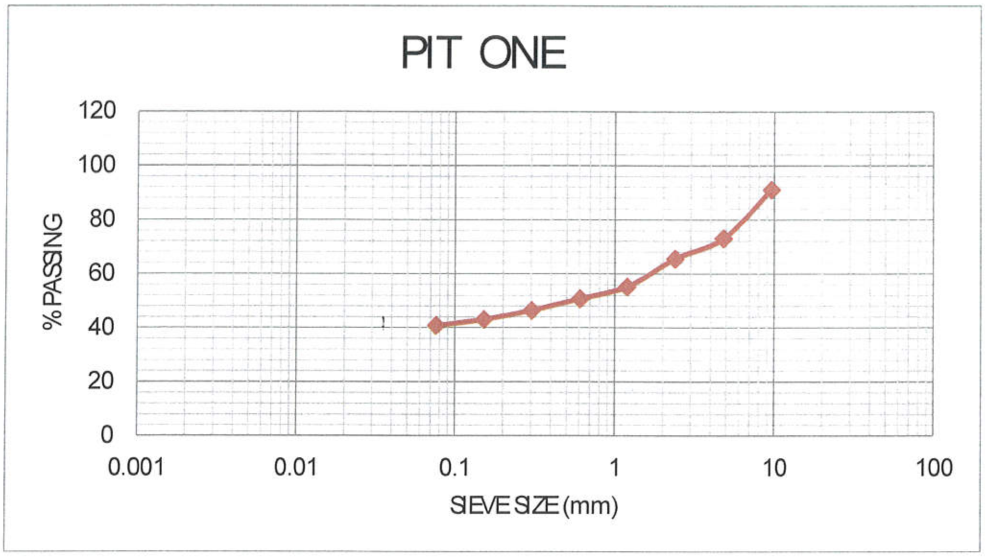

Fig. 4.1: Percentage of soil particles passing in relation to sieve size for pit 1

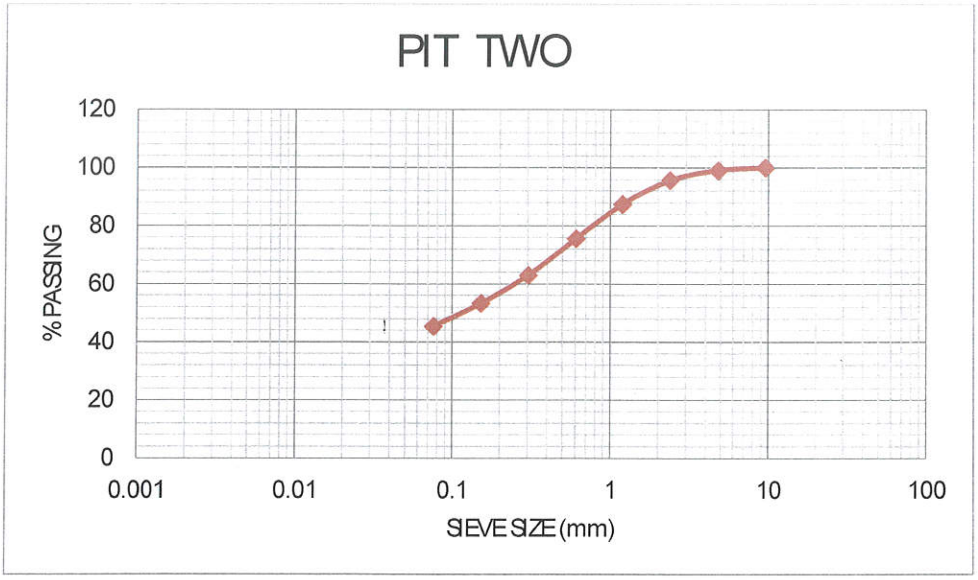

Fig. 4.2: Percentage of soil particles passing in relation to sieve size for pit 2

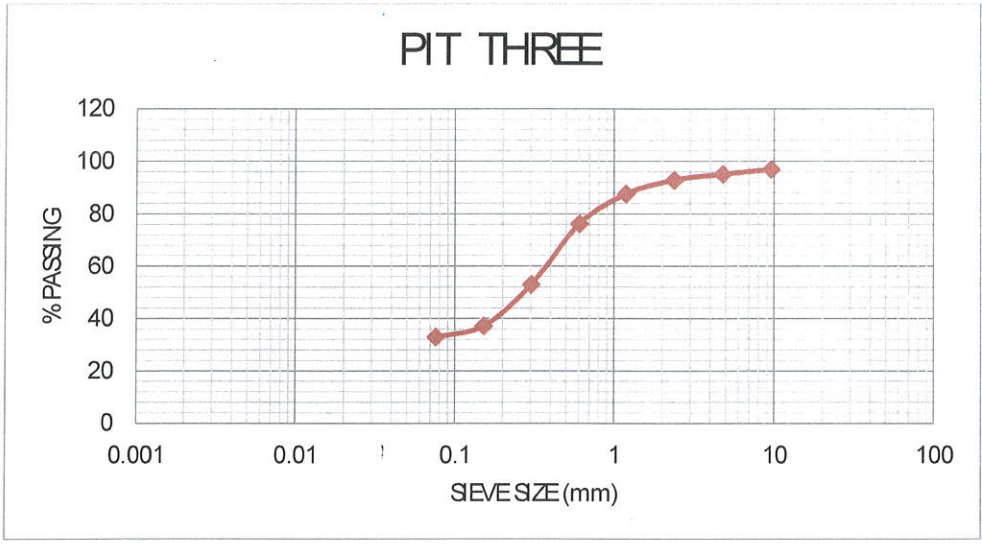

Fig. 4.3: Percentage of soil particles passing in relation to sieve size for pit 3

4.5 Atterberg Limits

At 0% the liquid limits for the three soils are 43.80%, 47.20% and 41.00% while their plasticity index are 15.95%, 22.10% and 24.95% for pit 1, pit 2 and pit 3 respectively.

On addition of lime, the liquid limits reduced from 44.10% to 35.00% and plasticity index reduced from 15.95% to 6.10% at 2% to 8% respectively for pit 1. The liquid limit reduced from 46.20% to 40.30% and plasticity index reduced from 14.25% to 10.10% at 2% to 8% respectively for pit 2. The liquid limit reduced from 41.00% to 38.00% and plasticity index reduced from 20.30% to 2.90% at 2% to 8% respectively for pit 3. These are as represented in figures 4.4 to 4.6 below:

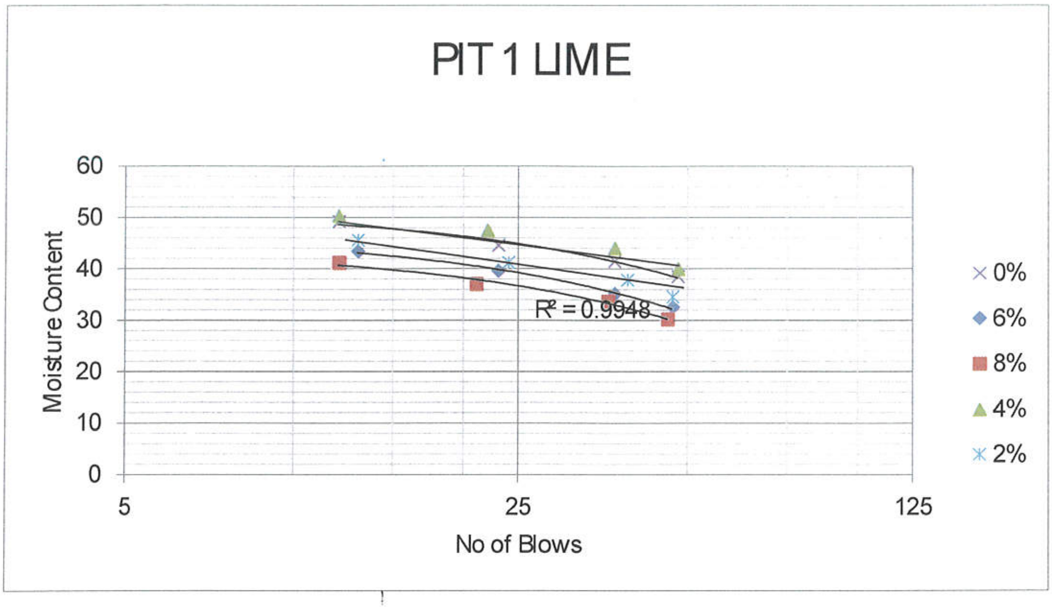

Fig. 4.4: Liquid limit result on addition of lime for pit 1.

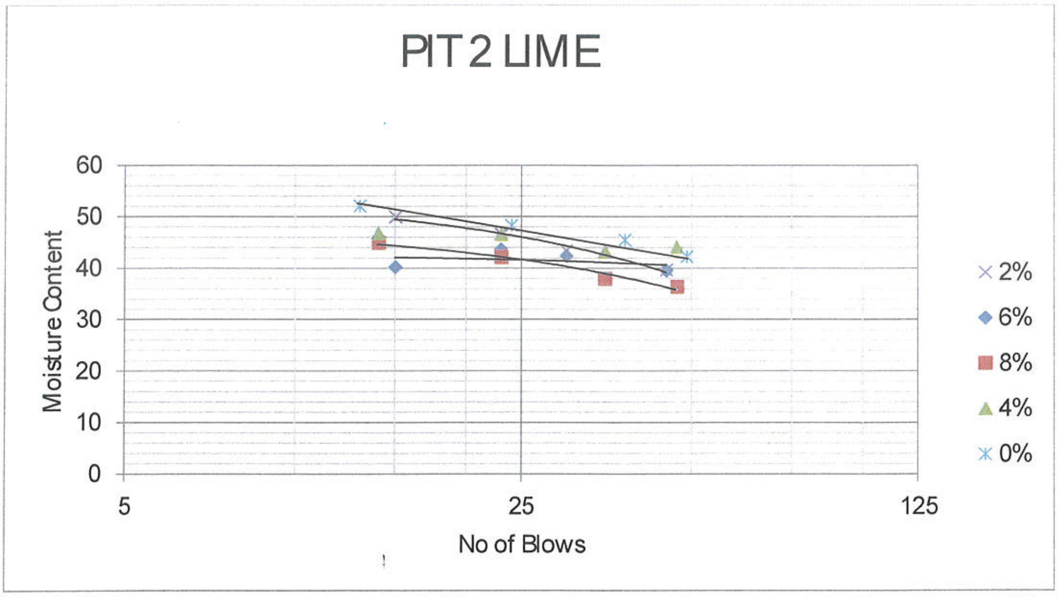

Fig. 4.5: Liquid limit result on addition of lime for pit 2.

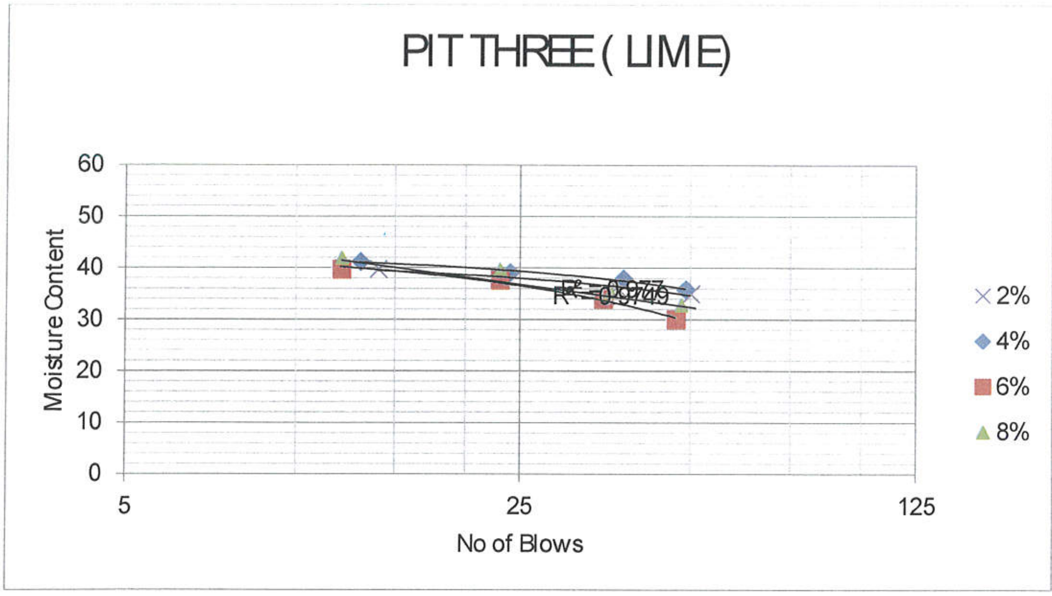

Fig. 4.6: Liquid limit result on addition of lime for pit 3.

On addition of ash, the liquid limits reduced from 40.00% to 39.40% and plasticity index reduced from 21.95% to 19.40% at 2% to 8% respectively but in between for 4% to 6% respectively, the liquid limit increased to 42.10% to 43.80% and plasticity index increased from 22.10% to 24.00% for pit 1. The liquid limit reduced from 45.60% to 40.00% and plasticity index reduced from 27.70% to 20.45% at 2% to 8% respectively for pit 2. The liquid limit reduced from 41.00% to 38.00% and plasticity index reduced from 24.95% to 2.90% at 2% to 8% respectively for pit 3. These are as represented in figures 4.7 to 4.9 below:

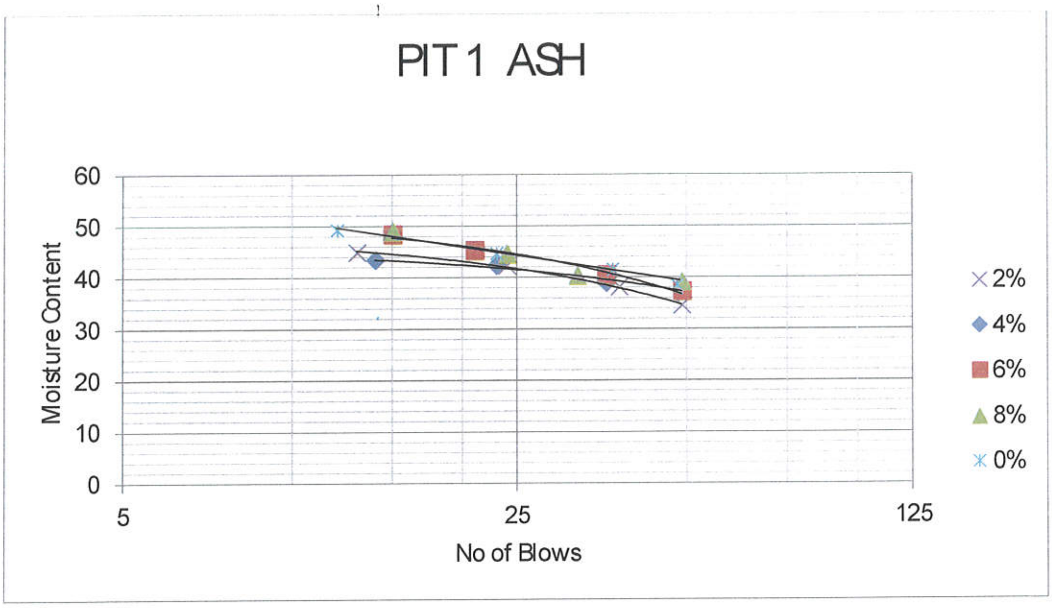

Fig. 4.7: Liquid limit result on addition of ash for pit 1.

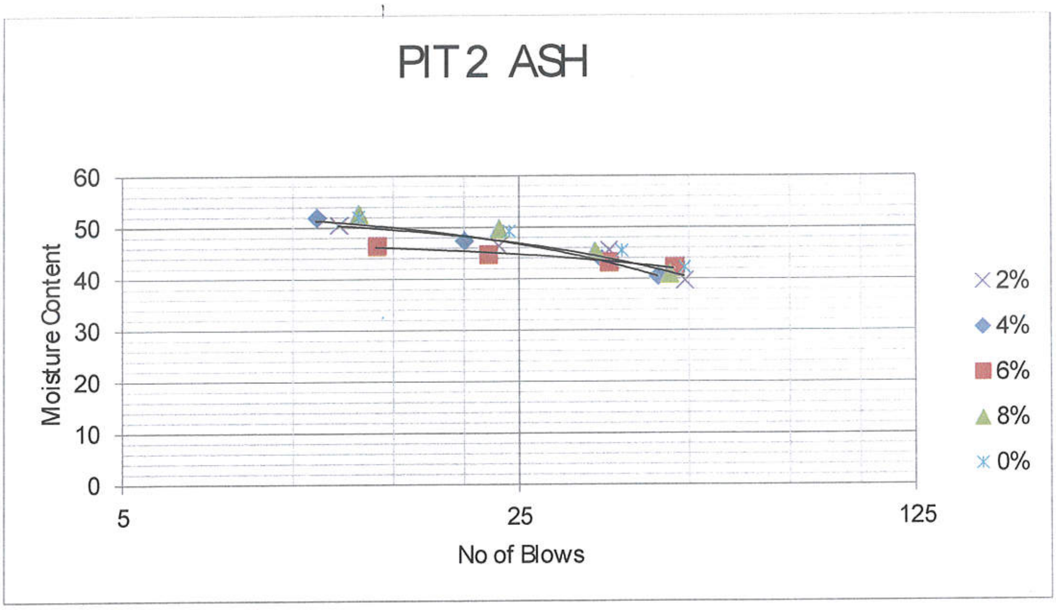

Fig. 4.8: Liquid limit result on addition of ash for pit 2.

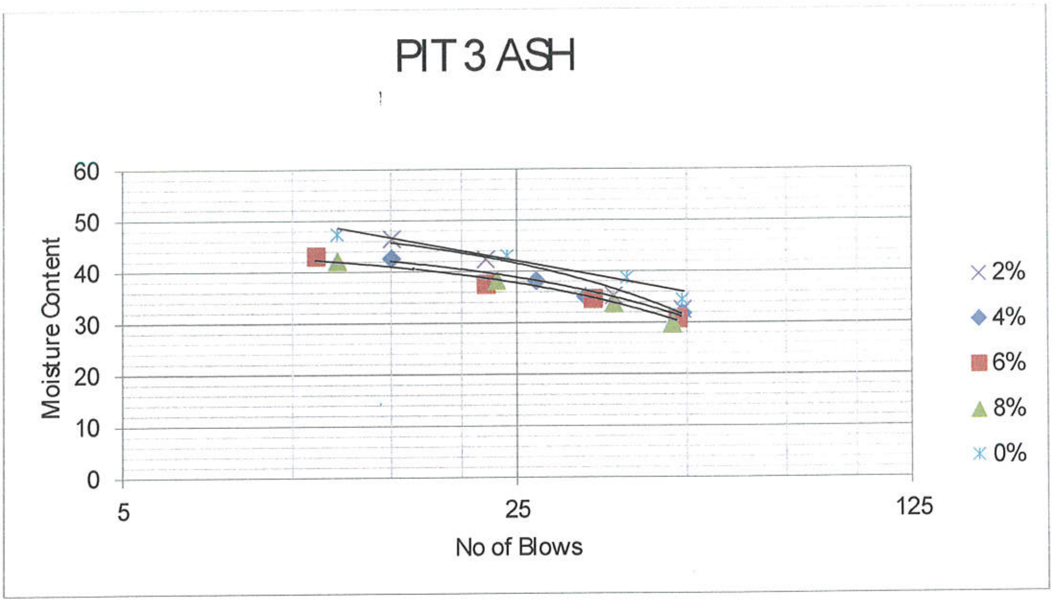

Fig. 4.9: Liquid limit result on addition of ash for pit 3.

On addition of lime and ash, the liquid limits reduced from 44.10% to 33.90% and plasticity index reduced from 24.50% to 20.00% at 2% to 8% respectively for pit 1. The liquid limit reduced from 43.00% to 37.00% at 2% to 8% respectively and plasticity index reduced from 22.90% to 17.30% at 2% to 6% respectively and increased to 18.50% at 8% for pit 2. The liquid limit reduced from 38.00% to 34.20% and plasticity index reduced from 19.30% to 17.90% at 2% to 8% respectively for pit 3. These are as represented in figures 4.10 to 4.12 below:

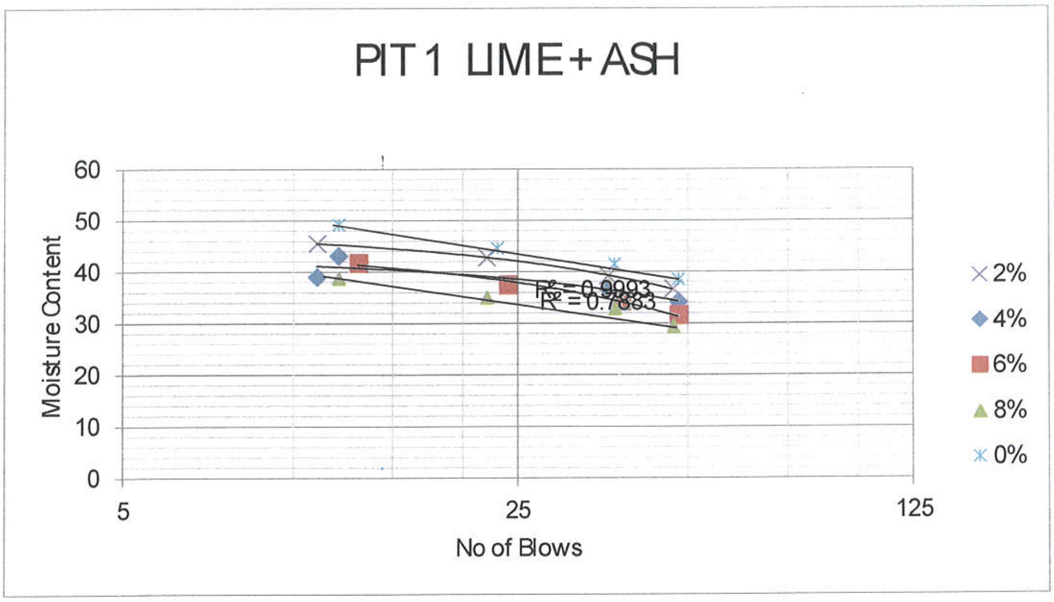

Fig. 4.10: Liquid limit result on addition of lime and ash for pit 1.

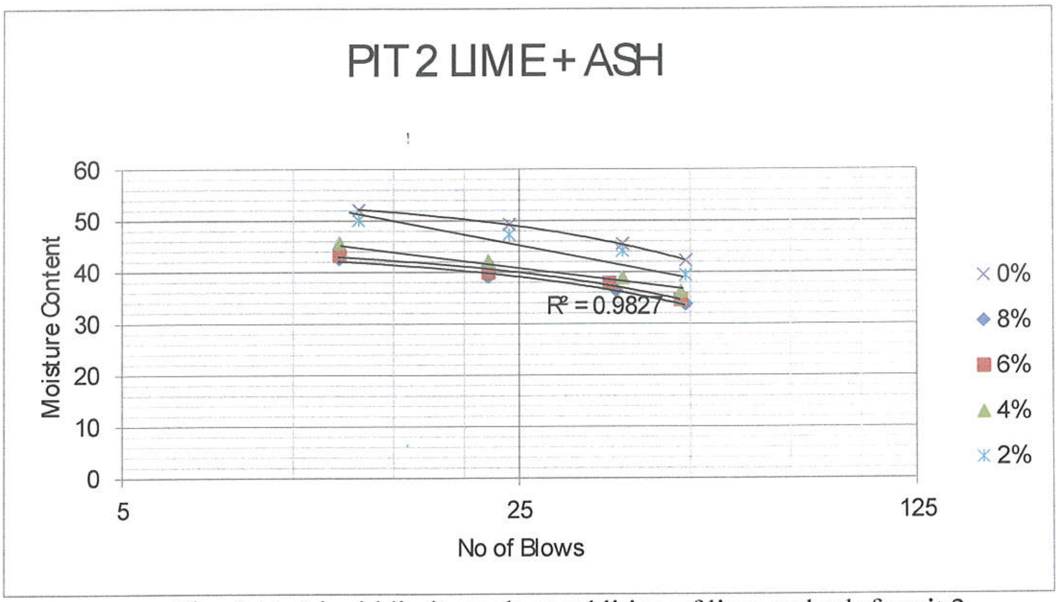

Fig. 4.11: Liquid limit result on addition of lime and ash for pit 2.

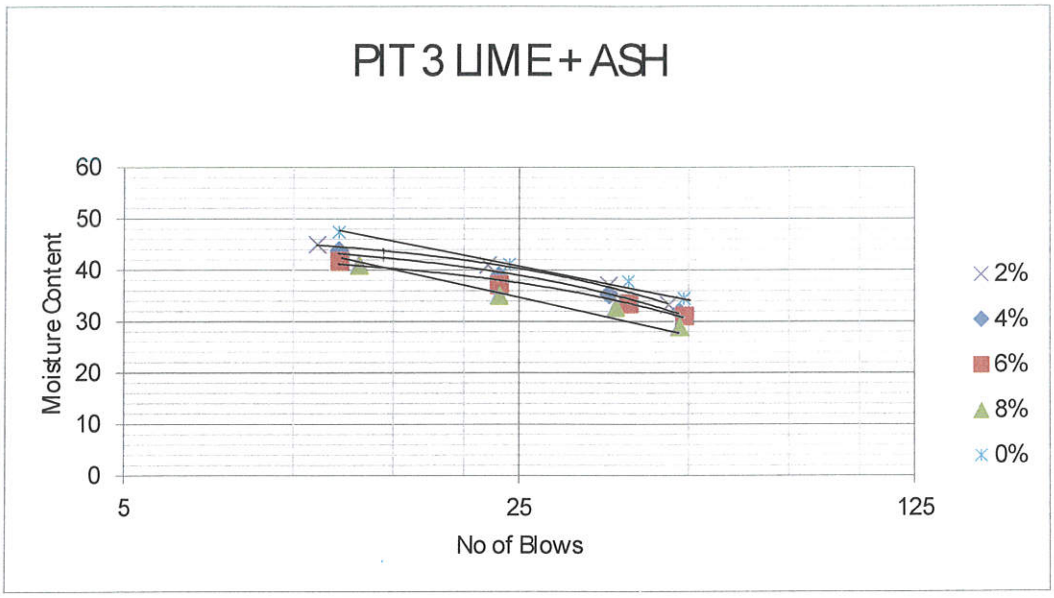

Fig. 4.12: Liquid limit result on addition of lime and ash for pit 3.

Amuu and Akinyele (2005), From the table 2.0 showing the relationship between liquid limit and plasticity index of soils, when liquid limit is less than 50% and plasticity index is less than 25% the soil has a low potential swell.

The initial liquid limit for the soils are less than 50% and plasticity index less than 25%. On addition of lime, the liquid limit values were still less than 50% and plasticity index less than 25% but lesser than their initial values before addition. On addition of ash, the liquid limit was still less than 50% and plasticity index less than 25% except for 2% of pit 2 and 2% and 4% of pit 3 that had values above 25%. On addition of lime and ash, the liquid limit was still less than 50% and plasticity index less than 25%.

Table 4.0: Summary table for atterberg limit tests result.

The summary for all results of the atterberg limits test are given in the table below:

PIT	Per.	LIME			ASH			LIME + ASH		
		L.L	P.L	P.I	L.L	P.L	P.I	L.L	P.L	P.I
PIT 1	0%	43.80	27.85	15.95	43.80	27.85	15.95	43.80	27.85	15.95
	2%	44.10	27.60	16.50	40.00	18.05	21.95	41.00	16.50	24.50
	4%	43.10	28.30	14.80	42.10	20.00	22.10	37.10	15.50	21.60
	6%	38.70	28.60	10.10	43.80	19.80	24.00	36.00	14.60	21.40
	8%	35.00	28.90	6.10	39.40	20.00	19.40	33.90	13.90	20.00
PIT 2	0%	47.20	25.10	22.10	47.20	25.10	22.10	47.20	25.10	22.10
	2%	46.20	31.95	14.25	45.60	17.90	27.70	43.00	20.10	22.90
	4%	44.50	35.70	8.80	44.20	20.20	24.00	43.00	20.20	22.80
	6%	42.00	29.10	12.90	41.70	18.95	22.75	38.00	20.70	17.30
	8%	40.30	30.20	10.10	40.00	19.55	20.45	37.00	18.50	18.50
PIT 3	0%	41.00	16.05	24.95	41.00	16.05	24.95	41.00	16.05	24.95
	2%	40.00	19.70	20.30	38.00	12.45	25.55	38.00	18.70	19.30
	4%	38.80	18.65	20.15	39.80	13.15	26.65	37.40	17.90	19.50
	6%	35.20	19.30	15.90	37.30	18.50	18.80	36.00	17.60	18.40
	8%	38.00	35.10	2.90	36.10	15.20	20.90	34.20	16.30	17.90

4.6 Standard Proctor Compaction

The initial values of OMC and MDD are 15.0% and 1.69kg/m^3, 19.3% and 1.60kg/m^3, 12.1% and 1.74kg/m^3 for pit 1, pit 2 and pit 3 respectively.

On addition of lime the OMC reduced from 15.1% at 2% to 14.7% at 4% and increased to 18.0% at 6% then at 8%, it reduced to 14.9% while the MDD reduced from 1.88kg/m3 at 2% to 1.79kg/m3 at 4% and increased to 1.95kg/m3 at 6% and further to 2.00kgm3 at 8% for pit 1. Figures 4.13 below shows the graphical illustration on addition of lime.

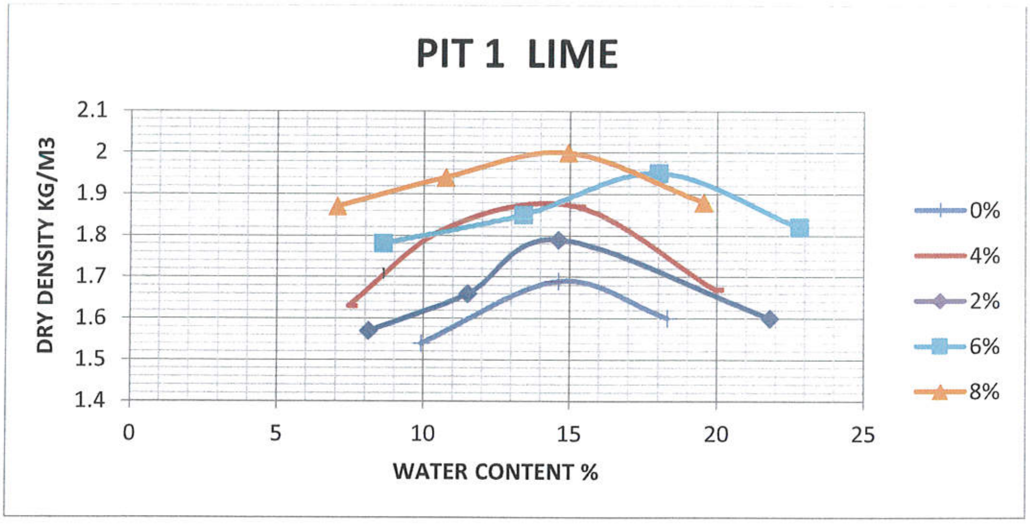

Fig. 4.13: OMC and MDD values on addition of lime for pit 1.

The OMC decreased from 22.1% at 2% to 15.4% at 4% and increased to 16.4% at 6% and further to 16.7% at 8% while MDD increased from 1.64kg/m3 at 2% to 1.97kg/m3 at 8% for pit 2 as shown in figure 4.14 below.

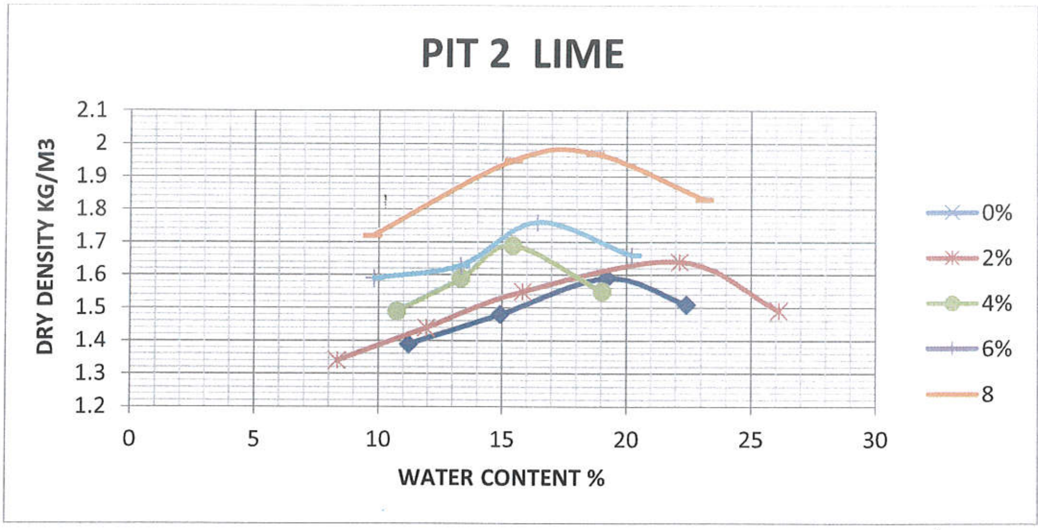

Fig. 4.14: OMC and MDD values on addition of lime for pit 2.

The OMC increased from 13.8% at 2% to 17.2 % at 6% and a reduction at 8% while MDD increased from 1.74kg/m3 at 2% to 1.97kg/m3 at 8% as shown in figure 4.15 below:

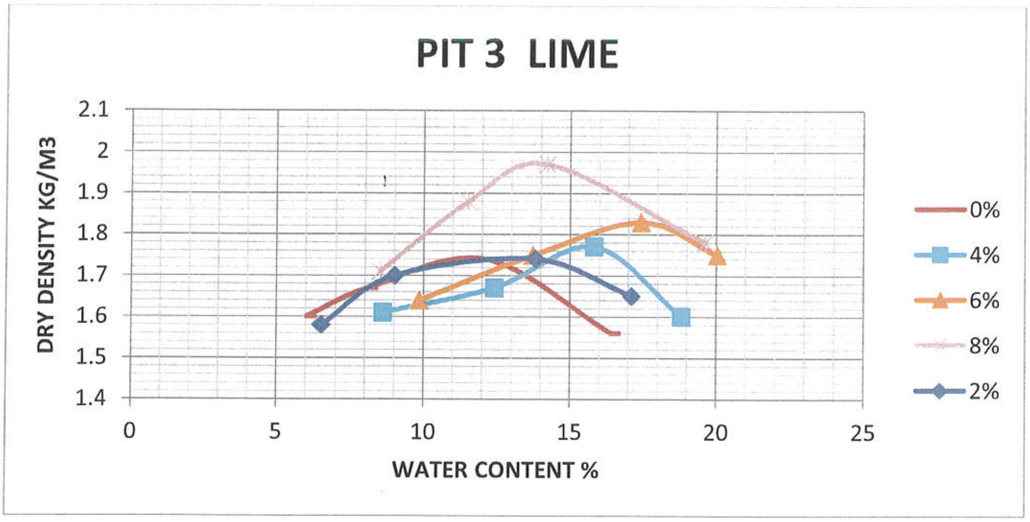

Fig. 4.15: OMC and MDD values on addition of lime for pit 3.

On addition of ash, the OMC increased from 13.4% at 2% to 17.0% at 4% and reduced from 14.3% to 13.5% at 6% to 8% respectively while the MDD reduced from 1.65kg/m3 at 2% to 1.72kg/m3 at 6% and reduced to 1.68kg/m3 at 8% for pit 1 as shown in figure 4.16 below:

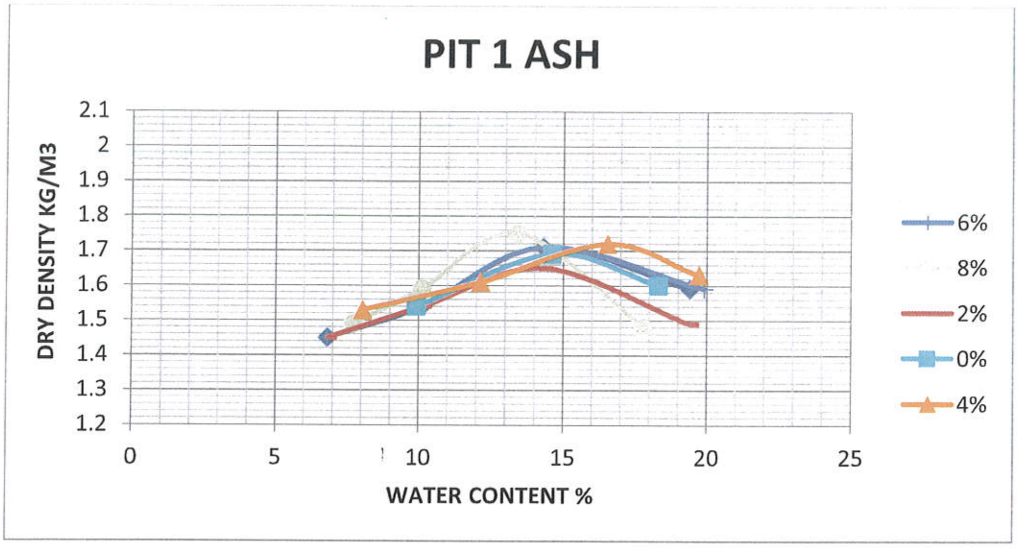

Fig. 4.16: OMC and MDD values on addition of ash for pit 1.

The OMC increased from 16.7% at 2% to 20.6% at 4% and reduced to 17.3% at 6% to further increase to 21.0% at 8% while the MDD increased from 1.59kg/m3 at 2% to 1.70kg/m3 at 8% for pit 2 as shown in figure 4.17 below:

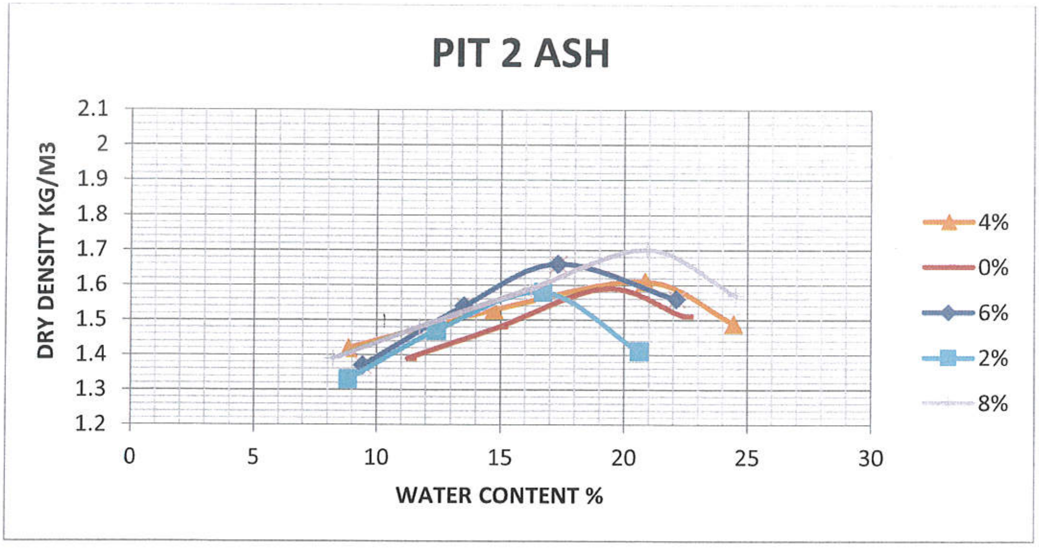

Fig. 4.17: OMC and MDD values on addition of ash for pit 2.

The OMC increased from 15.9% at 2% to 19.0% at 4% and reduced to 15.4% at 6% till 14.7% at 8% while the MDD increased from 1.68kg/m3 at 2% to 1.88kg/m3 at 8% for pit 3 as shown in figure 4.18 below.

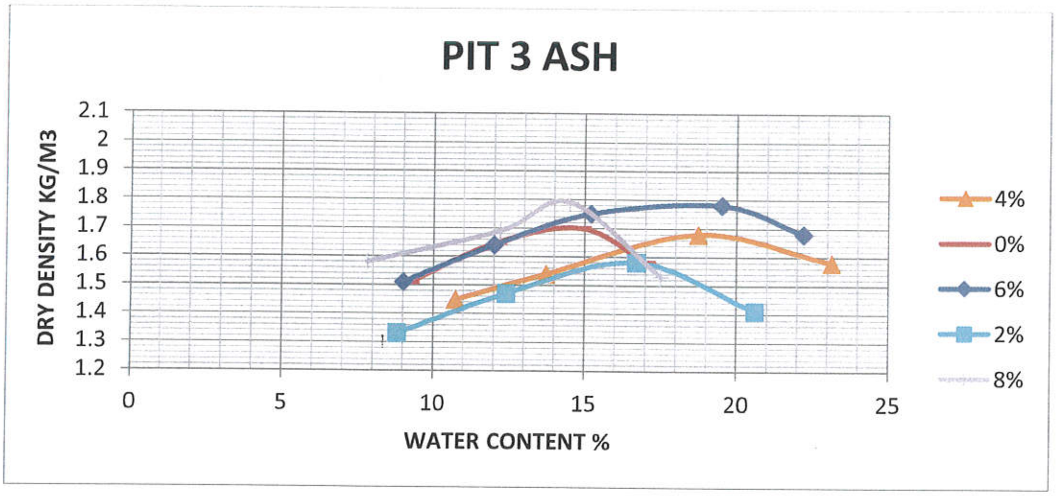

Fig. 4.18: OMC and MDD values on addition of ash for pit 3.

On addition of lime and ash, the OMC increased from 17.5% at 2% to 18.0% at 4% and reduced to 16.0% at 6% and then to 14.5% at 8% while the MDD increased from 1.70kg/m3 at 2% to 1.80kg/m3 at 6% and reduced to 1.76kg/m3 at 8% for pit 1 as shown in figure 4.19 below:

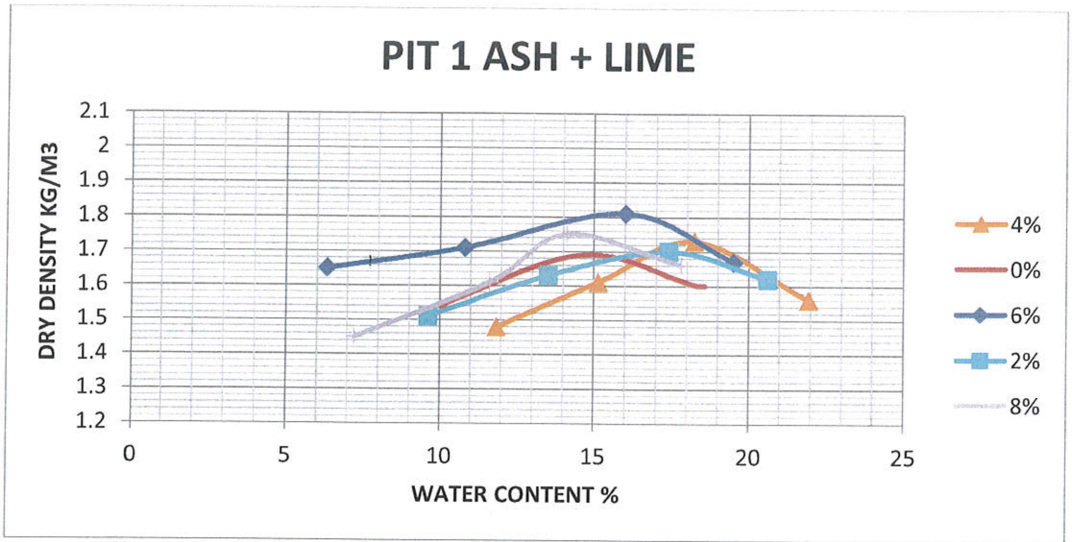

Fig. 4.19: OMC and MDD values on addition of lime and ash for pit 1.

The OMC reduced from 19.5% at 2% to 18.0% at 4% and increased to 19.7% at 6% and to 21.0% at 8% while the MDD increased from 1.60kg/m3 at 2% to 1.78kg/m3 at 8% for pit 2 as shown in figure 4.20 below:

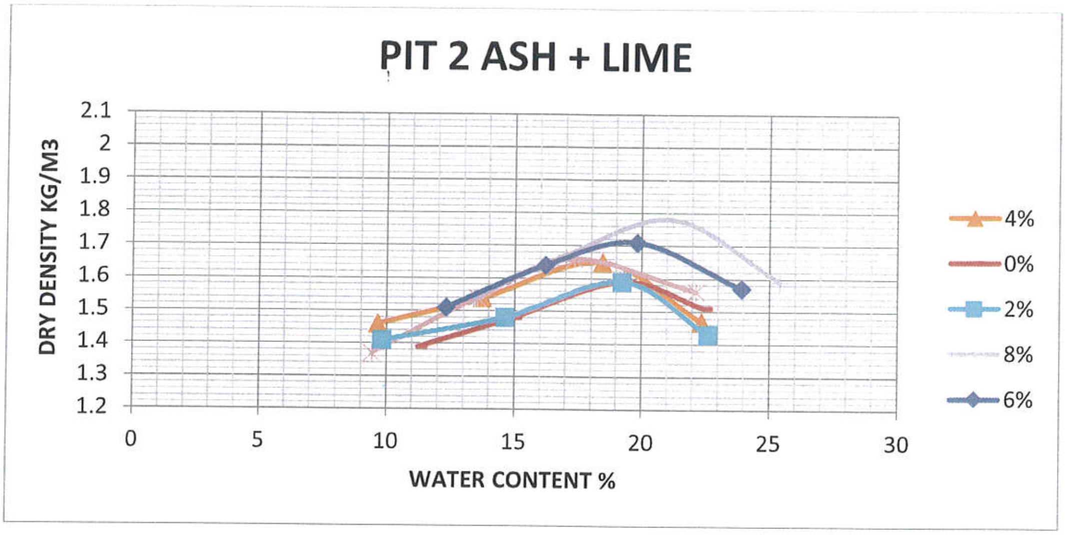

Fig. 4.20: OMC and MDD values on addition of lime and ash for pit 2.

The OMC reduced from 20.0% at 2% to 17.5% at 4% and increased to 18.0% at 6% and then to 19.8% at 8% while the MDD increased from 1.68kg/m3 at 2% to 1.86kg/m3 at 8% for pit 3 as shown in figure 4.21 below:

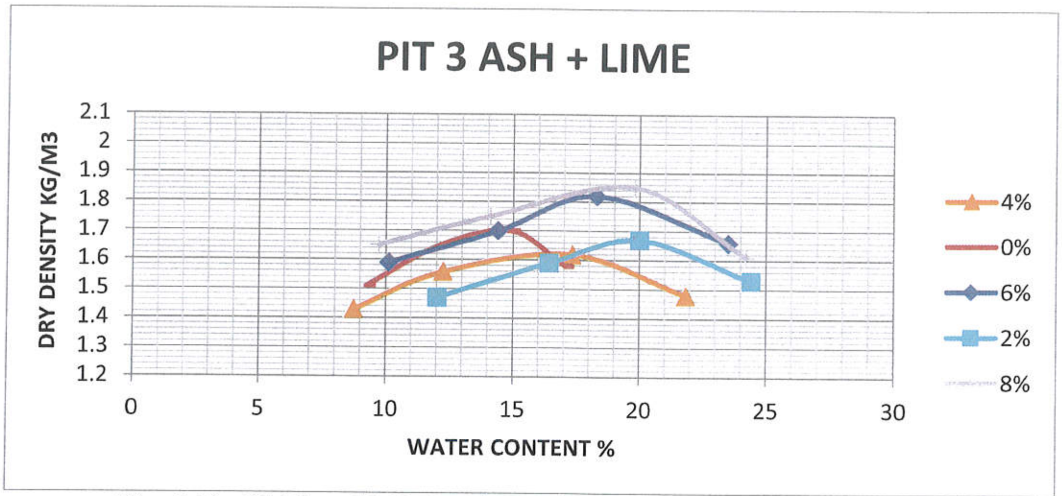

Fig. 4.21: OMC and MDD values on addition of lime and ash for pit 3.

Table 4.1: Summary table for standard proctor compaction test result.

The summary table for the results of the standard proctor compaction test is represented below:

PIT	Percentage	LIME		ASH		LIME + ASH	
		OMC	MDD	OMC	MDD	OMC	MDD
PIT 1	0%	15.0	1.69	15.0	1.69	15.0	1.69
	2%	15.1	1.88	13.4	1.65	17.5	1.70
	4%	14.7	1.79	17.0	1.72	18.0	1.73
	6%	18.0	1.95	14.3	1.72	16.0	1.80
	8%	14.9	2.00	13.5	1.68	14.5	1.76
PIT 2	0%	19.3	1.60	19.3	1.60	19.3	1.69
	2%	22.1	1.64	16.7	1.59	19.5	1.60
	4%	15.4	1.69	20.6	1.62	18.0	1.61
	6%	16.4	1.76	17.3	1.67	19.7	1.71
	8%	16.7	1.97	21.0	1.70	21.0	1.78
PIT 3	0%	12.1	1.74	12.1	1.74	12.1	1.74
	2%	13.8	1.74	15.9	1.68	20.0	1.68
	4%	15.8	1.77	19.0	1.68	17.5	1.63
	6%	17.2	1.83	15.4	1.76	18.0	1.82
	8%	14.3	1.97	14.7	1.88	19.8	1.86

According to Amu *et al.*,(2011), increase in MDD values indicates improvement in Soil properties While increase in OMC values implies that more water is needed to compact the soil (Joel and Joseph, 2015).

4.7 Unconfined Compressive Strength Test

The initial values for stress are 239.20KN/m^2, 259.30KN/m^2, 410.50KN/m^2 for pit 1, pit 2 and pit 3 respectively.

The highest value of stress 152.30KN/m2 was recorded at 6% addition of lime for pit 1 which was a reduction from initial value. The highest value of stress 413.10KN/m2 was obtained at 6% addition of lime for pit 2 which was a better increase from the initial value. The highest value of stress 174.00KN/m2 was recorded at 8% addition of lime for pit 3 which was a reduction from initial value.

The highest value of stress 500.30KN/m2 was recorded at 6% addition of ash for pit 1 which was increase in initial value. The highest value of stress 410.50KN/m2 was obtained at 8% addition of lime for pit 2 which was a better increase from the initial value. The highest value of stress 350.30KN/m2 was recorded at 8% addition of lime for pit 3 which was an increase in initial value.

The highest value of stress 324.10KN/m2 was recorded at 6% addition of lime and ash for pit 1 which was an increase in initial value. The highest value of stress 324.10KN/m2 was obtained at 6% addition of lime and ash for pit 2 which was a better increase in the initial value. The highest value of stress 174.00KN/m2 was recorded at 2% addition of lime and ash for pit 3 which was a reduction from initial value.

4.8 California Bearing Ratio Test

According to ASTM, CBR of 3 equates to tilled farmland, a CBR of 4.75 equates to turf or moist clay while moist sand may have a CBR of 10. High quality crushed rock has a CBR over 80. The standard material for this test is crushed California limestone which has a value of 100, meaning that it is not unusual to see CBR values of over 100 in well-compacted areas. The harder the material, the higher the CBR value. A CBR value of 2% is usually found for clay, high-quality sub-base will have CBR values between 80% and 100% and some sands may have values around 10%.

Table 4.2: Summary table for values of CBR

The summary table for the values of the unsoaked CBR:

PIT		0%	2%	4%	6%	8%
PIT 1 LIME	**250**	66%	71%	51%	70%	71%
	500	71%	71%	74%	76%	82%
PIT 2 LIME	**250**	33%	39%	41%	42%	36%
	500	43%	43%	47%	51%	50%
PIT 3 LIME	**250**	33%	40%	44%	51%	54%
	500	50%	49%	55%	59%	62%
PIT 1 ASH	**250**	66%	62%	58%	62%	66%
	500	71%	70%	70%	74%	72%
PIT 2 ASH	**250**	33%	26%	32%	41%	43%
	500	43%	36%	43%	45%	48%
PIT 3 ASH	**250**	33%	27%	30%	39%	45%
	500	50%	44%	47%	49%	52%
PIT 1 LIME + ASH	**250**	66%	67%	69%	70%	71%
	500	71%	71%	72%	75%	73%
PIT 2 LIME + ASH	**250**	33%	28%	34%	40%	35%
	500	43%	41%	44%	48%	48%
PIT 3 LIME + ASH	**250**	33%	35%	42%	48%	48%
	500	50%	45%	48%	53%	55%

From the foregoing, the three pits can be described as consisting of low quality crushed rock which makes pit 1 best, pit 3 better and pit 2 good.

On application of lime: for pit 1, there was an improvement to 82%. For pit 2, there was an increment from 43% to 51% at 6% and a reduction to 50% at 8%. For pit 3, there was an increment from 50% to 62% as shown in figures 4.22 to 4.24 below.

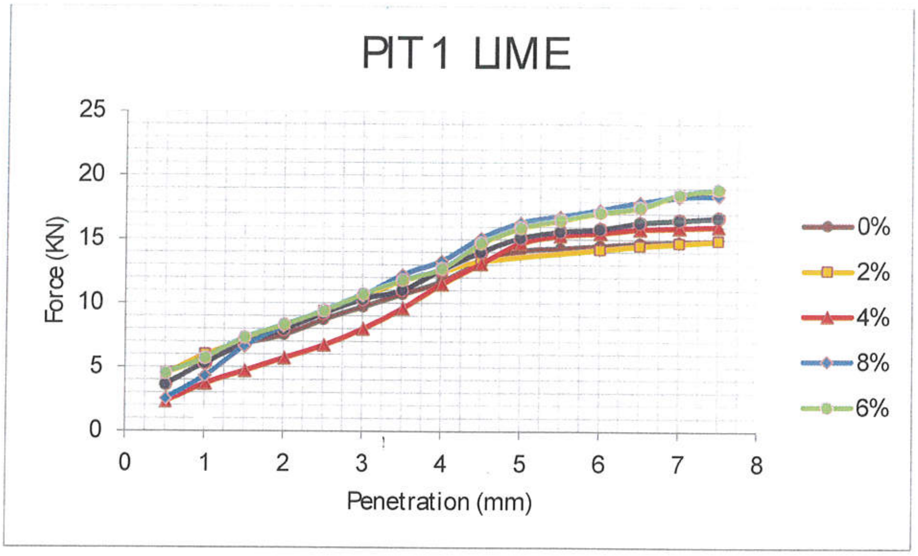

Fig. 4.22: Effect of lime on the CBR values of soil sample in pit 1.

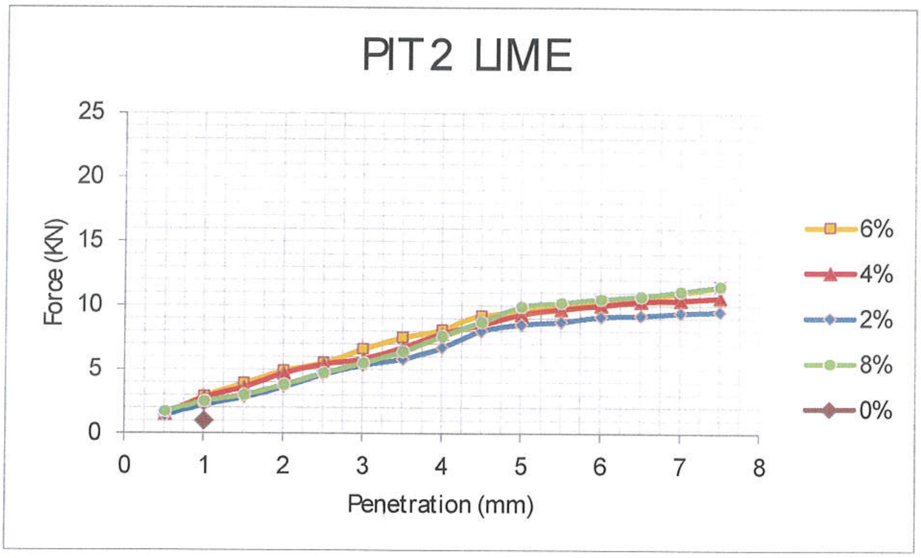

Fig. 4.23: Effect of lime on the CBR values of soil sample in pit 2.

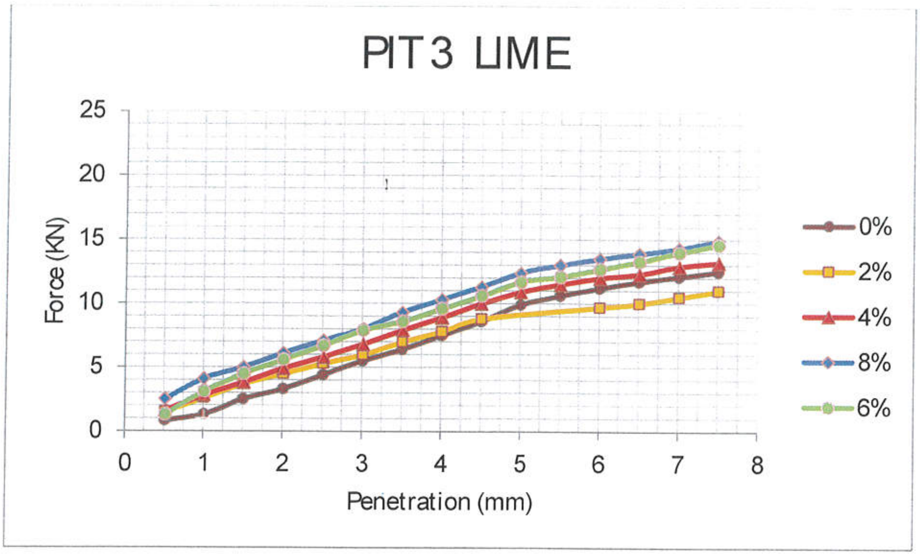

Fig. 4.24: Effect of lime on the CBR values of soil sample in pit 3.

On application of ash: For pit 1, there was an upward increment from 70% to 74% at 6% and dropped to 72% at 8%. For pit 2, there was an increment from 36% to 48%. For pit 3, there was an increment from 44% to 52% as shown in figures 4.25 to 4.27 below.

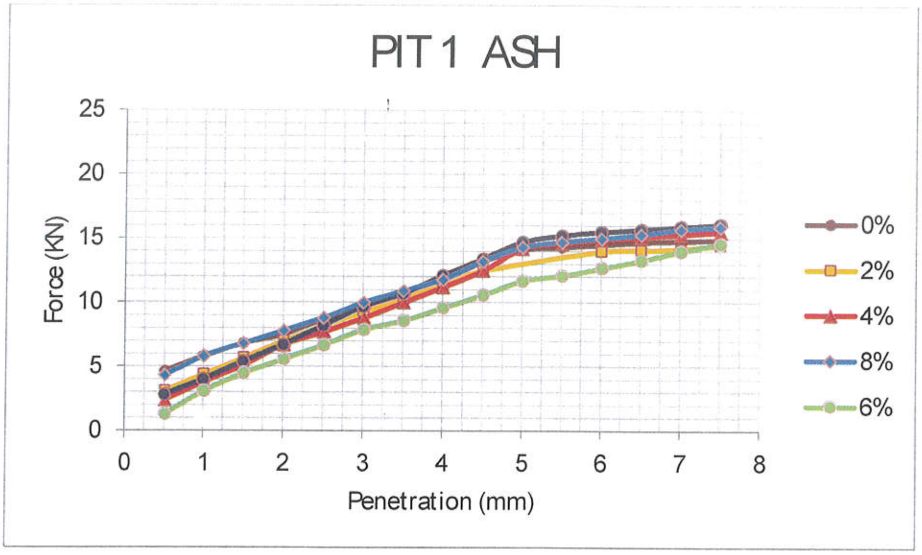

Fig. 4.25: Effect of ash on the CBR values of soil sample in pit 1.

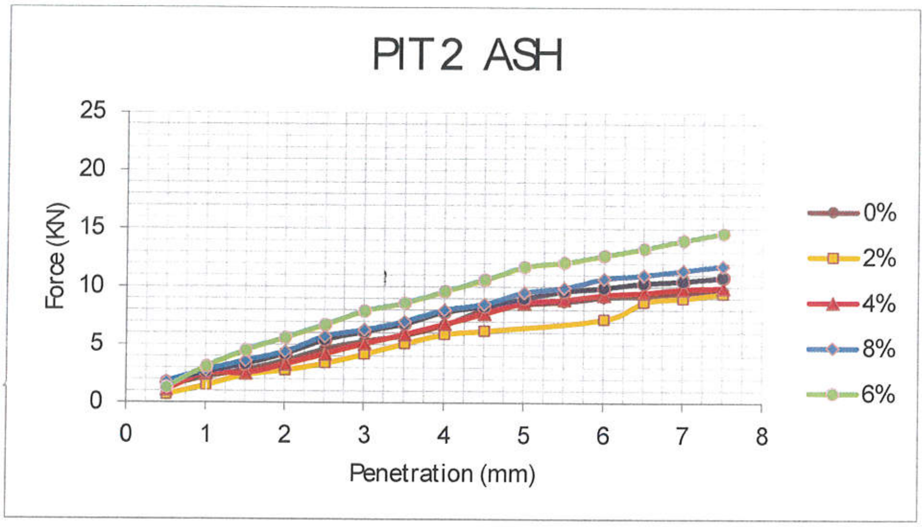

Fig. 4.26: Effect of ash on the CBR values of soil sample in pit 2.

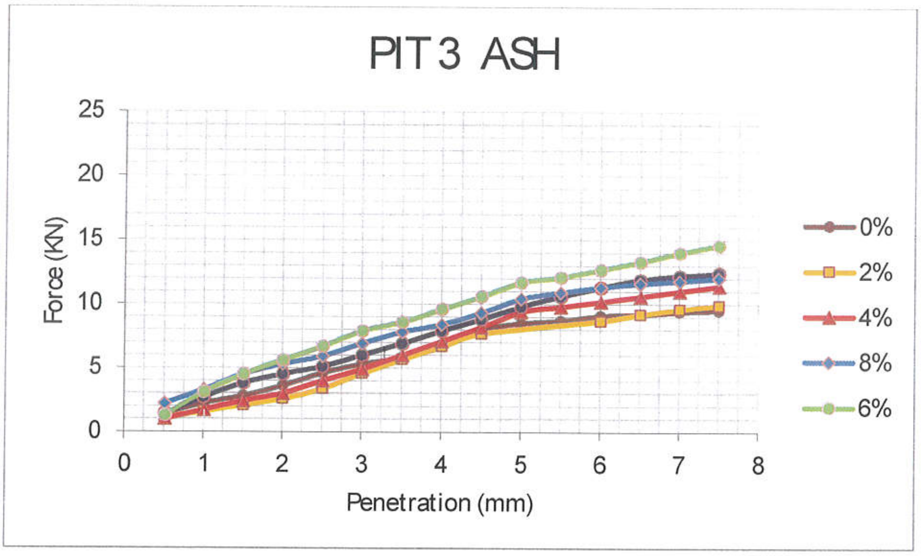

Fig. 4.27: Effect of ash on the CBR values of soil sample in pit 3.

On application of lime and ash: For pit 1, there was an increment from 71% to 75% at 6% and a decrease to 73% at 8%. For pit 2, there was an increment from 41% to 48%. For pit 3, there was an increment from 45% to 55% as shown in figures 4.28 to 4.30 below.

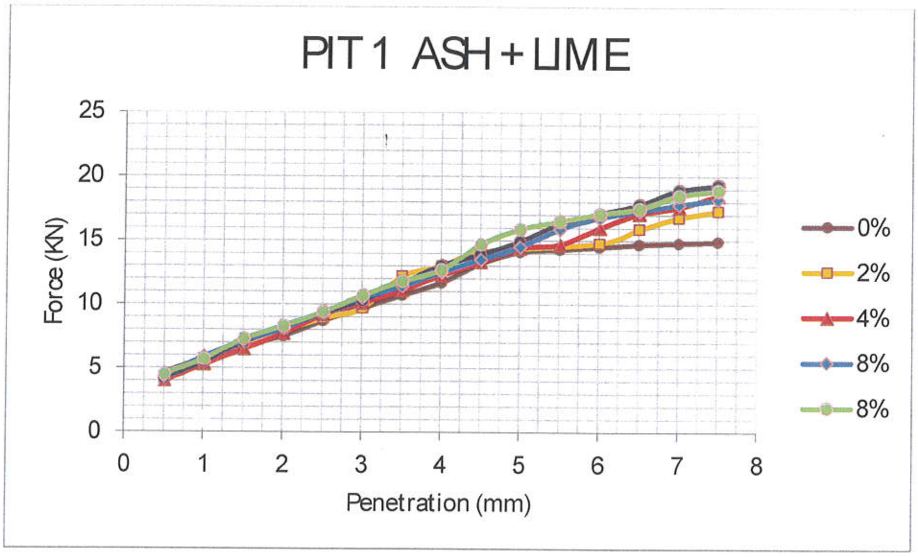

Fig. 4.28: Effect of ash and lime on the CBR values of soil sample in pit 1.

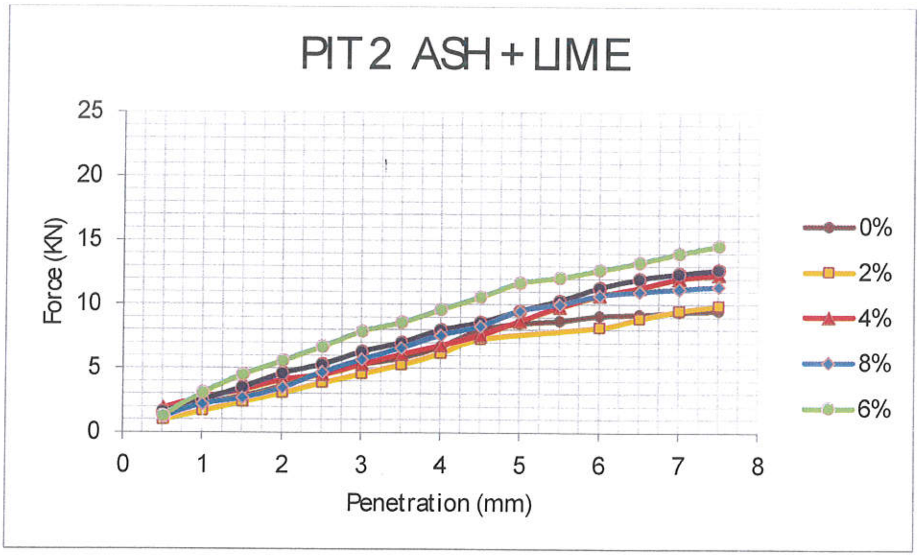

Fig. 4.29: Effect of ash and lime on the CBR values of soil sample in pit 2.

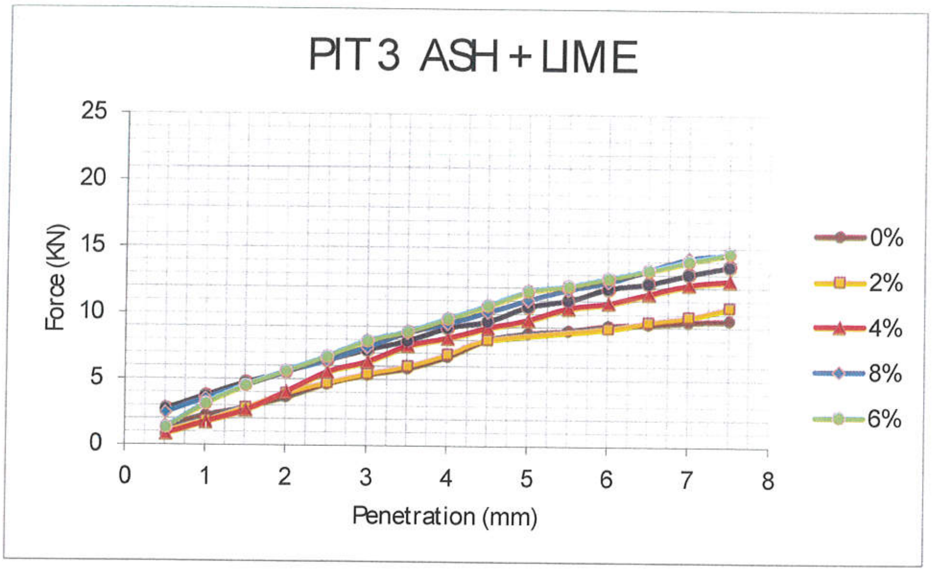

Fig. 4.30: Effect of ash and lime on the CBR values of soil sample in pit 3.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The natural moisture content of the soils are respectively 9.55%, 6.80% and 10.6% indicating that pit 3 has the highest moisture content and pit 2 has the lowest moisture content of the three soils.

The soil particles contain clay minerals for pit 1 and pit 2 while pit 3 soil contains certain heavy substances.

The soils are classified as silts and clays of low compressibility and in the A-7 group which is poor for construction. Hence, the need for stabilization.

The soil particles initially had a low potential swell. The addition of lime maintained the property to a greater extent, the addition of ash increased the plasticity indexes of the pit 2 and pit 3 at some percentages of addition while the addition of ash and lime left the soil at a low potential to swell.

All values of OMC and MDD range between OMC of 12.0% to 22.0% and MDD of 1.60kg/m^3 to 1.97kg/m^3. The addition of lime gave the best result, addition of lime and ash gave better result and addition of ash gave a good result. In other words, increase in MDD values indicates improvement in Soil properties while increase in OMC values implies that more water is needed to compact the soil.

For the unconfined compressive strength test, pit 1 had its highest values at 6% as each additives were added. The addition of ash only and lime-ash had greater effect than addition of lime only.

The soils contain low quality crushed rocks in their initial states. All the three additives had effect on the soils but lime increased the quality of the soil up to 6% addition in pit 2. Also, ash increased the quality of soil up to 6% addition in pit 1. Lime had good effect on pit 1, subsequent addition above 8% to pit 3 could improve its quality. Subsequent addition of ash to pit 2 and 3 could also improve the soil qualities. Ash and lime improved the soils quality more. The results therefore showed that the strength of the samples in terms of their load bearing capacity greatly increased with lime and OPFA.

5.2 Recommendation and Limitation

The following are recommended:

Oil Palm Frond Ash and Lime is a good additive combination that can be substituted with other substances used to improve soil characteristics to enhance their engineering performance.

Pit 1 could serve as a borrow pit after addition of lime to the soil.

Lime and Oil Palm Frond Ash should be added to soils above 8%, that is, at 10% and above as there was a progressive increase from 0% to 8% in the engineering tests.

The research was limited to 8% addition of additives that is, the tests were not conducted beyond 8% of addition of the two additives.

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APPENDIX

APPENDIX A

APPENDIX A1 - SEIVE ANALYSIS

S/N	PIT 1 Weight Retained	Percentage Retained	Percentage Passing
9.50	43.1	8.6	91.4
4.75	91.6	18.3	73.1
2.36	38.0	7.6	65.5
1.18	51.6	10.3	55.2
600	22.1	4.4	50.8
300	21.6	4.3	46.5
150	16.4	3.3	43.2
75	11.6	2.3	40.9

S/N	PIT 2 Weight Retained	Percentage Retained	Percentage Passing
9.50	0.0	0.0	100.0
4.75	4.4	0.9	99.1
2.36	18.4	3.4	95.7
1.18	41.2	8.2	87.5
600	59.3	11.9	75.6
300	62.4	12.5	63.1
150	48.7	9.7	53.4
75	38.8	7.8	45.6

S/N	PIT 3 Weight Retained	Percentage Retained	Percentage Passing
9.50	15.2	3.0	97.0
4.75	9.1	1.8	95.2
2.36	13.4	2.3	92.9
1.18	26.0	5.2	87.7
600	57.2	11.4	76.3
300	114.8	23.0	55.3
150	79.7	15.9	37.4
75	20.8	4.2	33.2

APPENDIX A2 – NATURAL MOISTURE CONTENT TEST RESULT

	PIT 3		PIT 2		PIT 1	
Weight of can + wet soil	70.2	79.5	66.6	65.9	76.9	72.8
Weight of can + dry soil	65.7	74.4	63.5	63.1	71.8	68.1
Weight of empty can	19.3	20.0	19.9	19.7	26.8	19.9
Weight of dry soil	46.4	54.4	43.6	43.4	45.0	48.2
Weight of water	4.5	5.1	3.1	2.8	5.1	4.7
Moisture content	9.7	9.4	7.1	6.5	11.3	9.8
Natural Moisture Content	**9.55%**		**6.8%**		**10.6%**	

APPENDIX A3 - SPECIFIC GRAVITY TEST RESULT

	PIT 1		PIT 2		PIT 3	
Weight of empty bottle(W1)	25.6	25.6	26.4	26.5	25.9	25.8
Weight of bottle + soil(W2)	54.3	55.2	54.3	52.9	53.1	52.8
Wt of bottle + soil + distilled water(W3)	95.8	96.1	96.1	97.2	94.7	95.0
Wt of bottle filled with distilled water(W4)	78.3	78.5	79.8	79.9	75.0	75.3
Specific gravity	2.56	2.47	2.41	2.90	3.63	3.70
Average specific gravity	2.52		2.67		3.67	

APPENDIX A4 - ATTERBERG LIMIT TESTS RESULTS

	PIT 1 (0% LIME) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	37	23	12		
Can No	B1	W1	VR	N5	TP	TE
Wt of Empty Can	26.9	26.7	21.5	26.7	7.7	14.2
Can + Wet Soil	54.7	56.6	48.8	53.7	23.1	30.0
Can + Dry Soil	46.2	47.1	39.8	44.7	19.7	26.6
Wt of Water	8.5	9.5	9.0	9.0	3.4	3.4
Wt of Dry Soil	19.3	20.4	18.3	18.0	12.0	12.4
Moisture Content	44.0	46.6	49.2	50.0	28.3	27.4
					P.L.= 27.85	

PIT 1 (2% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	39	27	13		
Can No	G1	G2	G3	G4	G5	G6
Wt of Empty Can	26.5	19.9	19.8	20.8	19.8	16.9
Can + Wet Soil	45.0	41.3	42.8	46.0	35.1	30.2
Can + Dry Soil	39.3	34.4	35.0	37.0	32.6	27.9
Wt of Water	5.7	6.9	7.8	9.0	2.5	2.3
Wt of Dry Soil	12.8	14.5	15.2	16.5	12.8	11.0
Moisture Content	44.5	47.6	51.3	54.5	19.5	20.9
					P.L. = 27.60	

PIT 1 (4% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	37	22	12		
Can No	E1	E2	E3	E4	E5	E6
Wt of Empty Can	19.8	20.1	17.9	18.7	19.8	19.9
Can + Wet Soil	40.5	43.3	45.2	48.0	25.2	34.3
Can + Dry Soil	36.4	36.2	36.4	38.2	22.8	32.1
Wt of Water	5.9	7.1	8.8	9.8	2.4	2.2
Wt of Dry Soil	14.8	16.1	18.5	19.5	13.0	12.2
Moisture Content	40.0	44.1	47.6	50.3	18.5	18.0
					P.L. = 28.30	

PIT 1 (6% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	37	23	13		
Can No	T4	UN	B4	N	TE	AD
Wt of Empty Can	19.6	26.9	20.0	10.1	14.2	9.1
					35.4	32.9
Can + Wet Soil	44.5	52.4	49.6	49.4	30.7	27.6
Can + Dry Soil	37.0	44.1	39.8	35.7	4.7	5.3
Wt of Water	7.5	8.3	9.8	13.7	16.5	18.5
Wt of Dry Soil	17.4	17.2	19.8	25.6	28.5	28.7
Moisture Content	43.1	48.3	49.5	53.5	P.L.= 28.60	

PIT 1 (8% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	46	36	21	12		
Can No	UV	D2	W3	PZ	J4	CM3
Wt of Empty Can	26.9	19.8	20.0	20.1	18.6	14.9
Can + Wet Soil	47.2	43.0	47.8	47.5	35.9	29.7
Can + Dry Soil	40.8	35.4	38.6	38.1	32.0	26.4
Wt of Water	6.4	7.6	9.2	9.4	3.9	3.3
Wt of Dry Soil	13.9	15.6	18.6	18.0	13.4	11.5
Moisture Content	46.0	48.7	49.5	52.5	29.1	28.7
					P.L.= 28.90	

PIT 2 (0% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	38	24	13		
Can No	S10	S11	S12	S13	S14	S15
Wt of Empty Can	15.6	16.1	17.8	18.9	15.8	17.5
Can + Wet Soil	33.1	37.2	41.4	44.9	30.9	31.1
Can + Dry Soil	27.9	30.6	33.6	36.0	27.9	28.3
Wt of Water	5.2	6.6	7.8	8.9	3.0	2.8
Wt of Dry Soil	12.3	14.5	15.8	17.1	12.1	10.8
Moisture Content	42.3	45.5	49.4	52.0	24.8	25.9
					P. L. = 25.10	

PIT 2 (2% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	45	30	23	15		
Can No	A1	GG	II	A	GRPB	J3
Wt of Empty Can	13.8	15.8	27.1	11.7	18.4	19.9
Can + Wet Soil	35.2	40.6	54.8	41.7	33.8	36.7
Can + Dry Soil	29.4	33.4	46.3	31.9	30.1	32.6
Wt of Water	5.8	7.2	8.5	9.8	7.3	4.1
Wt of Dry Soil	15.6	17.6	19.2	20.2	11.7	12.7
Moisture Content	37.2	40.9	44.3	48.5	31.6	32.3
					P.L.= 31.95	

PIT 2 (4% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	44	33	20	12		
Can No	ZZ	G1	C4	5A	B5	UV
Wt of Empty Can	26.8	10.0	19.8	20.1	28.5	26.9
Can + Wet Soil	50.2	32.8	45.2	48.4	28.8	43.1
Can + Dry Soil	43.3	25.6	37.2	39.3	23.5	38.8
Wt of Water	6.9	7.2	8.0	9.1	5.3	4.3
Wt of Dry Soil	16.5	15.6	17.4	19.2	15.0	11.9
Moisture Content	41.8	46.2	46.0	47.4	35.3	36.1
					P.L.= 35.70	

PIT 2 (6% LIME)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	45	32	22	13		
Can No	T2	OO	UI	4	AD	G5
Wt of Empty Can	19.5	20.0	19.9	14.3	9.1	13.5
Can + Wet Soil	41.4	46.2	49.8	42.8	24.7	26.8
Can + Dry Soil	34.7	37.9	40.3	33.8	20.5	23.2
Wt of Water	6.7	8.3	9.5	9.0	4.2	3.6
Wt of Dry Soil	15.2	17.9	20.4	19.5	11.4	9.7
Moisture Content	44.1	46.4	46.8	46.2	36.8	37.1
					P.L.= 29.10	

PIT 2 (8% LIME)	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	35	23	14		
Can No	V2	B3	YZ	S1	AZ	AFZ
Wt of Empty Can	19.9	20.2	21.0	20.2	19.9	19.9
Can + Wet Soil	43.4	40.4	48.0	48.4	39.5	42.2
Can + Dry Soil	36.2	34.3	39.4	39.4	34.9	37.1
Wt of Water	7.2	6.1	8.6	9.0	4.6	5.1
Wt of Dry Soil	16.3	14.1	18.4	19.2	15.0	17.2
Moisture Content	44.2	43.3	46.7	46.9	30.7	29.7
					P. L. = 30.20	

PIT 3 (0% LIME)	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	39	24	12		
Can No	H10	H11	H12	H13	J4	PZ
Wt of Empty Can	10.1	12.9	19.9	19.9	18.5	20.1
Can + Wet Soil	27.3	31.9	41.4	44.2	39.0	35.0
Can + Dry Soil	22.9	26.6	34.9	36.4	36.3	32.9
Wt of Water	4.4	5.3	6.5	7.8	2.7	2.1
Wt of Dry Soil	12.8	13.7	15.0	16.5	17.8	12.8
Moisture Content	34.4	38.7	43.3	47.3	15.7	16.4
					P.L.= 16.05	

PIT 3 (2% LIME)	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	50	37	23	14		
Can No	5	Ur	W3	P1	DB	CNB
Wt of Empty Can	26.9	26.7	20.0	19.8	8.7	14.9
Can + Wet Soil	53.6	49.1	52.4	54.9	22.9	32.2
Can + Dry Soil	46.7	43.2	43.5	44.9	20.6	29.3
Wt of Water	6.9	5.9	8.9	10.0	2.3	2.9
Wt of Dry Soil	19.8	16.3	23.5	25.1	11.9	14.4
Moisture Content	34.9	36.2	37.9	39.8	19.3	20.1
					P.L.= 19.70	

	PIT 3 (4% LIME) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	38	24	13		
Can No	KE	P2	Z1	B4	UR	GRPP
Wt of Empty Can	23.0	20.2	20.1	19.8	26.9	18.4
Can + Wet Soil	46.9	48.2	51.4		39.1	35.5
Can + Dry Soil	37.0	45.8	47.6	43.2	37.2	32.8
Wt of Water	5.2	6.5	7.8	9.2	1.9	2.7
Wt of Dry Soil	17.3	19.1	20.7	23.2	10.3	14.4
Moisture Content	30.0	34.0	37.7	39.7	18.5	18.8
					P.L.= 18.65	

	PIT 3 (6% LIME) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	35	23	12		
Can No	D3	K	W2	N1	B5	G5
Wt of Empty Can	19.7	26.7	26.9	20.0	8.5	13.4
Can + Wet Soil	43.2	52.3	55.4	52.4	23.6	35.3
Can + Dry Soil	37.0	45.8	47.6	43.2	21.2	31.7
Wt of Water	5.2	6.5	7.8	9.2	2.4	3.6
Wt of Dry Soil	17.3	19.1	20.7	23.2	12.7	18.3
Moisture Content	30.0	34.0	37.7	39.7	18.9	19.7
					P.L.= 19.30	

	PIT 3 (8% LIME) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	36	23	12		
Can No	UN	N	W5	IO	WH	D2
Wt of Empty Can	26.8	10.0	27.4	26.6	19.9	9.8
Can + Wet Soil	54.5	41.0	55.5	63.8	39.6	23.6
Can + Dry Soil	46.9	32.3	47.5	52.8	34.5	20.0
Wt of Water	6.6	7.9	8.0	11.0	5.1	3.6
Wt of Dry Soil	20.1	22.3	20.1	26.2	14.6	10.2
Moisture Content	32.8	35.4	39.8	42.0	34.9	35.3
					P.L.= 35.10	

	PIT 1 (2% ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	38	24	13		
Can No	IO	T1	J2	C4	A	B4
Wt of Empty Can	26.7	26.7	19.6	19.8	11.7	15.2
Can + Wet Soil	50.1	48.8	45.5	49.5	27.9	30.0
Can + Dry Soil	44.1	42.7	37.8	40.3	25.0	27.3
Wt of Water	6.0	6.1	7.7	9.2	2.9	2.7
Wt of Dry Soil	17.4	16.0	18.2	20.5	16.2	14.8
Moisture Content	34.5	38.1	42.3	44.9	17.9	18.2
					P.L.= 18.05	

PIT 1 (4% ASH)	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	36	23	14	DB	AD
Can No	YZ	UI	WI	B3	8.6	9.0
Wt of Empty Can	25.0	19.9	26.8	20.3	20.4	21.7
Can + Wet Soil	44.8	40.6	50.0	52.7	18.2	19.5
Can + Dry Soil	38.3	34.8	43.1	42.9	2.2	2.2
Wt of Water	5.0	5.8	6.9	9.8	11.8	12.7
Wt of Dry Soil	13.3	14.9	16.3	22.6	18.6	17.3
Moisture Content	37.6	38.9	42.3	43.4	P.L.= 20.00	

PIT 1 (6% ASH)	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	36	21	15	C5	COMP
Can No	J4	GG	Z5	G1	10.6	21.1
Wt of Empty Can	18.6	15.9	30.2	10.2	21.9	34.5
Can + Wet Soil	41.8	39.9	59.6	37.2	19.9	32.9
Can + Dry Soil	35.5	33.0	31.6	28.4	2.0	1.6
Wt of Water	6.3	6.9	7.7	8.8	11.3	13.4
Wt of Dry Soil	16.9	17.1	17.0	18.2	17.6	11.9
Moisture Content	37.3	40.4	45.3	48.4	P.L.= 19.80	

PIT 1 (8% ASH)	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	37	24	15	AJ1	AA
Can No	UN	SA	OO	V2	17.1	12.1
Wt of Empty Can	26.9	20.22	20.0	21.9	31.3	26.3
Can + Wet Soil	51.8	45.9	46.6	50.6	28.9	23.7
Can + Dry Soil	44.8	38.5	38.4	41.1	2.4	2.6
Wt of Water	7.0	7.4	8.2	9.5	13.6	14.2
Wt of Dry Soil	17.9	18.3	18.4	19.2	17.6	18.3
Moisture Content	39.1	40.4	44.6	49.5	P.L.= 20.00	

PIT 2 (2% ASH)	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	36	23	12	GRPB	3G
Can No	5I	D2	G5	PZ	18.4	10.7
Wt of Empty Can	20.3	19.9	13.4	22.0	31.5	22.7
Can + Wet Soil	36.8	44.6	40.5	52.3	29.1	20.6
Can + Dry Soil	32.1	37.1	31.9	42.0	2.4	2.1
Wt of Water	4.7	7.5	8.6	10.3	13.1	12.0
Wt of Dry Soil	11.8	17.2	18.5	20.0	18.3	17.5
Moisture Content	39.8	43.6	46.5	51.5	P.L.= 17.90	

PIT 2 (4% ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	44	35	20	11		
Can No	T4	N	B4	W3	QF	CK
Wt of Empty Can	19.6	10.2	16.7	23.9	15.5	16.9
Can + Wet Soil	46.3	39.1	44.7	55.4	31.5	33.7
Can + Dry Soil	38.6	30.3	35.7	44.1	28.8	30.9
Wt of Water	7.7	8.8	9.0	10.5	2.7	2.8
Wt of Dry Soil	19.0	20.1	19.0	20.2	13.3	14.0
Moisture Content	40.5	43.8	47.4	52.0	20.3	20.0
					P. L. = 20.20	

PIT 2 (6% ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	36	22	14		
Can No	UV	UR	B5	ZZ	TE	AO
Wt of Empty Can	26.8	27.0	26.4	26.8	14.2	8.4
Can + Wet Soil	48.7	52.2	32.2	56.1	24.3	22.0
Can + Dry Soil	42.2	44.6	24.9	46.8	22.4	14.4
Wt of Water	6.5	7.6	8.3	9.3	1.9	2.6
Wt of Dry Soil	15.4	17.6	18.5	20.0	10.1	13.6
Moisture Content	42.2	43.2	44.9	46.5	18.8	19.1
					P.L.= 18.95	

PIT 2 (8% ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	46	34	23	13		
Can No	II	K	4	X2	COM	T1T1
Wt of Empty Can	24.2	26.7	17.1	13.8	19.5	19.9
Can + Wet Soil	51.6	50.7	43.6	45.8	32.4	38.2
Can + Dry Soil	44.5	43.2	34.8	35.9	29.9	34.6
Wt of Water	7.1	7.5	8.8	9.9	2.5	3.6
Wt of Dry Soil	17.3	16.5	17.7	22.1	12.9	18.3
Moisture Content	41.0	45.5	49.7	44.8	19.4	19.7
					P.L.= 19.55	

PIT 3 (2% ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	37	22	14		
Can No	A3	M	A5	C5	BA	32
Wt of Empty Can	13.8	12.1	8.4	11.7	19.3	21.2
Can + Wet Soil	39.7	39.4	37.1	41.6	29.7	30.1
Can + Dry Soil	33.9	33.0	30.1	33.6	28.4	29.0
Wt of Water	5.8	6.4	7.0	8.0	1.3	1.1
Wt of Dry Soil	20.2	20.9	21.7	21.9	10.4	8.9
Moisture Content	28.7	30.6	32.3	36.5	12.5	12.4
					P.L.= 12.45	

PIT 3 (4% ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	33	27	15		
Can No	KK	T1T1	FE	DB	X2	J4
Wt of Empty Can	16.4	20.2	19.8	20.9	13.8	18.5
Can + Wet Soil	39.1	42.9	42.6	45.2	25.1	31.6
Can + Dry Soil	34.5	37.7	36.7	38.5	23.6	29.9
Wt of Water	4.6	5.0	5.7	6.7	1.5	1.7
Wt of Dry Soil	18.1	17.7	17.1	17.6	11.3	13.5
Moisture Content	25.4	28.2	33.3	38.1	13.3	13.0
					P.L.= 13.15	

PIT 3 (6% ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	34	22	11		
Can No	KA	KB	KC	KD	KZ	KB11
Wt of Empty Can	19.5	15.2	26.8	19.7	26.7	18.9
Can + Wet Soil	42.4	42.2	52.7	49.2	49.9	40.7
Can + Dry Soil	37.0	35.8	45.5	40.3	46.5	37.1
Wt of Water	5.4	6.4	7.2	8.9	3.4	3.6
Wt of Dry Soil	17.5	20.6	18.7	20.6	19.8	18.2
Moisture Content	30.9	34.8	38.5	43.2	17.2	19.8
					P. L. = 18.50	

PIT 3 (8% ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	37	12	12		
Can No	A2	BA	COMP	COMP	KK	W1
Wt of Empty Can	12.0	19.3	26.7	21.2	16.3	26.8
Can + Wet Soil	38.1	46.3	55.2	57.9	28.7	43.4
Can + Dry Soil	32.1	39.5	47.3	42.6	26.8	40.9
Wt of Water	6.0	6.8	7.9	9.0	1.9	2.5
Wt of Dry Soil	20.1	20.2	20.6	21.4	12.4	16.6
Moisture Content	29.9	33.7	38.3	42.1	15.3	15.1
					P.L.= 15.20	

PIT 1 (2% LIME + ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	36	22	11		
Can No	A1	A2	A3	A4	A5	A6
Wt of Empty Can	16.0	10.9	7.8	17.8	8.8	10.2
Can + Wet Soil	36.1	34.4	31.8	45.3	27.1	29.8
Can + Dry Soil	30.7	27.8	24.6	36.7	24.9	26.9
Wt of Water	5.4	6.6	7.2	8.6	2.7	2.6
Wt of Dry Soil	14.7	16.9	16.8	18.9	15.6	18.7
Moisture Content	36.7	39.1	42.9	45.6	17.3	15.6
					P. L. = 16.50	

	PIT 1 (4% LIME + ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	36	22	12		
Can No	B1	B2	B3	B4	B5	B6
Wt of Empty Can	11.8	9.9	9.9	20.0	11.8	15.0
Can + Wet Soil	33.4	33.8	37.9	50.1	30.8	30.6
Can + Dry Soil	27.9	27.4	30.0	41.0	28.4	28.4
Wt of Water	5.5	6.4	7.9	9.1	2.4	2.2
Wt of Dry Soil	16.1	17.5	20.1	21.0	16.6	13.4
Moisture Content	34.1	36.6	39.3	43.3	14.5	16.4
					P. L. =.15.50	

	PIT 1 (6% LIME + ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	38	24	23		
Can No	C14	C21	C31	C41	C20	C18
Wt of Empty Can	15.5	12.9	8.9	9.9	9.9	9.8
Can + Wet Soil	41.4	43.5	40.4	42.2	27.7	28.1
Can + Dry Soil	35.2	35.7	31.8	32.8	25.4	25.8
Wt of Water	6.2	7.8	8.6	9.6	2.3	2.3
Wt of Dry Soil	19.7	22.8	22.9	22.9	15.5	16.0
Moisture Content	31.5	34.2	37.6	41.9	14.8	14.4
					P. L. = 14.60	

	PIT 1 (8% LIME + ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	47	37	22	12		
Can No	D22	D23	D24	D25	D26	D27
Wt of Empty Can	14.7	18.9	14.6	18.2	12.2	13.4
Can + Wet Soil	42.6	49.8	49.3	54.3	28.6	30.7
Can + Dry Soil	35.7	42.1	40.3	44.2	26.6	28.6
Wt of Water	6.9	7.7	9.0	10.1	2.0	2.1
Wt of Dry Soil	21.0	23.4	25.7	26.0	14.4	15.2
Moisture Content	29.4	32.9	35.0	38.8	13.9	13.8
					P. L. 13.90	

	PIT 2 (2% LIME + ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	38	24	13		
Can No	H20	H21	H22	H23	H24	H25
Wt of Empty Can	14.9	18.9	20.1	20.0	9.9	10.0
Can + Wet Soil	32.6	39.1	43.5	55.9	26.0	26.6
Can + Dry Soil	27.6	32.9	36.0	47.6	23.0	23.8
Wt of Water	5.0	6.2	7.5	8.3	3.0	2.8
Wt of Dry Soil	12.7	14.0	15.9	27.6	15.1	13.8
Moisture Content	39.4	44.3	47.2	30.1	19.9	20.2
					P. L. = 20.10	

PIT 2 (4% LIME + ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	38	22	12	K24	K25
Can No	K20	K21	K22	K23	15.6	14.9
Wt of Empty Can	11.7	9.9	9.7	15.7	29.1	30.0
Can + Wet Soil	29.7	31.6	36.3	43.7	26.8	27.5
Can + Dry Soil	24.9	25.5	28.4	34.9	2.3	2.5
Wt of Water	4.8	6.1	7.9	8.8	11.2	12.6
Wt of Dry Soil	13.2	15.6	18.7	19.2	20.5	19.8
Moisture Content	36.4	39.1	42.2	45.8	P. L. = 20.20	

PIT 2 (6% LIME + ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	36	22	12	V24	V25
Can No	V20	V21	V22	V23	20.1	20.7
Wt of Empty Can	9.9	6.8	15.7	19.6	34.4	39.3
Can + Wet Soil	32.9	32.0	44.0	49.4	32.0	37.1
Can + Dry Soil	27.0	25.1	35.9	40.4	2.4	2.2
Wt of Water	5.9	6.9	8.1	9.0	11.9	10.9
Wt of Dry Soil	17.1	18.3	20.2	20.8	20.2	21.2
Moisture Content	34.5	37.7	40.1	43.3	P. L. = 20.70	

PIT 2 (8% LIME + ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	37	22	12	T24	T25
Can No	T20	T21	T22	T23	15.8	17.9
Wt of Empty Can	12.6	20.1	17.0	18.7	30.2	29.6
Can + Wet Soil	37.2	49.3	48.7	50.9	28.0	27.7
Can + Dry Soil	31.0	41.5	39.8	41.3	2.2	1.9
Wt of Water	6.2	7.8	8.9	9.6	12.2	10.0
Wt of Dry Soil	18.4	21.4	22.8	22.6	18.0	19.0
Moisture Content	33.7	36.4	39.0	42.5	P. L. = 18.50	

PIT 3 (2% LIME + ASH)

	LIQUID LIMIT				PLASTIC LIMIT	
No of blows	46	36	22	11	AF	AG
Can No	AB	AC	AD	AE	15.6	16.8
Wt of Empty Can	17.9	16.9	16.2	12.6	31.9	31.0
Can + Wet Soil	34.7	36.8	37.5	37.1	29.4	28.7
Can + Dry Soil	30.5	31.4	31.2	29.5	2.5	2.3
Wt of Water	4.2	5.4	6.3	7.6	13.8	11.9
Wt of Dry Soil	12.6	14.5	15.0	16.9	18.1	19.3
Moisture Content	33.3	37.2	42.0	45.0	P. L. = 18.70	

	PIT 3 (4% LIME + ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	36	23	12	BE	BD
Can No	BA	BB	BC	BD	9.9	12.8
Wt of Empty Can	18.8	26.7	26.7	20.4	24.3	28.7
Can + Wet Soil	38.3	49.0	50.7	46.7	22.1	26.3
Can + Dry Soil	33.6	43.2	44.0	38.7	2.2	2.4
Wt of Water	4.7	5.8	6.7	8.0	12.2	13.5
Wt of Dry Soil	14.8	16.5	17.3	18.3	18.0	17.8
Moisture Content	31.8	35.2	38.9	43.7	P. L. = 17.90	

	PIT 3 (6% LIME + ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	49	39	23	12	MM	VT
Can No	MZ	CO	DE	ST	9.8	12.1
Wt of Empty Can	19.9	20.1	18.7	26.7	24.1	27.5
Can + Wet Soil	44.4	47.2	47.8	56.6	21.9	25.2
Can + Dry Soil	38.6	40.4	39.9	47.8	2.2	2.3
Wt of Water	5.8	6.8	7.9	8.8	12.1	13.1
Wt of Dry Soil	18.7	20.3	21.2	21.1	18.2	17.6
Moisture Content	31.0	33.5	37.3	41.7	P. L. = 17.60	

	PIT 3 (8% LIME + ASH) LIQUID LIMIT				PLASTIC LIMIT	
No of blows	48	37	23	13	F24	F25
Can No	F20	F21	F22	F23	18.9	20.1
Wt of Empty Can	15.7	10.8	9.9	20.0	34.1	35.7
Can + Wet Soil	42.8	41.2	43.3	52.6	31.9	33.6
Can + Dry Soil	36.7	33.7	34.6	43.1	2.2	2.1
Wt of Water	6.1	7.5	8.9	9.5	13.0	13.5
Wt of Dry Soil	21.0	22.9	24.7	23.1	16.9	15.6
Moisture Content	29.0	32.8	36.0	41.1	P. L. = 16.30	

APPENDIX B

APPENDIX B1 - STANDARD PROCTOR COMPACTION TEST RESULTS

PIT 1 (0%) LIME				
Mould + wet soil	5000	5250	5200	5300
Mould	3300	3300	3300	3300
Weight of wet soil	1700	1950	1900	1800
Volume of mould	1000	1000	1000	1000
Wet density	1.70	195	1.90	1.80
Can No	BB	G5	Q	R
Can weight	19.7	13.4	26.7	15.0
Can + wet soil	65.0	54.9	91.0	85.5
Can + dry soil	55.1	49.6	85.1	75.1
Weight of dry soil	3.5	5.3	6.9	4.3
Weight of water	35.4	36.2	37.9	37.0
Moisture content	9.9	14.6	18.2	13.7
Dry density	1.64	1.83	1.73	1.65

PIT 1 (2%) LIME				
Mould + wet soil	5050	5350	5450	5300
Mould	3300	3300	3300	3300
Weight of wet soil	1750	2050	2150	2000
Volume of mould	1000	1000	1000	1000
Wet density	1.75	2.05	2.15	2.00
Can No	D3	P2	B1	V3
Can weight	19.7	20.0	26.9	21.9
Can + wet soil	86.3	75.4	85.7	78.8
Can + dry soil	81.7	70.0	78.1	61.0
Weight of dry soil	4.6	5.4	7.8	7.8
Weight of water	62.0	50.0	51.2	39.1
Moisture content	7.4	10.8	15.2	19.9
Dry density	1.63	1.82	1.87	1.67

PIT 1 (4%) LIME				
Mould + wet soil	5050	5350	5450	5300
Mould	3300	3300	3300	3300
Weight of wet soil	1750	2050	2150	2000
Volume of mould	1000	1000	1000	1000
Wet density	1.75	2.05	2.15	2.00
Can No	D3	P2	B1	V3
Can weight	19.7	20.0	26.9	21.9
Can + wet soil	86.3	75.4	85.7	78.8
Can + dry soil	81.7	70.0	78.1	61.0
Weight of dry soil	4.6	5.4	7.8	7.8
Weight of water	62.0	50.0	51.2	39.1
Moisture content	7.4	10.8	15.2	19.9
Dry density	1.63	1.82	1.87	1.67

PIT 1 (6%) OF LIME				
Mould + wet soil	5250	5400	5600	5550
Mould	3300	3300	3300	3300
Weight of wet soil	1950	2100	2300	2250
Volume of mould	1000	1000	1000	1000
Wet density	1.95	2.10	2.3	2.25
Can No	KB	QB	TU	Q5
Can weight	21.5	19.8	27.0	27.0
Can + wet soil	100.1	88.5	89.4	90.0
Can + dry soil	93.9	80.4	79.9	78.3
Weight of dry soil	72.4	60.6	52.9	51.3
Weight of water	6.2	8.1	9.5	11.7
Moisture content	8.6	13.4	18.0	22.8
Dry density	1.78	1.85	1.95	1.83

PIT 1 (8%) OF LIME				
Mould + wet soil	5300	5450	5600	5550
Mould	3300	3300	3300	3300
Weight of wet soil	2000	2150	2300	2250
Volume of mould	1000	1000	1000	1000
Wet density	2.00	2.15	2.30	2.25
Can No	D3	N2	N1	N5
Can weight	19.7	20.1	26.8	27.6
Can + wet soil	71.9	80.0	78.5	100.0
Can + dry soil	68.5	74.1	71.8	88.2
Weight of dry soil	48.8	54.0	45.0	60.6
Weight of water	3.4	5.9	6.7	11.8
Moisture content	7.0	10.9	14.9	19.5
Dry density	1.87	1.94	2.0	1.88

PIT 2 (0%) OF LIME				
Mould + wet soil	4850	5000	5200	5150
Mould	3300	3300	3300	3300
Weight of wet soil	1550	1700	1900	1850
Volume of mould	1000	1000	1000	1000
Wet density	1.55	1.70	1.90	1.85
Can No	T3	B3	B4	FI
Can weight	26.8	20.2	19.9	26.9
Can + wet soil	79.6	65.4	62.1	73.4
Can + dry soil	74.3	59.8	55.3	64.9
Weight of dry soil	5.3	5.9	6.8	8.5
Weight of water	47.5	39.8	35.4	38.0
Moisture content	11.2	14.9	19.2	22.4
Dry density	1.39	1.48	1.59	1.51

PIT 2 (2%) OF LIME				
Mould + wet soil	5000	5250	5400	5100
Mould	3400	3400	3400	3400
Weight of wet soil	1600	1850	2000	1850
Volume of mould	1000	1000	1000	1000
Wet density	1.60	1.85	2.00	1.85
Can No	T1	T2	T3	T4
Can weight	18.7	21.8	19.9	20.0
Can + wet soil	99.8	87.0	76.2	80.4
Can + dry soil	92.0	78.1	66.0	67.9
Weight of dry soil	7.8	8.9	10.2	12.5
Weight of water	73.3	56.3	46.1	47.9
Moisture content	10.6	15.8	22.1	26.1
Dry density	1.45	1.60	1.64	1.47

PIT 2 (4%) OF LIME				
Mould + wet soil	4950	5100	5300	5150
Mould	3300	3300	3300	3300
Weight of wet soil	1680	1800	2000	1850
Volume of mould	1000	1000	1000	1000
Wet density	1.65	1.80	2.00	
Can No	A1	A2	A3	A4
Can weight	16.3	18.5	11.7	11.9
Can + wet soil	61.8	68.8	61.1	61.7
Can + dry soil	57.4	62.9	54.5	53.7
Weight of dry soil	4.4	5.9	6.6	8.0
Weight of water	41.1	44.4	42.8	42.0
Moisture content	10.7	13.3	15.4	19.0
Dry density	1.49	1.59	1.69	1.55

PIT 2 (6%) OF LIME				
Mould + wet soil	5050	5150	5350	5300
Mould	3300	3300	3300	3300
Weight of wet soil	1750	1850	2050	2000
Volume of mould	1000	1000	1000	1000
Wet density	1.75	1.85	2.05	2.00
Can No	C1	C2	C3	C4
Can weight	15.6	18.2	16.7	12.2
Can + wet soil	62.5	64.3	61.5	57.4
Can + dry soil	58.3	58.9	55.2	49.8
Weight of dry soil	4.2	5.4	6.3	7.6
Weight of water	42.7	40.7	38.5	37.6
Moisture content	9.8	13.3	16.4	20.2
Dry density	1.59	1.63	1.76	1.66

PIT 2 (8%) OF LIME				
Mould + wet soil	5250	5450	5600	5500
Mould	3300	3300	3300	3300
Weight of wet soil	1950	2150	2300	2200
Volume of mould	1000	1000	1000	1000
Wet density	1.95	2.15	2.30	2.20
Can No	A	B	C	D
Can weight	9.9	13.7	12.2	21.6
Can + wet soil	72.9	68.8	72.6	86.6
Can + dry soil	67.9	62.7	64.4	76.3
Weight of dry soil	5.0	6.1	8.2	10.3
Weight of water	58.0	49.0	52.2	54.7
Moisture content	8.6	12.4	15.8	18.8
Dry density	1.61	1.69	1.77	1.60

PIT 3 (0%) OF LIME				
Mould + wet soil	5500	5650	5700	5600
Mould	3800	3800	3800	3800
Weight of wet soil	1700	1850	1900	1800
Volume of mould	1000	1000	1000	1000
Wet density	1.70	1.85	1.90	1.80
Can No	N1	N	J1	S1
Can weight	20.0	19.0	22.2	20.1
Can + wet soil	65.8	60.6	82.9	76.6
Can + dry soil	63.2	56.8	78.5	71.3
Weight of dry soil	2.6	3.8	4.4	5.3
Weight of water	43.2	46.8	36.3	51.2
Moisture content	6.0	8.1	12.1	10.4
Dry density	1.60	1.71	1.74	1.56

PIT 3 (2%) OF LIME				
Mould + wet soil	5100	5250	5400	5300
Mould	3400	3400	3400	3400
Weight of wet soil	1700	1850	2000	1900
Volume of mould	1000	1000	1000	1000
Wet density	1.70	1.85	2.00	1.90
Can No	V1	V2	V3	V4
Can weight	15.7	19.9	26.7	18.9
Can + wet soil	100.4	97.4	91.8	78.5
Can + dry soil	95.2	91.0	83.9	69.8
Weight of dry soil	5.2	6.4	7.9	8.7
Weight of water	79.5	71.1	57.2	50.9
Moisture content	6.5	9.0	13.8	17.1
Dry density	1.60	1.70	1.75	1.62

PIT 3 (4%) OF LIME				
Mould + wet soil	5250	5450	5600	5500
Mould	3300	3300	3300	3300
Weight of wet soil	1950	2150	2300	2200
Volume of mould	1000	1000	1000	1000
Wet density	1.95	2.15	2.30	2.20
Can No	A	B	C	D
Can weight	9.9	13.7	12.2	21.6
Can + wet soil	72.9	68.8	72.6	86.6
Can + dry soil	67.9	62.7	64.4	76.3
Weight of dry soil	5.0	6.1	8.2	10.3
Weight of water	58.0	49.0	52.2	54.7
Moisture content	8.6	12.4	15.8	18.8
Dry density	1.61	1.69	1.77	1.60

PIT 3 PIT 3 (6%) OF LIME				
Mould + wet soil	5100	5300	5450	5400
Mould	3300	3300	3300	3300
Weight of wet soil	1800	2000	2150	2100
Volume of mould	1000	1000	1000	1000
Wet density	1.80	2.00	2.15	2.1
Can No	J_3	B_1	Z_1	Comp 14
Can weight	20.0	27.0	20.1	19.3
Can + wet soil	75.9	83.5	81.6	100.1
Can + dry soil	70.9	76.7	72.5	86.7
Weight of dry soil	50.9	49.7	52.4	67.4
Weight of water	5.00	6.8	9.1	13.4
Moisture content	9.8	13.7	17.4	19.9
Dry density	1.64	1.75	1.83	1.75

PIT 3 (8%) OF LIME				
Mould + wet soil	5200	5350	5550	5500
Mould	3300	3300	3300	3300
Can No	WH	KE	D_4	Z_3
Can weight	20.0	22.9	20.4	21.3
Can + wet soil	86.0	85.9	96.6	76.2
Can + dry soil	77.5	79.4	87.1	65.7
W.W	5.9	6.5	9.5	10.5
Weight of wet soil	1900	2050	2250	2200
Volume of mould	1000	1000	1000	1000
Wet density	1.90	2.05	2.25	2.2
Weight of dry soil	57.5	56.5	66.7	44.4
M.C	8.5	11.5	14.2	23.6
Dry density	1.81	1.84	1.97	1.78

PIT 1 (2%) OF ASH				
Mould + wet soil	5500	5600	5700	5600
Mould	3800	3800	3800	3800
Weight of wet soil	1700	1800	1900	1800
Volume of mould	1000	1000	1000	1000
Wet density	1.70	1.80	1.90	1.80
Can No	Y	J3	P	5
Can weight	26.9	15.2	19.8	26.8
Can + wet soil	81.7	59.3	88.5	71.6
Can + dry soil	78.1	55.0	79.2	65.4
Weight of dry soil	3.6	4.3	5.3	6.2
Weight of water	51.2	39.8	38.0	38.6
Moisture content	7.0	10.8	13.2	16.1
Dry density	1.59	1.62	1.68	1.55

PIT 1 (4%) OF ASH				
Mould + wet soil	5050	5200	5400	5350
Mould	3400	3400	3400	3400
Weight of wet soil	1650	1800	2000	1950
Volume of mould	1000	1000	1000	1000
Wet density	1.65	1.80	2.00	1.95
Can No	Z1	Z2	Z3	Z4
Can weight	10.0	26.5	19.7	20.1
Can + wet soil	70.5	84.7	82.6	82.8
Can + dry soil	66.0	78.4	73.7	72.5
Weight of dry soil	4.5	6.3	8.9	10.3
Weight of water	56.0	51.9	54.0	52.4
Moisture content	8.0	12.1	16.5	19.7
Dry density	1.53	1.61	1.72	1.63

PIT 1 (6%) OF ASH				
Mould + wet soil	5350	5500	5650	5500
Mould	3800	3800	3800	3800
Weight of wet soil	1550	1700	1850	1700
Volume of mould	1000	1000	1000	1000
Wet density	1.55	1.70	1.85	1.70
Can No	GROUP2	TTI	4	II
Can weight	19.9	26.8	14.2	27.2
Can + wet soil	67.9	72.0	53.7	89.1
Can + dry soil	64.4	67.8	48.9	81.0
Weight of dry soil	3.5	4.2	4.8	8.1
Weight of water	54.5	48.6	40.0	68.8
Moisture content	10.0	13.2	12.5	14.7
Dry density	1.45	1.55	1.65	1.50

PIT 1 (8%) OF ASH				
Mould + wet soil	5450	5550	5700	5550
Mould	3800	3800	3800	3800
Weight of wet soil	1600	1750	1900	1750
Volume of mould	1000	1000	1000	1000
Wet density	1.60	1.75	1.90	1.75
Can No	V4	G1	P1	P4
Can weight	27.4	25.6	20.1	26.9
Can + wet soil	74.7	64.6	67.6	94.0
Can + dry soil	71.3	60.3	62.0	85.3
Weight of dry soil	3.4	4.5	5.6	8.7
Weight of water	43.9	44.0	41.9	58.4
Moisture content	7.7	10.2	13.4	14.9
Dry density	1.49	1.59	1.68	1.48

PIT 2 (2%) OF ASH				
Mould + wet soil	4750	4950	5150	5000
Mould	3300	3300	3300	3300
Weight of wet soil	1450	1650	1850	1700
Volume of mould	1000	1000	1000	1000
Wet density	1.45	1.65	1.85	1.70
Can No	D1	D2	D3	D4
Can weight	26.7	15.9	18.7	11.4
Can + wet soil	79.7	66.5	80.9	75.2
Can + dry soil	75.4	60.9	72.0	64.3
Weight of dry soil	4.3	5.6	8.9	10.9
Weight of water	48.7	45.0	53.3	52.8
Moisture content	8.8	12.4	16.7	20.6
Dry density	1.33	1.47	1.58	1.41

PIT 2 (4%) OF ASH				
Mould + wet soil	4850	5050	5250	5150
Mould	3300	3300	3300	3300
Weight of wet soil	1550	1750	1950	1850
Volume of mould	1000	1000	1000	1000
Wet density	1.55	1.75	1.95	1.85
Can No	K1	K2	K3	K4
Can weight	26.7	26.6	26.6	19.9
Can + wet soil	79.7	68.8	78.8	80.6
Can + dry soil	75.4	63.4	69.8	68.7
Weight of dry soil	48.7	36.8	43.2	48.8
Weight of water	4.3	5.4	9.0	11.9
Moisture content	8.8	14.7	20.8	24.4
Dry density	1.42	1.53	1.61	1.49

PIT 2 (6%) OF ASH				
Mould + wet soil	4800	5050	5250	5200
Mould	3300	3300	3300	3300
Weight of wet soil	1500	1750	1950	1900
Volume of mould	1000	1000	1000	1000
Wet density	1.50	1.75	1.95	1.90
Can No	U3	YZ	OO	V1
Can weight	19.8	21.0	20.0	19.9
Can + wet soil	68.9	70.8	60.7	70.7
Can + dry soil	64.7	65.6	54.7	61.5
Weight of dry soil	4.2	5.2	6.0	9.2
Weight of water	44.9	36.0	34.7	41.6
Moisture content	9.4	13.5	17.3	22.1
Dry density	1.37	1.54	1.66	1.56

PIT 2 (8%) OF ASH				
Mould + wet soil	4900	5050	5250	5450
Mould	3400	3400	3400	3400
Weight of wet soil	1500	1650	1850	2050
Volume of mould	1000	1000	1000	1000
Wet density	1.50	1.65	1.85	2.05
Can No	K0	K1	K2	K3
Can weight	20.1	20.0	26.9	26.9
Can + wet soil	82.1	74.4	82.2	80.1
Can + dry soil	77.4	68.7	74.5	70.9
Weight of dry soil	4.7	5.7	7.7	9.2
Weight of water	57.3	48.7	47.6	44.0
Moisture content	8.2	11.7	16.2	20.9
Dry density	1.39	1.48	1.59	1.70

PIT 3 (2%) OF ASH				
Mould + wet soil	4950	5150	5250	5150
Mould	3300	3300	3300	3300
Weight of wet soil	1650	1850	1950	1850
Volume of mould	1000	1000	1000	1000
Wet density	1.65	1.85	1.95	1.85
Can No	Q1	Q2	Q3	Q4
Can weight	9.9	13.7	10.3	20.1
Can + wet soil	54.9	58.2	62.8	79.4
Can + dry soil	51.1	53.4	56.0	70.8
Weight of dry soil	3.8	4.8	6.8	8.6
Weight of water	41.2	39.7	45.7	50.7
Moisture content.	9.2	12.1	14.9	17.0
Dry density	1.51	1.65	1.70	1.5

PIT 3 (4%) OF ASH				
Mould + wet soil	5000	5150	5400	5350
Mould	3400	3400	3400	3400
Weight of wet soil	1600	1750	2000	1950
Volume of mould	1000	1000	1000	1000
Wet density	1.60	1.75	2.00	1.95
Can No	X1	X2	X3	X4
Can weight	26.8	12.8	12.1	26.7
Can + wet soil	81.7	69.4	58.5	74.2
Can + dry soil	76.3	62.6	51.2	65.3
Weight of dry soil	5.4	6.8	7.3	8.9
Weight of water	49.5	49.8	39.1	38.6
Moisture content	10.7	13.7	18.7	23.1
Dry density	1.45	1.54	1.68	1.58

PIT 3 (6%) OF ASH				
Mould + wet soil	4950	5250	5400	5500
Mould	3300	3300	3300	3300
Weight of wet soil	1650	1950	2100	2200
Volume of mould	1000	1000	1000	1000
Wet density	1.65	1.95	2.10	2..20
Can No	W0	W1	W2	W3
Can weight	19.5	19.6	19.9	19.9
Can + wet soil	74.1	67.0	66.6	63.8
Can + dry soil	69.6	62.1	59.1	55.0
Weight of dry soil	4.5	5.1	7.5	8.8
Weight of water	50.1	42.5	49.2	45.1
Moisture content	9.0	12.0	15.2	19.5
Dry density	1.51	1.74	1.75	1.84

PIT 3 (8%) OF ASH				
Mould + wet soil	5100	5250	5350	5200
Mould	3400	3400	3400	3400
Weight of wet soil	1700	1850	1950	1800
Volume of mould	1000	1000	1000	1000
Wet density	1.70	1.85	1.95	1.80
Can No	Z1	Z2	Z3	Z4
Can weight	20.0	16.9	18.3	30.5
Can + wet soil	92.9	84.1	88.7	96.9
Can + dry soil	87.6	76.8	78.9	87.0
Weight of dry soil	5.3	7.3	8.8	9.9
Weight of water	67.6	59.9	64.6	56.5
Moisture content	7.8	12.2	14.5	17.5
Dry density	1.58	1.65	1.79	1.53

PIT 1 (2%) LIME + ASH				
Mould + wet soil	5150	5350	5500	5450
Mould	3500	3500	3500	3500
Weight of wet soil	1650	1850	2000	1950
Volume of mould	1000	1000	1000	1000
Wet density	1.60	1.85	2.00	1.95
Can No	A1	A2	A3	A4
Can weight	19.9	19.7	19.4	19.8
Can + wet soil	63.1	64.4	70.2	68.3
Can + dry soil	59.3	59.1	63.7	60.0
Weight of dry soil	3.8	5.3	6.5	8.3
Weight of water	39.4	39.4	37.3	40.2
Moisture content	9.6	13.5	17.4	20.6
Dry density	1.51	1.63	1.70	1.62

PIT 1 (4%) LIME + ASH				
Mould + wet soil	5150	5350	5550	5400
Mould	3500	3500	3500	3500
Weight of wet soil	1650	1850	2050	1900
Volume of mould	1000	1000	1000	1000
Wet density	1.65	1.85	2.05	1.90
Can No	N1	N2	N3	N4
Can weight	19.9	19.8	21.6	22.5
Can + wet soil	75.6	73.9	73.5	77.5
Can + dry soil	69.7	66.8	65.5	67.7
Weight of dry soil	5.9	7.1	8.0	9.8
Weight of water	49.8	47.0	43.9	45.2
Moisture content	11.8	15.1	18.2	21.9
Dry density	1.48	1.61	1.73	1.56

PIT 1 (6%) LIME + ASH				
Mould + wet soil	5250	5400	5600	5500
Mould	3500	3500	3500	3500
Weight of wet soil	1750	1900	2100	2000
Volume of mould	1000	1000	1000	1000
Wet density	1.75	1.90	2.10	2.00
Can No	F1	F2	F3	F4
Can weight	20.0	20.0	20.3	19.1
Can + wet soil	72.0	75.5	74.7	79.8
Can + dry soil	68.9	70.1	67.7	69.9
Weight of dry soil	3.1	5.4	7.5	9.9
Weight of water	48.9	50.1	46.9	50.8
Moisture content	6.3	10.8	16.0	19.5
Dry density	1.65	1.71	1.81	1.67

PIT 1 (8%) LIME + ASH				
Mould + wet soil	5100	5300	5500	5450
Mould	3500	3500	3500	3500
Weight of wet soil	1600	1800	2000	1950
Volume of mould	1000	1000	1000	1000
Wet density	1.60	1.80	2.00	1.95
Can No	J1	J2	J3	J4
Can weight	10.0	26.8	20.0	26.8
Can + wet soil	74.3	84.7	77.6	83.7
Can + dry soil	70.0	78.7	70.5	75.1
Weight of dry soil	4.3	6.0	7.1	8.6
Weight of water	60.0	51.9	50.5	48.3
Moisture content	7.2	11.6	14.1	17.8
Dry density	1.49	1.61	1.75	1.66

PIT 2 (2%) LIME + ASH				
Mould + wet soil	5050	5200	5300	5250
Mould	3500	3500	3500	3500
Weight of wet soil	1550	1700	1800	1750
Volume of mould	1000	1000	1000	1000
Wet density	1.55	1.70	1.80	1.75
Can No	U1	U2	U3	U4
Can weight	27.0	27.1	16.2	15.8
Can + wet soil	71.7	75.3	61.0	44.7
Can + dry soil	66.5	68.3	53.8	38.4
Weight of dry soil	5.2	6.0	7.2	9.3
Weight of water	52.8	41.2	37.5	41.4
Moisture content	9.8	14.6	19.2	22.6
Dry density	1.41	1.48	1.59	1.43

PIT 2 (4%) LIME + ASH				
Mould + wet soil	5100	5250	5450	5350
Mould	3500	3500	3500	3500
Weight of wet soil	1600	1750	1950	1850
Volume of mould	1000	1000	1000	1000
Wet density	1.60	1.75	1.95	1.85
Can No	V1	V2	V3	V4
Can weight	19.8	20.1	17.9	24.6
Can + wet soil	78.2	77.2	71.3	83.3
Can + dry soil	73.1	70.3	63.0	72.6
Weight of dry soil	5.1	6.9	8.3	10.7
Weight of water	53.3	50.2	45.1	48.0
Moisture content	9.6	13.7	18.4	22.3
Dry density	1.46	1.54	1.65	1.47

PIT 2 (6%) LIME + ASH				
Mould + wet soil	5200	5400	5550	5450
Mould	3500	3500	3500	3500
Weight of wet soil	1700	1900	2050	1950
Volume of mould	1000	1000	1000	1000
Wet density	1.70	1.90	2.05	1.95
Can No	M1	M2	M3	M4
Can weight	19.7	16.0	23.1	23.1
Can + wet soil	87.1	69.5	82.5	81.2
Can + dry soil	79.7	61.2	72.7	70.0
Weight of dry soil	7.4	8.3	9.8	11.2
Weight of water	60.0	51.2	49.6	46.9
Moisture content	12.3	16.2	19.8	23.9
Dry density	1.51	1.64	1.71	1.57

PIT 2 (8%) LIME + ASH				
Mould + wet soil	5250	5450	5650	5500
Mould	3500	3500	3500	3500
Weight of wet soil	1750	1950	2150	2000
Volume of mould	1000	1000	1000	1000
Wet density	1.75	1.95	2.15	2.00
Can No	D1	D2	D3	D4
Can weight	15.7	22.2	12.1	22.0
Can + wet soil	62.2	71.3	61.4	71.0
Can + dry soil	56.7	65.0	53.6	61.8
Weight of dry soil	5.5	6.3	8.8	10.1
Weight of water	41.0	37.0	41.5	39.8
Moisture content	13.4	17.0	21.2	25.4
Dry density	1.54	1.67	1.78	1.59

PIT 3 (2%) LIME + ASH				
Mould + wet soil	5150	5350	5450	5350
Mould	3500	3500	3500	3500
Weight of wet soil	1650	1850	2000	1900
Volume of mould	1000	1000	1000	1000
Wet density	1.65	1.85	2.00	1.90
Can No	K1	K2	K3	K4
Can weight	15.2	11.7	10.7	22.5
Can + wet soil	54.6	50.7	68.4	78.5
Can + dry soil	50.6	45.2	58.8	67.5
Weight of dry soil	4.0	5.5	9.6	11.0
Weight of water	35.4	33.5	48.1	45.0
Moisture content	12.0	16.4	20.0	24.4
Dry density	1.47	1.59	1.67	1.53

PIT 3 (4%) LIME + ASH				
Mould + wet soil	5050	5250	5400	5300
Mould	3500	3500	3500	3500
Weight of wet soil	1550	1750	1900	1800
Volume of mould	1000	1000	1000	1000
Wet density	1.55	1.75	1.90	1.80
Can No	S1	S2	S3	S4
Can weight	18.7	26.7	19.8	19.9
Can + wet soil	77.6	82.6	72.6	75.3
Can + dry soil	72.9	76.6	64.8	65.4
Weight of dry soil	4.7	6.1	7.8	9.9
Weight of water	54.2	49.9	45.0	45.5
Moisture content	8.7	12.2	17.3	21.8
Dry density	1.43	1.56	1.62	1.48

PIT 3 (6%) LIME + ASH				
Mould + wet soil	5250	5450	5650	5550
Mould	3500	3500	3500	3500
Weight of wet soil	1750	1950	2150	2050
Volume of mould	1000	1000	1000	1000
Wet density	1.75	1.95	2.15	2.05
Can No	X1	X2	X3	X4
Can weight	18.0	24.6	18.4	20.2
Can + wet soil	68.2	66.0	66.8	77.4
Can + dry soil	63.6	60.8	59.3	66.5
Weight of dry soil	4.6	5.2	7.5	10.9
Weight of water	45.6	36.2	40.9	46.3
Moisture content	10.1	14.4	18.3	23.5
Dry density	1.59	1.70	1.82	1.66

PIT 3 (8%) LIME + ASH				
Mould + wet soil	5300	5500	5700	5500
Mould	3500	3500	3500	3500
Weight of wet soil	1800	2000	2200	2000
Volume of mould	1000	1000	1000	1000
Wet density	1.80	2.00	2.20	2.00
Can No	W1	W2	W3	W4
Can weight	18.3	26.7	15.3	27.9
Can + wet soil	76.0	85.1	72.3	85.8
Can + dry soil	70.9	77.9	62.8	74.5
Weight of dry soil	5.1	7.2	9.0	11.3
Weight of water	52.6	51.2	47.5	46.6
Moisture content	9.4	14.1	19.8	24.2
Dry density	1.65	1.75	1.85	1.61

APPENDIX B2 - CALIFORNIA BEARING RATIO TEST RESULTS

Pen. = Penetration; DGR = Dial Guage reading

	PIT 1 (LIME)									
	0%		2%		4%		6%		8%	
Pen.	DGR	Load	DGR	Load	DGR	Load	DGR	Load	DGR	Load
50	187	4.6	179	4.5	93	2.3	144	3.6	99	2.5
100	233	5.8	241	6.0	146	3.7	212	5.3	171	4.3
150	271	6.8	280	7.0	186	4.7	272	6.8	265	6.6
200	301	7.5	323	8.1	229	5.7	315	7.9	326	8.2
250	346	8.7	377	9.4	263	6.7	369	9.2	377	9.4
300	388	9.7	421	10.5	318	8.0	410	10.3	423	10.6
350	428	10.7	470	11.8	385	9.6	441	11.0	486	12.2
400	469	11.7	501	12.5	461	11.5	504	12.6	531	13.3
450	527	13.2	532	13.3	522	13.1	558	14.0	602	15.1
500	564	14.1	568	14.2	589	14.7	605	15.1	652	16.3
550	571	14.3	580	14.5	610	15.3	622	15.6	673	16.8
600	579	14.5	588	14.7	621	15.5	631	15.8	691	17.3
650	586	14.7	596	14.9	630	15.8	650	16.3	715	17.9
700	592	14.8	606	15.2	635	15.9	659	16.5	731	18.3
750	596	14.9	611	15.3	640	16.0	667	16.7	739	18.4

	PIT 2 (LIME)									
	0%		2%		4%		6%		8%	
Pen.	DGR	Load	DGR	Load	DGR	Load	DGR	Load	DGR	Load
50	57	1.4	45	1.1	61	1.5	59	1.5	66	1.7
100	89	2.2	103	2.3	111	2.8	115	2.9	199	2.5
150	110	2.8	137	3.4	143	3.6	154	3.9	120	3.0
200	142	3.6	175	4.4	187	4.7	197	4.9	152	3.8
250	183	4.6	205	5.1	215	5.4	220	5.5	187	4.7
300	211	5.3	225	5.6	230	5.8	259	6.5	220	5.5
350	233	5.8	245	6.1	267	6.7	300	7.5	255	6.4
400	267	6.7	285	7.1	312	7.8	322	8.1	302	7.6
450	318	8.0	327	8.2	340	8.5	368	9.2	349	8.7
500	338	8.5	341	8.5	373	9.3	405	10.1	397	9.9
550	346	8.7	356	8.9	388	9.7	421	10.5	407	10.2
600	362	9.1	371	9.3	400	10.0	438	11.0	419	10.5
650	369	9.2	378	9.5	410	10.3	461	11.5	426	10.7
700	375	9.4	384	9.6	416	10.4	472	11.8	442	11.1
750	379	9.5	390	9.8	422	10.6	488	12.2	459	11.5

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	PIT 3 (LIME)									
	0%		2%		4%		6%		8%	
Pen.	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**
50	32	0.8	59	1.5	65	1.6	52	1.3	100	2.5
100	53	1.3	102	2.6	110	2.8	125	3.1	164	4.1
150	98	2.5	148	3.7	151	3.8	179	4.5	201	5.0
200	132	3.3	181	4.5	197	4.9	222	5.6	243	6.1
250	177	4.4	211	5.3	233	5.8	269	6.7	284	7.1
300	218	5.5	239	6.0	271	6.8	317	7.9	321	8.0
350	256	6.4	279	7.0	316	7.9	345	8.6	373	9.3
400	300	7.5	310	7.8	355	8.9	385	9.6	411	10.3
450	344	8.6	350	8.8	401	10.0	423	10.6	450	11.3
500	395	9.9	388	9.7	434	10.9	469	11.7	496	12.4
550	424	10.6	398	10.0	458	11.5	482	12.1	518	13.0
600	447	11.2	420	10.5	479	12.0	506	12.7	538	13.5
650	469	11.7	441	11.0	492	12.3	531	13.3	555	13.9
700	483	12.1	464	11.6	515	12.9	560	14.0	571	14.3
750	501	12.5	482	12.1	529	13.2	583	14.6	594	14.9

	PIT 1 (ASH)									
	0%		2%		4%		6%		8%	
Pen.	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**
50	187	4.6	125	3.1	97	2.4	110	2.8	172	4.3
100	233	5.8	177	4.4	153	3.8	161	4.0	230	5.8
150	271	6.8	228	5.7	204	5.1	216	5.4	273	6.8
200	301	7.5	279	7.0	262	6.7	269	6.7	310	7.8
250	346	8.7	327	8.2	308	7.7	328	8.2	351	8.8
300	388	9.7	370	9.3	350	8.8	387	9.7	399	10.0
350	428	10.7	413	10.3	400	10.0	439	10.7	435	10.9
400	469	11.7	460	11.5	447	11.2	482	12.1	473	11.8
450	527	13.2	501	12.5	499	12.5	534	13.4	528	13.2
500	564	14.1	558	14.0	569	14.2	589	14.7	573	14.3
550	571	14.3	563	14.1	584	14.6	608	15.2	589	14.7
600	579	14.5	569	14.2	596	14.9	618	15.5	601	15.0
650	586	14.7	578	14.5	604	15.1	628	15.7	613	15.3
700	592	14.8	586	14.7	610	15.3	636	15.9	628	15.7
750	596	14.9	592	14.8	618	15.5	642	16.1	636	15.9

	PIT 2 (ASH)									
	0%		2%		4%		6%		8%	
Pen.	DGR	Load	DGR	Load	DGR	Load	DGR	Load	DGR	Load
50	57	1.4	28	0.7	45	1.1	68	1.7	71	1.8
100	89	2.2	59	1.5	82	2.5	104	2.6	112	2.8
150	110	2.8	97	2.4	101	2.5	133	3.3	144	3.6
200	142	3.6	112	2.8	132	3.3	168	4.2	175	4.4
250	183	4.6	134	3.4	166	4.2	217	5.4	226	5.7
300	211	5.3	169	4.2	204	5.1	245	6.1	250	6.3
350	233	5.8	203	5.1	236	5.9	272	6.8	281	7.0
400	267	6.7	237	5.9	271	6.8	311	7.8	320	8.0
450	318	8.0	286	6.2	307	7.7	333	8.3	341	8.5
500	338	8.5	339	7.2	342	8.6	361	9.0	378	9.5
550	346	8.7	349	8.7	356	8.9	382	9.6	394	9.9
600	362	9.1	360	9.0	371	9.3	395	9.9	423	10.7
650	369	9.2	374	9.4	379	9.5	410	10.3	441	11.0
700	375	9.4	381	9.5	390	9.8	421	10.5	456	11.4
750	379	9.5	389	9.7	396	9.9	430	10.8	471	11.8

	PIT 3 (ASH)									
	0%		2%		4%		6%		8%	
Pen.	DGR	Load	DGR	Load	DGR	Load	DGR	Load	DGR	Load
50	32	0.8	45	1.1	38	1.0	56	1.4	98	2.2
100	53	1.3	63	1.6	66	1.7	107	2.7	132	3.3
150	98	2.5	82	2.1	95	2.4	153	3.8	178	4.5
200	132	3.3	105	2.6	120	3.0	181	4.5	212	5.3
250	177	4.4	137	3.4	159	4.0	202	5.1	236	5.9
300	218	5.5	185	4.6	197	4.9	239	6.0	276	6.9
350	256	6.4	228	5.7	238	6.0	277	6.9	310	7.8
400	300	7.5	266	6.7	282	7.1	315	7.9	334	8.4
450	344	8.6	306	7.7	328	8.2	351	8.8	371	9.3
500	395	9.9	347	8.7	375	9.4	390	9.8	415	10.4
550	424	10.6	366	9.2	392	9.8	425	10.6	437	10.9
600	447	11.2	382	9.6	409	10.2	452	11.3	453	11.3
650	469	11.7	395	9.9	422	10.6	476	11.9	464	11.6
700	483	12.1	405	10.1	438	11.0	486	12.2	472	11.8
750	501	12.5	415	10.4	455	11.4	497	12.4	481	12.0

					PIT 1 (LIME + ASH)					
	0%		2%		4%		6%		8%	
Pen.	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**
50	187	4.6	163	4.1	158	4.0	169	4.2	173	4.3
100	233	5.8	228	5.7	213	5.3	221	5.5	237	5.9
150	271	6.8	275	6.9	260	6.5	279	7.0	284	7.1
200	301	7.5	314	7.9	307	7.7	322	8.1	325	8.1
250	346	8.7	355	8.9	366	9.2	371	9.3	377	9.4
300	388	9.7	391	9.7	402	10.1	410	10.3	419	10.5
350	428	10.7	432	10.8	444	11.1	462	11.6	456	11.4
400	469	11.7	487	12.2	493	12.3	520	13.0	501	12.5
450	527	13.2	521	13.0	532	13.3	555	13.9	539	13.5
500	564	14.1	562	14.1	574	14.4	596	14.9	580	14.5
550	571	14.3	591	14.8	587	14.7	651	16.3	637	15.9
600	579	14.5	635	15.9	641	16.0	684	17.1	670	16.8
650	586	14.7	671	16.8	683	17.1	710	17.8	691	17.3
700	592	14.8	692	17.3	702	17.6	755	18.9	713	17.8
750	596	14.9	721	18.0	739	18.5	771	19.3	728	18.2

					PIT 2 (LIME + ASH)					
	0%		2%		4%		6%		8%	
Pen.	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**	**DGR**	**Load**
50	57	1.4	39	1.0	77	1.9	64	1.6	49	1.2
100	89	2.2	68	1.7	107	2.7	98	2.5	87	2.2
150	110	2.8	97	2.4	131	3.3	139	3.5	109	2.7
200	142	3.6	122	3.1	163	4.1	185	4.6	140	3.5
250	183	4.6	157	3.9	120	4.5	213	5.3	188	4.7
300	211	5.3	182	4.6	212	5.3	252	6.3	227	5.7
350	233	5.8	211	5.3	244	6.1	278	7.0	265	6.6
400	267	6.7	248	6.2	271	6.8	319	8.0	302	7.6
450	318	8.0	291	7.3	302	7.6	344	8.6	331	8.3
500	338	8.5	327	8.2	349	8.7	381	9.5	378	9.5
550	346	8.7	355	8.9	391	9.8	410	10.3	399	10.0
600	362	9.1	379	9.5	428	10.7	452	11.3	428	10.7
650	369	9.2	395	9.9	451	11.3	479	12.0	439	11.0
700	375	9.4	418	10.5	478	12.0	496	12.4	447	11.2
750	379	9.5	429	10.7	492	12.3	508	12.7	455	11.4

PIT 3 (LIME + ASH)										
	0%		2%		4%		6%		8%	
Pen.	DGR	Load	DGR	Load	DGR	Load	DGR	Load	DGR	Load
50	32	0.8	41	1.0	33	0.8	109	2.7	97	2.4
100	53	1.3	76	1.8	69	1.7	147	3.7	139	3.5
150	98	2.5	110	2.8	102	2.6	187	4.7	181	4.5
200	132	3.3	154	3.9	158	4.0	220	5.5	219	5.5
250	177	4.4	188	4.7	220	5.5	256	6.4	257	6.4
300	218	5.5	215	5.4	275	6.3	288	7.2	295	7.4
350	256	6.4	241	6.0	299	7.5	316	7.9	338	8.5
400	300	7.5	275	6.9	324	8.1	341	8.5	366	9.2
450	344	8.6	319	8.0	357	8.9	374	9.4	402	10.1
500	395	9.9	355	8.9	381	9.5	419	10.5	441	11.0
550	424	10.6	374	9.4	415	10.4	438	11.0	472	11.8
600	447	11.2	392	9.8	433	10.8	477	11.9	497	12.4
650	469	11.7	420	10.5	461	11.5	491	12.3	533	13.3
700	483	12.1	452	11.3	489	12.2	520	13.0	568	14.2
750	501	12.5	479	12.0	501	12.5	543	13.6	581	14.5

APPENDIX B3 - UNCONFINED COMPRESSIVE STRENGTH RESULTS

Initial diameter of the sample = 38mm; Proving Ring Factor = 0.025

Percentage (%)	DGR	Length (mm)	Strain	Load DGR	Force (KN)		Corrected Area(A)	Stress (KN/m2)
PIT 1 (LIME)								
0%	100	76	1.0	11	0.275	1.316	8.6204	239.2
2%	50	76	0.5	5	0.125	6.579	1.1478	108.0
4%	150	76	1.5	7	0.175	1.974	1.1571	151.2
6%	100	76	1.0	7	0.175	1.316	1.1494	152.3
8%	100	76	1.0	5	0.125	1.316	1.1494	108.8
PIT 2 (LIME)								
0%	150	76	1.5	12	0.300	1.974	1.1571	259.3
2%	150	76	1.5	11	0.275	1.974	1.1571	237.7
4%	100	76	1.0	17	0.425	1.316	1.1494	369.7
6%	100	76	1.0	19	0.475	1.316	1.1494	413.1
8%	100	76	1.0	12	0.300	1.316	1.1494	261.0
PIT 3 (LIME)								
0%	150	76	1.5	19	0.475	1.974	1.1571	410.5
2%	50	76	0.5	6	O.150	6.579	1.1571	129.6
4%	100	76	1.0	5	0.125	1.316	1.1494	108.8
6%	50	76	0.5	5	0.125	6.579	1.1571	108.0
8%	100	76	1.0	8	0.200	1.316	1.1494	174.0
PIT 1 (ASH)								
2%	100	76	1.0	11	0.275	1.316	1.1494	239.3
4%	50	76	0.5	12	0.300	6.579	1.1418	262.7
6%	100	76	1.0	23	0.575	1.316	1.1494	500.3
8%	100	76	1.0	15	0.375	1.316	1.1494	326.3
PIT 2 (ASH)								
2%	100	76	1.0	17	0.425	1.316	1.1494	372.2
4%	50	76	0.5	17	0.425	6.579	1.1418	372.2
6%	50	76	0.5	14	0.350	6.579	1.1418	306.5
8%	150	76	1.5	19	0.425	1.974	1.1571	410.5
PIT 3 (ASH)								
2%	150	76	1.5	13	0.325	1.974	1.1571	280.9
4%	100	76	1.0	12	0.300	1.316	1.1494	261.0
6%	100	76	1.0	14	0.350	1.316	1.1494	304.5
8%	50	76	0.5	16	0.400	6.579	1.1418	350.3
PIT 1 (LIME + ASH)								
2%	100	76	1.0	6	0.150	1.316	1.1494	130.5
4%	50	76	0.5	13	0.325	6.576	1.1418	284.6
6%	150	76	1.5	15	0.375	1.974	1.1571	324.1
8%	50	76	5.0	9	0.225	6.579	1.1481	197.1
PIT 2 (LIME + ASH)								
2%	50	76	0.5	10	0.250	6.579	1.1418	218.9
4%	100	76	1.0	10	0.250	1.316	1.1494	219.0
6%	150	76	1.5	15	0.375	1.974	1.1571	324.1
8%	100	76	1.0	11	0.275	1.316	1.1494	239.3
PIT 3 (LIME + ASH)								