STABILISATION OF TROPICAL SOIL WITH HYDRATED LIME AND PLASTIC BLEND

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MEng/SIPET/2018/8019

DEPARTMENT OF CIVIL ENINEERING

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ABSTRACT

Effective utilization of locally available deficient soils by imparting additional strength using stabilising agents enable reduction in construction cost and improved performance for engineering projects. This work focuses on stabilisation of lateritic soil with hydrated lime and plastic blend using appropriate additions. The tests were carried out in accordance with the test procedures specified in BS 1377: 1992. This is done to determine the effect of the additives on the index and engineering properties of the treated soil being tested. From the results, the soil was classified as A-7-6 and CL according to AASHTO and USC systems respectively. It is a poorly drain soil with a liquid limit of 47% and requires a form of stabilisation before it could be used for construction purpose. On treatment with the blend of the additives, the plasticity index of the soil reduced from 20.86 to as low as 1 at 6% lime + 3% plastic blend. It then increased to 2.2 at 6% lime + 4% plastic blend. On the compaction, there was an overall increase in the OMC from 15.3% to 19.5% and a decrease in the MDD from 1.92 g/cm^3 to 1.71g/cm^3 at 6% lime + 4% plastic blend. CBR result shows that there was a significant increase in the CBR value from 6.04% to 57.03% at 6% lime + 1% plastic blend and then decreased to 12.61% at 6% lime + 4%plastic blend. For the strength test, there was an increase from 388.4 kN/m^2 to a maximum UCS value of 792.2 kN/m^2 at 4% lime + 1% plastic blend additive without curing. The UCS value increased to 1397.24kN/m^2 at 4% lime + 1% plastic blend after 7 days curing, which satisfied the specification in Highway manual, Federal Ministry of Works. After curing the samples for 28 days, the maximum UCS obtained was 1723.2 kN/m^2 at 4% lime + 2% plastic blend from the UCS test after curing for 28 days.

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CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

Soil is a loose, unconsolidated inorganic material on the earth's crust which is formed by mechanical and chemical weathering of solid rocks (Maneeth *et al*., 2014). While the chemical weathering process of the soil causes decomposition of the rock minerals by oxidation, reduction, hydrolysis, chelation and carbonation, the physical weathering cause disintegration of the rock mass (Alayaki *et al*., 2015). Tropical soils have properties and behaviour that are different from sedimentary soils due to the diversity of the environment of formation. 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. Red tropical soils are formed from the decomposition of parent rock materials like basalt, sandstone and schist under warm and humid climate of the tropics. The nomenclature proposed by researchers in their-desire to precisely define tropical soils has increased both in size and complexity making data gathered on these soils underutilized. Such terms as laterite, läteritic soil, latosol, ferricrete, ferruginous soil, ferrisol, oxisol and ferrasol have been used for the same type of material in one case and applied to various types of materials in other cases (Nwakanma, 1979).

Laterite is a soil type which is rich in iron and aluminium and is commonly considered to have formed in hot and wet tropical areas. Nearly all laterites are of rusty-red colouration, because of high iron oxide content. In tropical parts of the world, lateritic soils are used as a road making material and they form the subgrade of most tropical roads. They are used as sub-base and bases for low-cost roads and these carry low to medium traffic (Onyelowe, 2016).

In order the solve the challenge of weak soils in construction works, soil stabilisation is more economical way of handling weak soils, reducing the cost of weak soil replacement. Stabilisation of soil may be effective to improve the soil properties rather than removing and replacing the material which is usually more expensive depending on the haulage distance of material for replacement (Shiva *et al.*, 2016).

Soil stabilisation involves the use of stabilising agents (binder materials) in weak soils to improve its strength and other properties. Regular chemical stabilisation agents are Portland cement, lime, asphalt, biomass ashes, calcium chloride, sodium chloride, and paper mill waste. Soil conditions, stabiliser properties, and construction types (roads and houses) decide the adequacy of these added substances (Etekume and Onwualu, 2020).

Cementation is the main aim of the process referred to as 'soil stabilisation' used in practical geotechnical applications to improve the properties of the soil in terms of strength, stiffness and hydraulic behaviour. One of the main functions of the stabilising medium is to reduce the swelling properties of the soil. The purposes of stabilisation of soil include, to; a) increase the wet strength of the soil, b) provide adequate cohesion, c) increase volume stability, d) increase durability, resistance to erosion and frost attack, e) lower permeability (Bryan, 1988).

Kassa *et al.* (2020) Studies, show that incorporating the waste plastic strip as reinforcement into the soil serves as tensile inclusions in expansive soil to increase the resistance to shear, CBR value and reduction in swelling. Effective utilization of local weak soils by imparting additional strength using stabilisation materials enable reduction in construction cost and improved performance for roads (Vora *et al*., 2018). Subgrade stabilisation of clayey soil was found to make the upper part of road structure become stable and decrease pavement thickness. Soil stabilisation has also been used in erosion control and extensively used in the construction of foundations of buildings, roads, airfields, earth dams and embankments.

1.2 Statement of the Research Problem

Tropical soil in its natural state may have low bearing capacity and low strength due to high content of clay. Activity (A) value greater than 1.4 makes the soil an active clay containing montmorillonite which can result in very weak bonding and high swelling when in contact with water (Achmad *et al*., 2016). This may result to cracks and damage of pavement and the surfacing, low bearing capacity in building foundations or any other civil engineering construction projects. The large amount of plastic waste is being generated and causing nuisance to the environment (Ashraf *et al*., 2011). The increasing cost of cement, lime and bitumen, as stabilising agents necessitate the search for cheaper and locally available alternatives. Hence, this project involves a study on the possible use of a blend of lime and plastic material for stabilisation of deficient soils.

1.3 Aim and Objectives of the Study

The Aim of this research is to stabilise the soil with the addition of a blend of lime and plastic materials.

These Objectives include determining:

i. The index properties and compaction characteristics of the natural soil and soil treated with lime - plastic blend at 2, 4, 6 and 8% of lime admixed with 1, 2, 3 and 4% of plastic waste each.

ii. To determine the strength characteristic of tropical soil treated with lime –plastic blend using the chosen percentages.
iii. To determine the microstructural properties of the soil samples.

1.4 Justification of Study

The need to improve the strength and durability of weak soils in recent times has become imperative; this has geared researchers towards using stabilising materials that can be sourced locally at a very low cost (Bello *et al*., 2015). Stabilising weak soils with a blend of plastic and lime will considerably enhance the properties of the soil utilized in the development of road infrastructure as results shows a stronger and longer lasting road with inflated loading capability and reduced soil porousness. This will also provide an alternative and beneficial re-use of plastic waste thereby reducing the nuisance caused to the environment. It may be effectively utilized in strengthening the soil for road embankments and in getting ready an appropriate base for the higher pavement structure. Since it will increase the bearing capability of soil significantly, the land use may be inflated. It will lower the building and maintenance cost whereas increasing the quality of its structure and surface.

1.5 Scope of Study

This research covers the determination of index and engineering properties of the tropical soil, treated with a blend of lime and plastic materials. This research on tropical soil sourced around suburbs of Minna, Niger State, shall be achieved through various laboratory tests, and evaluation of the results. The findings will enable the geotechnical engineers know the impact of this blend of additives on the performance and engineering properties of test soil.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 General Review

This chapter includes review of past related studies on the use of lime and plastic wastes for stabilisation of soils. Emphasis has been laid on the mechanism involve in lime – clay interactions, and the effect on the soil properties. Soils are generally stabilised to increase their strength and durability or to prevent soil erosion. According to Anzar (2017), stabilisation of soil for ground improvement is applicable in geotechnical engineering such as repair of failed slope, backfill for earth retaining structures, repair of failed slopes, thin layers of soil and sub-grade and landfill liners and covers. It was also stated that the objective of ground improvement aids the absorption ability of soil of generated shear stress through the reinforcement introduced which reduce the loads that can cause failure due to shear or excessive deformation. Randomly distributing reinforcement into the soil can help achieve stability and reliability of geotechnical structures.

2.2 Soils

Soils are described as multi-phase materials consisting of a solid, liquid and gaseous phase. The solid phase consists of a mineral fraction (usually silicates but also carbonates and metal oxides or hydroxides) and possibly some organic fractions. The resulting solid particles constitute the soil skeleton. Voids around these particles can be filled with water and/or air (Zoheir, 2015). Use is made of the term "red tropical soils" to explain broadly a soil which fits the definition of "laterite" given by Sivarajasingham *et al*. (1962), this includes 'highly weathered material rich in secondary forms of iron, aluminium or both.

Activity (A) value of soil greater than 1.4 means the soil is active with the clay mineral Montmorillinite (Achmad *et al*., 2016). The active soil can expand by several times its original volume when it comes in contact with water, resulting in very weak bonding and high swelling. Such material should be avoided or stabilised to improve the engineering properties (Achmad *et al*., 2016). The proportion of clay mineral (namely; kaolinite, montmorillonite, and illite) flakes (< 2 mm size) in a fine soil affects the state of the soil, particularly its tendency to swell and shrink with changes in water content. Activity values of clay minerals are presented in Table 2.1 and the classification of soil based on plasticity is also presented in Table F1 in the Appendix.

Activity value < 0.75 = Non active soil

Activity value of 0.75 – 1.25 = Normal soil

Activity value > 1.25 = Active soil

Table 2.1: Activity Values of Clay Minerals

Minerals	Activity value
Na - monmorillonite	4-7
Ca - monmorillonite	1.5
Illite	0.5 – 1.3
Kaolinite	0.3 – 0.5
Halloysite (Hydrated)	0.1
Calcite	0.2
Quartz	0

(Source: Skempton, 1953)

Lateritic soil can be defined as weathered tropical or sub-tropical leftover soil, generally covered with sesquioxide rich solidifications (Etekume and Onwualu, 2020). A Sesquioxide is an oxide containing three atoms of oxygen with two atoms of another element. For example, aluminium oxide is a sesquioxide. Many sesquioxides contain the metal in the +3 - oxidation state and the oxide ion. Examples include Al_2O_3 and Fe_2O_3.

A review of literatures on some Niger Delta Soil shows that some of the air-dried samples were physically strong and could not easily break unless some substantial effort was applied, while the individual grains of the other samples were relatively separated and broke easily without much force. The presence of higher amount of iron, calcium, and magnesium oxides explains why the former were strong from their hardening tendency compared to the latter soil samples with low amount of these oxides. These oxides in particular have been known to be responsible for the cementing effects in soil particles as reported by various authors showing how sesquioxides play an important role in the hardening and cementing tendency of soil (Alayaki *et al.*, 2015).

While more attention is usually given to the physical properties of soil during their analysis, the chemical properties which are the inherent properties that define the soil are rarely tested. The silica-sesquioxides property of the soil is one of such properties which are rarely tested in the application of the soil for construction purposes. According to Alayaki *et al*. (2015), earlier researches carried out on analysis of silica, aluminium and iron oxides to quantify the silica – sesquioxides ratio used the formula (SiO_2 / ($A1_2O_3$ + Fe_2O_3)) to group laterites. From this analysis the laterites were classified as;

a. True laterites if SR < 1.33
b. Lateritic soils if 1.33 ≤ SR ≤ 2.00
c. Non – Lateritic tropically weathered soils if SR > 2.00

Studies carried out on some selected soils in the Niger Delta revealed that not all the soils formed in the tropic (tropical soils) are indeed laterites. The silica-sesquioxide ratio (SR) was used to analyze these soils. This ratio is used because it is generally accepted parameter in the classification and specification of laterites and can be measured with some degree of accuracy in the laboratory. This ratio helps engineers to have a better understanding of the soils erroneously considered to be laterites soils. Recent review of literatures on this study reported the use of the molecular masses to normalize the individual oxides in the silica – sesquioxide ratio (SR) formula on specification for laterites in road pavement. This formula used in the study after reviewing previous works is:

Silica - Sesquioxide Ratio $$\frac{S}{R} = \frac{\frac{SiO2}{60}}{\frac{Al2O3}{102}+\frac{Fe2O3}{160}} \tag{2.1}$$

Results obtained from the study revealed that none of the five soils samples investigated could be classified as laterite or lateritic irrespective of the colour. The sesquioxide ratios were all above the specified limit of 2 as generally agreed by various researchers. It is of interest to note that the soils that could be attributed to be laterite based on the reddish or brownish colour have very high sesquioxide ratios: an indication that they are not (Alayaki *et al.*, 2015).

Usually 90 – 100% of iron, aluminium, titanium and manganese oxides makeup the oxide composition of laterite soil. Laterites contain lower percentages of nitrogen, phosphorus, potassium lime and magnesia and are usually reddish to yellow in colour. These laterites are usually formed under high temperature and heavy rainfall condition with alternate wet and dry periods leading to leaching of the soil, thereby leaving oxides of iron and aluminium. The hardening of laterite like iron formed in this condition and how easily it can be cut with spade especially when it is to be used for construction purpose makes it a good source of building material (Kayastha *et al.,* 2017).

A study on the Geology of Nigeria described laterites to consist of three layers, basal lateritic clay, middle laterite gravel and a surface laterite crust. The types of Laterites are thus given as:

(i) Laterite crust: Laterite crust has a cellular texture and is usually hard to break with a geologists' hammer. Light explosives may be required to excavate this type of laterite. It is commonly found on top of flat-topped hills or as boulders on slope surfaces and often is encountered while digging building foundations.

(ii) Laterite gravel: Laterite gravel may be found below a layer of laterite crust. At some locations, the gravel deposit is only covered by a thin layer of soil. Laterite gravel is usually pisolitic.

(iii) Laterite Clay: Laterite clay is often located below the gravel or the crust, and usually above the weathered basement. It has a very rich reddish-brown colour, with patches of pinkish white material (probably Kaolinite) (Obianyo and Onwualu, 2017).

Lateritic profiles are very complex and most of the times are polygenetic. As a result of this, no definition or classification exist for laterite generally. A standard lateritic profile includes a soft lateritic layer at the bottom, a hard one at the top and one that is overlay by a gravel-rich layer (Marcelino and Stoops, 2018). Some of the physical properties to observe about laterites include the reddish-brown colour and gravelly nature of its texture. This colour well becomes predominant and much visible when wet and becomes distinct when dry (Zolfeghari *et al.*, 2013).

Expansion and contraction associated with high and low temperatures which occur during the dry and wet seasons respectively, leave cracks in huge masses of rocks through which warm surface water rich in carbon dioxide and bacteria filter. During the dry season they can easily be seen as deep cracks of polygon patterns. This chemically active water decomposes the rocks (aluminous silicates). An upward suction of moisture to the surface-in the dry season and drainage of rainwater over the gentle and moderate relief of the tropics remove the dissolved minerals of the parent rock in solution. This process of relative accumulation is called leaching. The residuum comprises minerals like aluminium, iron, titanium, manganese, chromium and vanadium oxides, quartz and clay minerals (kaolinite, illite and montmorillonite). The types of minerals depend on the parent while their nature depends on other soil forming factors like topography, drainage and climatic - vegetational conditions (Nwakanma, 1979). Silica occurs as quartz which is formed at lower silica content in the solution, which can-be residual or alluvial, and in the amorphous form. The clay mineral that occurs most commonly in the soils is the 1:1 type, such as the kaolinite and/or halloysite (Harder, 1977). Flakes of micas are visible in hand specimens and often are used in the construction of earth dams (Nwakanma, 1979). In order to make this weak and expansive soil useful there is the need to stabilise them for improvement of the engineering properties of the soil.

2.3. Soil Stabilisation

Soil stabilisation can therefore be defined as any process which improves the engineering properties of soil, such as increasing shear strength and bearing capacity. Shear strength is the magnitude of the shear stress that a soil can sustain. The shear resistance of soil is derived from the friction between particles or bonding at particle contacts while bearing strength can be defined in geotechnical engineering as the capacity of soil to support the loads applied to the ground without causing failure. Thus, bearing capacity of soil is the maximum average contact pressure between the foundation and the soil which should not produce shear failure in the soil. Soil stabilisation can be accomplished by several methods. These methods/techniques can broadly be classified into three types, namely:

i. Mechanical Stabilisation: Mechanical Stabilisation is a physical process that involves altering the physical nature of native soil particles by either induced vibrations or compaction, or by incorporating other physical properties such as barriers and nailing. The oldest types of soil stabilisation were found to be mechanical in nature. Some examples of this type of stabilisation are the Dynamic compaction of soil which involves the use of heavy weight dropped repeatedly on the surface of the soil at regular intervals to ensuring a uniformly packed surface. Another is the use of Vibro Compaction which relies on vibration rather than deformation through kinetic force to stabilise the soil. Different types of soils materials (well-graded and uniformly graded) when mixed together improves its strength properties. In mechanical process, blending good quality soil in a required proportion is used to improve the properties of subgrade soil. After this, spreading and compaction of blended soil is done (Arif *et al.*, 2019, Singh and Khan, 2020).

ii. Chemical Stabilisation: Chemical Stabilisation involves initiating chemical reactions between stabilisers (cementitious material) and soil minerals (pozzolanic materials) to improve the properties of the soil (Ingles and Metcalf, 1972). In this case, additional materials are added to the soil which interacts with the soil physically and change its properties. Chemical additives frequently used for stabilisation include cement, lime, fly ash, or kiln dust (Arif *et al.*, 2019, Singh and Khan, 2020). Chemical treatment can affect clay structure in terms of both fabric changes and bonding (by the creation of artificially caused 'cementation' bonds between particles).

iii. Polymer/Alternative Stabilisation: Most of the newer discoveries and techniques developed thus far are polymer based in nature such as processed polymer fibre or wastage materials such as polythene bags, plastic bottles, recycled plastic pins. The use of polymer materials is cheaper and more effective in general than mechanical solutions. They are significantly less dangerous for the environment than many chemical solutions. Also, plastic strips are inert and degradable so it effectively remains in soil for many years (Jasmin *et al*., 2016, Singh and Khan, 2020).

The use of polymers such as plastics has been found to be economical in stabilising soils. These techniques can as well serve the purpose of reducing pollution and producing useful material from non-useful waste materials (Madavi and Patel, 2017). Expansive soils such as black cotton soil create problem in foundation and for this stabilisation of soil is required (Singh and Dixit, 2017). The essential principles of soil stabilisation according to Lande *et al.* (2020) include the following:

i. Studies of the natural soil properties before it is treated.
ii. Base on the physical properties of the soil obtained, the most effective and economical method of stabilisation is selected for the soil.
iii. Result obtained from the laboratory tests conducted on the soil are then analysed for the intended stability and sturdiness value expected.

Due to the bad condition of most soil mass used for construction and their effect on roads and foundations of engineering structures, Kumar *et al.* (2018) in a study concluded that in order to improve the strength of clayey soil waste plastic materials can be used. The results obtained shows that the specific gravity of the soil increased from 2.33 of the natural clay to 2.35 at 0.6% plastic waste, 2.37 at 0.8% plastic waste and 4 at 1% plastic waste. Also, it was observed from the same studies that both the MDD and the OMC of the soil increased with increase in the percentage of waste plastic added to the soil.

Sagar *et al.* (2019) carried out study research of various methodologies and experimental investigations needed to improve different soil properties and discovered from this study the problems of waste plastics in the environment. this discovery led to the recommendation of using plastic waste for stabilising soil as this would significantly reduce the troubles of disposing these wastes and improve properties of the stabilised soil. Also, Kumar *et al.* (2017) carried out a review work of various researches o stabilisation of soil using plastic and bottle strips materials in improving its strength. It was observed that the as the length of the plastic strips increased, CBR values increased to 6.20 at an optimum length of 5.0cm which was the optimum length of strips used for sub grade design. From the results obtained shear strength of the soil increases up to 5cm length of the strips and after which there was a decrement.

Lande *et al.* (2020) in a review of the performance of plastic fibre as a soil stabilisation material, the replacement of 0.5% plastic fibres to the expansive clayey soil reduce its OMC and increased the Maximum Dry Density. At 0.5% fibre increase in the unconfined compressive strength of the soil was observed. With 1% replacement it was observed that the MDD and UCC was less than the 0.5% replacement but was greater than the untreated soil. Further increase in the plastic replacement showed decrease in the MDD and the Unconfined Compressive Strength of the soil. Based on the non-problematic soil criteria, the optimum percentage of plastic is recommended as 0.5% which will enhance the engineering properties of the silty clay.

A study carried out using plastics such as shopping bags shows that inclusion of plastic strips in soil in appropriate amount has improved the strength and deformation behaviour of sub grade soils substantially (Arpitha *et al.,* 2017). Lande *et al.* (2020) conducted a study which involves the use of plastic bottles in the form of plastic strips. The author used both red soil and black cotton soil which were collected from the flexible pavement. The composition of soil is as follows- 4% gravel, 88% sand and 8% silt and clay. After experimentation it was found out that the soil has a maximum dry unit weight of 20.12kN/m^3 and 20.03kN/m^3 and an optimum moisture content of 14% and 11%

respectively was found out under standard proctor and modified proctor condition. Black cotton soil comprises of 2.6% gravel, 15.1% sand and 82.3% silt and 0.18% clay. It was found out that the soil has a maximum dry unit weight of 15.56kN/m^3 and 18.33kN/m^3 and an optimum moisture content of 13.63% and 10.78% under standard proctor and modified proctor condition respectively.

2.4 Plastics

Plastics are light weight, less expensive and durable materials which can readily be moulded into a variety of products especially in the fluid/processing state. Some of the properties of plastic wastes which makes them good alternative for soil stabilisation include; higher ductility, impervious to water and movement of flow, high tensile strength and their relatively low cost and ease of manufacture. Based on data available on Municipal Solid Waste (MSW) 2009, approximately 4000 - 5000 tons of plastic wastes are generated per day. Several million metric tons of plastic wastes are produced every year, and one method to reduce the plastic waste disposal problem is by recycling and utilizing them in the stabilisation of expansive soil. According to Awuchi (2019), an estimated average of 15.4 billion pieces of plastic wastes are generated per day. The leaching of plastics due to acidic environment of soil affects the organisms. So, there is a need to develop new methods to dispose the plastic materials. For disposing of plastic waste, we can use plastic waste for stabilisation of soil (Singh and Khan, 2020). The use of locally available plastic wastes is therefore encouraged for sustainable development. This is economical, enhances environmental balance and avoid waste plastic disposal problem (Sharan and Mahabir, 2017). Anzar (2017) concluded that stabilising soil with plastic strips not exceeding 5% (as the benefit of reinforcement as concluded increases to a level and after that it decreases the strength) is economical as these wastes are cheaply available and the engineering properties of the soil were improved like compressive strength, tensile strength and shear strength. Plastics generally are available in different forms as presented in Table 2.2.

Table 2.2: Origin of Plastics

Plastic	Used as
Poly-Ethylene Teryphthalate (PET)	Drinking water bottles
High Density Polyethylene (HDPE)	Carry bags, bottle caps and house-hold articles.
Low Density Polyethylene (LDPE)	Milk pouches, sacks, carry bags, bin linings, cosmetics and detergent bottles.
Polypropylene (PP)	Bottles caps and closures, wrappers of detergent, biscuit, wafers packets and microwave trays for meal.
Polyester resin	Casting, bonding fibres (glass, Kevlar, carbon fibre)
Urea formaldehyde	Electrical fittings, handles and knobs

(Source: Maneeth *et al.,* 2014)

Plastics are versatile, light, hard, and resistant to chemicals, water and impact. It has been proven that the use of plastic bottles, as innovative materials for building can be a proper solution for replacement of conventional materials (Jalaluddin, 2017). Also, use of plastic strips helps to prevent cracks, potholes and wheel path rutting in pavement, improves frictional resistance and ductility hence can be used for stabilising slopes and embankment, assist in improving compressive strength and reducing settlement, hence can be used for foundations.

The plastic has many characteristics properties like strength, brittle, durability, corrosion resistant, resistance to chemical attacks, insect attacks and abrasion resistant, insulating properties, heat resistant. For disposing of plastic waste, we can use plastic waste for stabilisation of soil. Thus, using plastic as a soil stabiliser is economical and gainful use in construction due to poor quality soil for various constructions, minimizing the amount of plastic waste and producing useful product from non-useful waste materials for reliable foundation and subgrade improvement. This new technique of soil stabilisation can be effectively used to solve the challenges of society and enhance the quality of soil used in construction of road infrastructure, foundation, stabilisation of embankment. It was suggested from this study that further research be done to determine boundary effects influence on test result and better effectiveness (Sagar *et al.,* 2019). Plastics are made by linking many monomers together to form a polymer. The stabilisation done by using plastic wastes or plastic materials are similar to aggre bind soil stabilisation. Aggre bind soil stabilisation is a unique, environmentally friendly, cross linked, water based, styrene acrylic polymer with proprietary tracers. aggrebind soil stabiliser is used to produce the roads from in-situ materials and for manufacturing soil stabilised blocks, bricks and pavers for buildings, homes without using any cement (Saravanaganesh *et al.,* 2016).

From a study carried out using plastic strips of various sizes, it was concluded that the MDD value of the soil decreases while the OMC increases with increase in the plastic strips. The maximum CBR value was obtained at 0.8% plastic strips of dry weight of soil.

It was concluded therefore that 0.8% of strips having length of 2cm is considered as required amount. (Kumar *et al.,* 2018).

Research by Rasul *et al.* (2018) showed that the stabilisation with either cement or lime or a combination of both resulted in a significant increase in UCS and Resilient Modulus (Mr) values as the stabiliser was increased. Research conducted by Hassan *et al*. (2021) shows that the stabilisation of soil with fibre increases the UCS to a specified ratio of fibre content beyond which the UCS decreases, but increased the Resilient Modulus (Mr) values. This kind of observation from this study explains a major difference between using chemical agents and fibres for stabilisation, in which it can be decided to choose the most suitable stabiliser for the corresponding required properties. For cases where the resilient modulus is an essential property to improve in the subgrade soils fibre can be used, whereas in a situation where other purpose are considered or the UCS is an important property in consideration a chemical agent would be more favourable. For those road pavements design codes of practice that use the CBR and Resilient Modulus (Mr) as design parameters, stabilising the soil with fibre is cost-effective and it can be used successfully for a sustainable road construction when compared with chemically stabilised soils. One of the advantages of fibre in soil stabilisation over chemical additives in stabilisation is that the chemical agent is accompanied by carbon dioxide emission, while fibres are not.

According to Lande *et al.* (2020) on "Soil Stabilisation Using Plastic Waste", Modified Proctor Test was recommended for test the soil rather than the Standard Proctor Test in a situation where the soil to be tested is intended to be used for road construction because it requires high compaction. From several researches and literature surveys on the use of plastic wastes, results show that soils stabilised with plastic bottles has less settlement and high ultimate bearing capacity than the plain soil. According to Kumar *et al.* (2017), the strength of the soil tested was increased on addition of 0.7% plastic strips to red soil and 0.5% plastic strips to black cotton soil. Similar results were said to have been obtained ranging from 0.6% to 0.75% optimum plastic content from past related works and plastic was used in different forms such as strips, powder and sheets (Lande *et al.,* 2020).

According to Gardete and Luzia (2020) plastic wastes can be available as flakes or fibres which affects the strength properties of the soil. The size and shape of plastic waste particles has influence on result achieved. Waste plastic fibres, strips or flakes have different interaction with the soil matrix by the properties and type of soil concerned. Soils treated with plastic wastes usually show increase in the CBR values obtained and the percentages are usually at best 2% but often in the 0.5 to 1.5% range. Some researchers refer that dry unit weight can increase with plastic waste content until a maximum is achieved, decreasing subsequently for higher contents. The author mentioned that wastes stabilised soils can have limitation for reuse and disposal at the end of the life of the infrastructure when compared to virgin soil. Making use of waste plastic materials for stabilisation of soil is a better technique for reuse the waste as it helps to reduce the various social challenges like reducing the quantity of waste, producing useful materials from non-useful waste materials. (Shekar, 2021)

Tarun *et al.* (2018) in a study on Behaviour of Soil by Mixing of Plastic Strips concluded that the MDD decreases with increase in the amount of plastic content while the OMC increases. The maximum CBR value was obtained at 0.8% plastic strips and this was considered as the required amount for stabilising the soil tested.

Attempts have been made to improve the soils of road using high density polyethylene (HDPE) strips as reinforcement for improving the engineering properties of the subgrade soil show that the reinforcement benefit is directly proportional to the length of the strip content. (Choudhary *et al.*, 2010). Chebet and Kalumba (2014) conducted a laboratory investigation making use of a randomly mixed f strips of HDPE (high density polyethylene) material from plastic shopping bags to determine its effect on the shear strength and bearing capacity of locally available sand and observed that the increase in strength for the reinforced soil can be attributed to the tensile stresses mobilised in the reinforcement. The factors identified to have an influence on the efficiency of the reinforcement materials were found to include properties of the plastic such as the length, concentration and width of the strips while that of the soil include gradation, shape and particle size.

Dhatrak *and* Konmare (2015) after carrying out series of experiment on a soil mixed with different percentages of plastics in proportions of 0.5,1, 1.5, 2 and 2.5% concluded from findings made that plastic waste strips will improve the soil strength. From studies carried out on soil using a mix of glass and plastic granules, it was concluded that addition of these will result in decrease of the MDD and increase in the OMC. The MDD of 1.53 gm/cc was obtained at 6% of glass and plastic. Maximum OMC was obtained as 22.6% at 6% mixing of additive. The UCS value increased from 0.609 Kg/cm^2 to 3.023 Kg/cm^2 which are about 5 times as that of virgin soil. Maximum CBR value was 7.14 %, about 2 times that of the natural soil. (Subhash, 2016).

Saravanaganesh *et al.* (2016) from the studies carried out concluded that plastic granules materials made by recycling the polypropylene plastic bags were used for the stabilisation of the soil which acts as a reinforcing material and resists the entry of water when mixed with varying proportions of the plastic granules. As the percentage of plastic granules increases, the strength of the soil also increases. The collected samples used are granules made up of polypropylene plastic which is 300 mm to 500 mm. According to Saravanaganesh *et al.* (2016) in a study carried out, the soil was said to behave as a reinforcing material when blended with the plastic granules and that the soil fill will be strengthened by relatively stiff high strength plastic metallic inclusions. In this studies, addition of plastic granules resulted in increase of optimum moisture content and CBR value. On the whole the study reveals that red soil can also be used as a sub grade soil.

On treating black cotton soil using glass and plastic mix, there was a reduction in the Free Swell of the soil up to 1.5 times than that of the untreated soil. There was an increase in the Soaked CBR value of the soil from 0.51% to 1.2% by the addition of Glass and Plastic at an optimum percentage of 4% which is almost 2.3 times. For the case of the UCS result, the Ultimate Bearing Strength of soil increased from 705.62 kN/m² to 1327.93kN/m². From the overall result obtained from the various tests conducted on the soil, effective stabilisation of the soil can be obtained at 6% Glass and Plastic mixed with the soil. Hence this was taken as the optimum percentage of Glass and Plastic for stabilising the black cotton soil (Paul *et al.,* 2019).

Poweth *et al.* (2013) on carrying investigation on a safe and productive way to dispose quarry dust, tyre waste and wastes-plastic arrived at a conclusion that they can be useful for stabilising pavements sub grade. Series of tests were conducted and the results shows that only quarry dust should be mixed with the soil plastic mix, to increase its maximum dry density and is suitable for constructing pavement sub grade. Tyres alone are not suitable for sub grade. Therefore, it was concluded that Soil plastic mixed with quarry dust maintains the CBR value within the required limit and can therefore be used to stabilise the soil

On making use of plastic waste pieces to treat clayey soil and sandy soil at a mixing ratio of 0, 2, 4, 6 and 8%, a significant improvement was observed in the strength property of the soils used in the experiment and decrease in MDD and OMC of the soil was attributed to the low specific gravity of plastic pieces used in the study (Nsaif, 2013). From a study carried out on treatment of a red mud with fly ash and plastic waste of different percentages based on light compaction tests, it was concluded by the authors that the MDD increased as the plastic increased up to 2% and decreased beyond this percentage while the OMC remained the same in each case. Hence from this result, it was concluded that pavement thickness can be reduced by addition of waste plastic content up to 2% (Paramkusam *et al.,* 2013).

CBR studies were carried out to find the variation in the strength characteristics of the soil stabilised with Glass and Plastic. When Glass and Plastic is added to the soil, the strength of the soil is increased initially. The increase in Glass and Plastic beyond an optimum percentage in soil caused a decrease in strength. Increase in the CBR value of the soil is due to the densification achieved by the filling of voids in soils with the Glass and Plastic. When the stabiliser content is increased beyond the optimum percentage (6%) there is a decrease in CBR value. This decrease may be caused by the adsorption of water by Glass and Plastic thus acting as a cushion in the soil and not providing enough water molecules to hold the soil particles together. A maximum CBR value of 7.14 was obtained at 6% of stabiliser beyond which there is a decrease of CBR value (Hassan *et al.,* 2021).

Laboratory test results from the use of polyethylene (PE) bottles and polypropylene (PP) in form of fibres in proportions of 1, 2, 3 and 4% of the soil weight revealed that the plastic pieces decrease maximum dry density (MDD) and optimum moisture content (OMC) of the stabilised soils, which are required for the construction of embankments of lightweight materials. In addition, there was a significant improvement in the UCS of soils by 76.4 and 96.6% for both lengths of PE fibres and 57.4% and 73.0% for both lengths of PP fibres, respectively. Results of the CBR tests demonstrated that the inclusion of plastic fibres in clayey soils improves the strength and deformation behaviour of the soil especially with 4% fibre content for both lengths 1.0 cm and 2.0 cm, respectively, to a figure of 185 to 150% for PE and PP, respectively (Hassan *et al.,* 2021).

Achmad *et al.* (2016) studied two soil samples R2 and R24 collected from various sites of Kuantan. Waste cutting HDPE and crushed waste glass were used as additives. The variations of additive contents were 4, 8 and 12% of total dry weight of soil sample respectively. Result of the evaluation showed that on addition of waste HDPE and glass there was an increase in the Plasticity Index (PI), about 10% for R24 and 2% for R2 samples respectively. The value of Optimum Moisture Content (OMC) decreased, while Maximum Dry Density (MDD) increased with increase in the additives. There was also an increase in California Bearing Ratio (CBR) value.

Ilies *et al.* (2017) in the research carried out, studied the mechanical behaviour of a silty soil that was reinforced with aleatorily distributed Polyethylene Terephthalate (PET) plastic fibre and discovered from tests conducted that the reinforced soil had a greater deformation capacity and hence can be used in structures that require a high deformation capacity such as landfills, sewage treatment deposits and dams to prevent failures due to cracking and possible leakage of contaminants or water. According to Ilies *et al.* (2017) it was also noted that the production of polyethylene grains as a stabiliser has a lower carbon footprint than cement or other hydraulic binders.

Shiva *et al.* (2016) observed that the UCS value did not change much with 0% to 0.05% plastic strips and when compared with 0.10% and 0.15%, attributing this to the fact that there was no significant increase in the length of plastic added and recommended that the length of the plastic strips be further increased which provides better integrity to the mould. Shiva *et al.* (2016) also concluded from the studies carried out using different percentages of 0.05%, 0.1%, 0.15%, and 0.2% plastic strips to the dry weight of soil that the unconfined compressive strength for Black Cotton soil is increased due to inclusion of plastic waste strips up to 0.2% but the changes was not large as expected. It was concluded from the study that this reduced increase in strength may be due to loss of integrity in soil-fibre system slippage between fibres.

Plastic bottles made of Polyethylene Terephthalate (PET) are un-decomposable and destructible. When melted it releases a compound gas which is very harmful to health and environment (Kamal and Anupam, 2019). Medical experts have remotely opined that these toxic gases overtime can cause cancer, high blood pressure and Asthma. The use of plastic cannot be completely stopped but we are able to recycle and reuse it in many ways and thereby causing minimum debilitating effect on the environment.

Poweth *et al.* (2014) investigated the effect of plastic granules on weak soil sample. The percentage of waste plastic was taken as 0.25, 0.5 and 0.75%. Maximum dry density was obtained when 0.25% plastic was added, and OMC obtained for the treated soil sample was less than that of the soil without plastic for this percentage. Further, CBR value decreases when 0.25% plastic was added but it was found to be increased for 0.75% of plastic. Kamal and Anupam (2019) on investigation of "Soil Stabilisation Using Plastic Waste" concluded that, 1% plastic strips (5mm x 3mm) of the total weight of the soil is the optimum proportion to be added to the soil for reinforcement.

Sai and Venkata (2019) performed an experimental study to investigate the stabilisation effect of waste plastic granules material on soil. Results of tests demonstrated that inclusion of different percentage of waste plastic and plastic granules (0.5, 1, 1.5 and 2.0%) in soil with appropriate amounts improved strength and deformation behaviour of sub grade soils substantially. For the standard proctor test, the increase in maximum dry density occurs at 0.5% of adding plastic waste and plastic granules. Results by CBR test concludes that the bearing capacity of the soil is increased at 1% of adding plastic waste and plastic granules. Result from UCS test concludes that the Compression strength of the soil was increased at 0.5% of adding plastic waste and plastic granules.

Ratna *et al.* (2018) studied the use of Plastic fibres with different proportions such as 0, 0.25, 0.5, 0.75 and 1% with respect to dry weight of soil and lime from 0% to 5%. Result showed there was an improvement in the properties of black cotton soil by adding 4% lime, and 0.75% plastic fibres. Various studies have been carried out by many researchers to find effective methods to reduce the pollution caused by plastic materials including recycling and reusing these materials in civil engineering applications as a solution to preserve the environment from the pollution of plastic waste materials. An effective method to utilise these materials is to be used as a soil stabiliser for road construction (Tatone *et al.*, 2018).

From the laboratory tests conducted in the research of including plastics waste into soil, data obtained revealed that the inclusion of plastic waste materials in soils decreases the OMC. This was attributed to the fact that plastics are not absorbent materials compared to clay soils, which have high affinity to water due to its surface tension. Also, there was no improvement in the MDD result after the addition of polyethylene (PE) which was due to the low specific gravity of polyethylene (PE). This can however still be an advantage for using polyethylene (PE) as a stabiliser to be one of the components for construction of embankments (Hassan *et al.*, 2021). On the other hand, making use of polypropylene (PP) as stabiliser at 3% and 4% fibre contents improved the MDD and this was attributed to the ease in mixing with soil, which behaves like multifilament fibre during the mixing action according to Olgun (2013). This study therefore concludes that polypropylene (PP) may be more efficient in terms of its impacts on distribution and bonding of soil particles than polyethylene (PE). The UCS results clearly showed that longer fibre lengths (2.0 cm) have higher increase in strength than shorter fibre lengths (1.0 cm). Many researchers such as Oliveira *et al.* (2018) have studied the effect of PE on UCS of soils with similar results. It is known that PE has higher tensile strength than PP; therefore, the UCS of soils stabilised with PE is higher than that of soils stabilised with PP (Hassan *et al.*, 2021).

Anand and Vageesh (2017) studied the Effect of Discarded plastic waste as stabiliser on engineering properties of cohesive soil. It was concluded that an optimum of 0.4% plastic waste from CBR test can be used to stabilise the black cotton soil. A combination of plastic waste and other stabilising agents (like cement and lime) at varying percentages was recommended from this study.

Kumar *et al.* (2018) in a study conducted using pieces of polyethylene (PE) plastics cut into 1.0 cm, 2.0 cm, and 3.0 cm lengths at various fibre contents of 0%, 0.2%, 0.5%, 0.8% and 1.0% of the dry weight of soil, found that as the length and content of the plastic increase, there was a corresponding decrease in the value of MDD. From the result obtained it was concluded that the highest reduction was found when plastic content was 1% of the dry weight of the soil and the optimum length of plastic strip inclusion in the soil was 3.0 cm. Taha *et al.* (2020) carried out investigation using polypropylene fibre (PF) 12.0 mm in length to determine its effect on the mechanical behaviour of clayey soils. The proportions of the fibre mixed with the soil include 0%, 1.5%, 2.25%, and 3% content by the soil weight. From the study it was observed and concluded that as the fibre content was increased, there was an increase in MDD and a decrease in OMC with an optimum fibre content of 3%.

For the soil stabilised with polyethylene PE, the addition of 1.0 cm increased the CBR values by 55% (from 4.0 to 6.2) and addition of 2.0 cm lengths of PE fibre increased it by 80% (from 4.0 to 7.2). Also, for the case of the soil stabilised with polypropylene PP, the addition of 1.0 cm and 2.0 cm lengths of PP fibre increased CBR by 42.5% (from 4.0 to 5.7) and 50% (from 4.0 to 6.0), respectively. From the results obtained from this study, it is clearly revealed that the fibre content and fibre length have significant effects on CBR values. The reason for the increase in CBR value with the inclusion of plastic fibre is generally considered to be due to the soil and fibre interactions which provide resistance to the penetration plunger; and eventually increases in the CBR value as resistance increases confirming the effect of the length and content of this fibre on the CBR value (Neopaney *et al.,* 2012). Madavi and Patel (2017) from investigations conducted also confirm this behaviour of fibres from the conclusion obtained of 4% optimum plastic content resulting in the highest CBR value. This increase in CBR values of subgrade soils can have significant impact on required foundation thicknesses, especially for those pavement design methods such as Design Manual for Roads and Bridges (DMRB) of Highway, in which the thickness of the pavement foundation is depending on the CBR and modulus of elasticity of subgrades. The increase in subgrade CBR and modulus of elasticity will usually reduce the required sub-base thickness considerably and results in the reduction in road pavement construction costs (Hassan *et al.,* 2021). From the conclusion of the experiment conducted using polypropylene and polyethylene, the increase in fibre content resulted in the increase in the values of CBR and Resilient Modulus but for the UCS values the fibre content was found to lies between 1% and 2% for both polypropylene PP and polyethylene PE.

According to Yaghoubi *et al.* (2016), increase in the fibre content used to stabilise a weak soil increases the interconnection between the particles of the soil and subsequently the strain to the applied stresses decreases. Studies show that reinforcement benefit is directly proportional to the length of the strips content of plastic waste as seen in a CBR test with the CBR values of the reinforced system being three (3) times that of the unreinforced system (Sagar *et al.*, 2019). According to Rather and Bhat (2021), some of the advantages of Plastic as a Soil Stabiliser are:

i. It improves the strength of the soil, thus, increasing the soil bearing capacity.
ii. It is a lot of economical each in terms of price and energy to extend.
iii. Bearing capacity of the soil instead of going for deep foundation or raft foundation.
iv. It offers more stability to the soil in slopes or other such places.
v. Sometimes soil stabilisation is also stop soil erosion or formation of mud, which is extremely helpful particularly in dry and arid weather.
vi. Stabilisation is also done for soil water-proofing; this prevents the seepage in soil and hence helps the soil from losing its strength.
vii. It helps in reducing the soil volume modification because of modification in temperature or wetness content.
viii. Stabilisation improves the workability and also durability of the soil.

2.5 Lime

Lime in terms of rock type is limestone. Quick lime, hydrated lime and lime putty are used in concrete or mortar (Neville, 1992). Amadi and Okeiyi (2017) conducted a laboratory study on a soil comparing the stabilisation effectiveness of different percentage (0, 2.5, 5, 7.5 and 10%) of quick lime and hydrated lime on lateritic soil. The study shows that Quicklime caused the soil to have plasticity 1.4 times lower than that produced by hydrated lime at 10% treatment. It yielded higher strengths and resulted in swell values about 2.2 times lower than the value observed for hydrated lime at 10% treatment. Although soil – hydrated lime mixtures yielded somewhat higher dry unit weights. In general, the results obtained from the investigation concluded that, quicklime exhibited somewhat superior engineering properties and therefore creates a more effective stabilisation alternative for the soil.

Lime has been mainly used for stabilising the road bases and the subgrade. Slaked lime is very effective in treating heavy plastic clayey soils and the Plasticity index of highly plastic soils are reduced by the addition of lime with soil. Lime may be used alone or in combination with cement, bitumen or fly ash. Sandy soils can be stabilised with these combinations. Lime changes the nature of the adsorbed layer and provides pozzolanic action. There is an increase in the optimum water content and a decrease in the maximum compacted density. The strength and durability of soil increases normally with 2 to 8% of lime for coarse grained soils and 5 to 8% of lime may be required for plastic soils. The amount of fly ash added as admixture may vary from 8 to 20% of the weight of the soil (Rather and Bhat, 2021).

In a system that requires large quantities of lime, quicklime would be preferred as the density is twice the density of hydrated lime. This reduces the storage and transportation costs. Where smaller quantities are required, hydrated lime is preferred because it is safer to use and less hazardous.

2.5.1 Chemistry of lime treatment

Lime is a common name used for calcium carbonate ($CaCO_3$), calcium hydroxide ($Ca(OH)_2$) and calcium oxide (CaO). These are commonly known as limestone, hydrated/slaked lime, and quicklime respectively. Chemical reactions in the production of lime is shown in equations (2.2) to (2.5).

Reaction for high calcium limestone is given as;

$$CaCO_3 + Heat \rightarrow CaO + CO_2 \tag{2.2}$$

Reaction for the hydration reaction of quicklime is given as;

$$CaO + H_2O \rightarrow Ca(OH)_2 \quad (2.3)$$

This is simply stated as follows:

$$Limestone + Heat \rightarrow Quicklime \quad (2.4)$$

$$Quicklime + water \rightarrow Hydrated\ lime \quad (2.5)$$

A wide variety of hydrated form of lime can be obtained in soil – lime reaction depending on the reaction conditions which include the following factors; quantity and type of lime, soil characteristics, moisture content, curing time and temperature. A typical soil – lime reaction is given as follows:

$$Clay + lime \rightarrow CSH\ (gel) + C_4AH_{13} + C_3AH_6 \quad (2.6)$$

Where C = CaO, S = SiO_2, A = Al_2O_3 and H = H_20 (Zoheir, 2015).

When lime is added to a soil, the following processes occur:

(a). Drying: This involves the immediate dehydration of the soil and release of heat where quicklime is used. In case of hydrated lime, it reduces the capacity of the soil to hold water and increases its stability while drying occurs only through the chemical changes in the soil.

(b). Modification: Hydrated lime (Ca $(OH)_2$), when mixed with soil in the presence of adequate moisture content, the divalent calcium (Ca^{2+}) ions and the monovalent hydroxyl (OH^-) ions will dissociate into pore solution and consequently increase the soil pH which favours the Ca^{2+} cations from lime with the monovalent cations (viz Na^+, K^+ and mg^+) present in the Diffused double layer (DDL) of negatively charged soil minerals. The increased Ca^{2+} concentration in pore solution also causes reduction of DDL thickness through cation exchange and flocculation – agglomeration reactions of clay particles. This is primarily responsible for improvement in workability through reduction of absorbed water. There is decrease in plasticity index and increase in bearing value of high percentage expansive clay soils. Summarily Ca^{++} which migrates to the surface of the clay particles reacts and displaces water and other ions. The soil becomes friable or more stable and granular making it easier to work and compact. As a result, the plasticity index PI drops instantaneously, so does its tendency to swell and shrink. There is also improved workability and immediate strength enhancement. The whole process is referred to as '**short-term-modification**' (Cherian and Arnepalli, 2015). Reduction in plasticity gives the soil-lime mixture a more friable texture, making the soil more amenable to movement and manipulation with field equipment.

(c). Stabilisation: Simultaneously, when adequate amount of lime and water are added to soil solution, the pH of soil-lime mixture is progressively increased to above 10.5 which enables the clay particles to breakdown and increase to 12.4 (approximately equal to that of saturated lime solution) by the dissolution of OH^- ions from lime. This highly alkaline pH condition induces the dissolution of reactive silica (Si^{4+}) and alumina (Al^{3+}) ions present in the soil minerals within the medium. Following, the pozzolanic reaction occur between free Ca^{2+} ions from lime and the dissolved silica Si^{4+} and alumina Al^{3+} ions from soil (a natural pozzolan as it contains silica and alumina) forming calcium silicate and calcium aluminate which later transforms to hydrates and form cementitious compounds (viz. calcium silicate hydrates (C-S-H), calcium aluminate hydrate (C-A-H) and Calcium Aluminate Silicate Hydrates(C-A-S-H)) in the presence of adequate moisture (Zoheir, 2015).

Red tropical soils are pozzolanas and hence react with lime. Their activity is associated mainly with their clay size fraction (Nwakanma, 1979). A pozzolan is "a siliceous and aluminous material which in itself possesses little or no cementitious value, but in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature (room temperature of 25^{o}C) to form compounds possessing cementitious properties" in form of gels. Fly ash and rice husk are also examples of pozzolana (Spears, 1995). This gel forms the matrix that contributes to the strength of lime stabilised soil layers. As the matrix forms, the soil is transformed from a sandy granular material to hard relatively impermeable layer. The pozzolanic reaction might extend to prolonged duration depending on the nature and availability of reactive clay minerals in the soil, eventually leading to a progressive development of strength, stiffness and durability of the stabilised soil. This phenomenon is termed "**long term stabilisation**", and is affected by the Clay Mineralogy, compaction state of soil-mix, as well as curing conditions. The matrix formed is permanent, durable and significantly impermeable, producing a structural layer that is both strong and flexible, unlike modification which no structural credit is accorded.

From the review of past related work done by Nwakanma (1979), it was reported that red tropical soils containing high proportions of aluminium oxide may show a decrease in strength with time due to the formation of CAH_{10} and C_2AH_8 which later change to a more stable and weaker form C_3AH_6. Unless this is the only reaction product, which is unlikely, the reduction in strength may be masked by a general increase in strength. With lime treatment most red tropical soils lose the little swell they had and permeability is greatly reduced. The accumulation of unreacted soil at the expense of reaction products is not desirable for increase in strength. However, an amount of unreacted material is required to overcome cracking of the soil-lime mix to some extent and to give optimal working conditions in practice. Lime is recommended to be used with fine grained soils. Lower effectiveness of lime with coarse grained soil can be attributed to scarcity of pozzolana (silicious and aluminicious material) which is required for pozzolanic (cementitious) reaction.

2.5.2 Carbonation reactions

These occur as a result of lime reacting with carbon dioxide (CO_2) in the air. When the CO_2 is dissolved in the pore water soil, it reacts with the hydroxyl ions, forming carbonate ions, which subsequently reacts with the calcium ions. This results in the formation of calcium carbonate $CaCO_3$, a weak cement whose formation is undesirable (unless a relatively high amount of lime is used), as this reaction consumes lime which would have otherwise been used in pozzolanic reactions for the formation of strong cementitious bonds. The carbonation reactions can be described as follows:

$$CO_2 + 2HO^- \rightarrow CO_3^{2-} + H_2O \qquad (2.7)$$

$$CO_3^{2-} + Ca^{2+} \rightarrow CaCO_3 \qquad (2.8)$$

Carbonation can either cause strength to continue to increase at a reduced rate and/or eventually begin to decline. Carbonation can cause a longer-term reversion to the original properties. So, some caution and special construction measures should be adopted when using such treated soil (Cherian and Arnepalli, 2015).

Also, primary “sulphate-induced heaving” problems arise when natural sulphate rich soil are stabilised with calcium-based additives (Puppala *et al.*, 2004). This is also known as “sulphate attack”. This heave is known to severely affect the performance of pavements and other geotechnical structures built on sulphate rich soils, stabilised with additives such as the calcium-based additive (Hunter, 1988).

The loss of strength at higher lime contents, was predominantly due to the limited amount of moisture available to hydrate the excess free lime, and thereby hinders the formation of cementitious products (Bozbey and Garaisayev, 2010). In order to increase the workability and strength and reduce the plasticity index and swell, lime is added to subgrade soil. Depending on the soil type, amount of lime varies from 4 to 6 %. The greater percentage of lime should be used for low quality subgrade soil (Arif *et al.*, 2019).

The quantity of lime needed to effectively treat a clay mineral is dependent on the type of mineral present. Kaolinite clay increases in strength with the addition of the first increment of lime, while for Illite, lime in excess of 4 - 6% must be added before any strength develops. The reaction of quartz with lime leads to strength built up, but for illite and montmorillonite little strength is developed until after the clay is saturated and the clay minerals begin to be destroyed (Eades and Grim, 1960).

Based on numerous studies, it is well known that the sustained and relatively slow pozzolanic reaction between lime and reactive silica (Si) and alumina (Al), is the key to effective and durable stabilisation of soil-lime matrix. The soil-lime interactions are time and temperature dependent, and continue for ages under apposite environmental conditions in a properly designed system (Arabi *et al.*, 1989).

Existing literature elucidate that the primary cause for increase in MDD with lime content was the filling up of void spaces with increasing amount of lime particles, thus densely packing the soil particles together (Cherian *et al.*, 2016). Further, drop in the MDD beyond OLC was associated with the excess moisture and lime remaining in the soil after pozzolanic reactions cease.

Generally, increase in lime increases OMC and decreases MDD (Bell, 1996). This is in agreement with Osula (1991) but not in agreement with Osinubi (1998a). The reason advanced is that the increased desire for water is somewhat commensurate to the increasing amount of lime. More water is needed for the dissociation of lime into Ca and

OH ions to supply more Ca ions for the cation exchange reaction. Johnson (1948) observed that of eleven (11) different soils tested two (2) exhibited an increase in density with the addition of lime (Monowar and Sujit, 2015).

Increase in OMC with increasing lime content may be due to increase of the fine fraction and the hydration of lime (Amadi and Okeiyi, 2017). Osinubi and Nwaiwu (2006) observed that maximum dry density of a laterite soil reduced until about 3% of lime and exhibited an increasing trend when the lime content was increased to 5%; beyond that, the density again reduced, which could be due to some local factors. Hence, it can be said that to obtain better compaction density of soils, more than 5% lime by weight should be added (Monowar and Sujit, 2015). Lime has been known to reduce the swelling potential, liquid limit, plasticity index and maximum dry density of the soil, and increases its optimum water content, shrinkage limit and strength. It improves the workability and compatibility of subgrade soils (Ali *et al.*, 2017). Broderick and Daniel (1990) reported that the lime and cement stabilised soils are less vulnerable to attack by organic chemicals in comparison to untreated soils.

2.5.3 Optimum lime content (OLC)

The term 'Optimum lime content' (OLC) defines the amount of lime required for satisfying the immediate/short term soil-lime interaction, still providing sufficient amount of free calcium and high residual pH necessary to initiate long term pozzolanic reaction.

It must be noted however that the optimal quantity of lime required for stabilisation primarily depends upon the reactive nature of soil, the degree of improvement desired, type of soil, clay content of soil and prevailing environmental conditions. Also, the accuracy of conventional tests is limited by combined influence of chemical and mineralogical properties of the soil, incorporating the influence of soil properties such as clay mineralogy, specific surface area, soil pH, cation exchange capacity, soil acidity, base saturation and buffer capacity. For stabilisation, pH of the soil plays a prominent role. At site, soil containing pH equal to or greater than 7 are highly reactive to lime as compare to those soils having pH less than 7. When lime is added to fine grained soil for improvement of its physico-mechanical properties initially all the available calcium (Ca^{2+}) ions are absorbed unto inter and intra layer surface of clay minerals present in the soil in order to satisfy the affinity owing to charge deficiency. This phenomenon is termed as lime fixation and the amount of lime fixed in the soil is referred to as Lime Fixation Point (Lm) or Initial Consumption of Lime, ICL. Accordingly, Hilt and Davison (1960) developed a mathematical correlation between Lm and clay-size fraction (≤ 2μm) which is represented in the form of equation 2.9.

$$\text{Lime Fixation Point (Lm)} = \frac{\text{Clay size fraction (\%)} + 1}{35} \qquad (2.9)$$

Bell (1996) indicated that the optimum addition of lime needed for maximum modification of the soil is normally between 1 and 3% lime by weight. Further additions of lime do not bring changes in the plastic limit, but increase the strength. According to Cherian *et al* (2016), the optimum lime content (OLC) for a micro clay soil was found to be 2% at 25°C and 4% at 40°C from a UCS test result. Basma and Tuncer suggested that OLC will always be higher than ICL, usually 2 – 8% by weight of dry soil as other studies reported the use of lime between 2 and 8% in soil stabilisation (Basma and Tuncer, 1991). The following reasons could account for this behaviour of lime with clay:

i. The lime causes aggregation of the particles to occupy larger spaces, and hence alters the effective grading of the soils;
ii. The specific gravity of lime is generally lower than the specific gravity of the soil tested;

iii. The pozzolanic reaction between the clay present in the soils and the lime is responsible for the increase in optimum moisture content (Ghobadi, 2014).

For pozzolanic reactions to occur, lime in excess of the complete saturation in calcium of the treated clayey soil must have been supplied. After complete saturation in calcium of the clay, with full completion of cationic exchange reactions, any excess calcium will then be available for pozzolanic reactions and the creation of cementitious products. It should be noted therefore that, more than the optimum of additives used is usually recommended. According to Monowar and Sujit (2015) it was recommended that to obtain better compaction density of soils, more than 5% lime which was the Optimum lime content (OLC) by weight should be added. The critical synthesis of available literature suggests that the so measured Optimum lime content (OLC) is the minimum amount of lime, only sufficient enough to create favourable conditions for dissolution of reactive clay minerals. It does not supply adequate amount of free Ca^{2+} ions to take part in the long-term pozzolanic reactions and thus to improve permanent soil strength (Cherian and Arnepalli, 2015). Hence more than the optimum is usually recommended for effective stabilisation to ensure the availability of excess Ca^{2+} ions to take part in the long-term pozzolanic reactions.

Note that the temperature of the soil during curing was reported as having a beneficial effect on chemical soil improvement. The effects are enhanced upon elevated temperatures and annihilated below 4ºC. Where lime is used solely to dry up wet soils for compaction and not for permanent stabilisation, the operation can be carried out in colder weather. In no case, however, should lime be applied to frozen soil to avoid annihilation of the reaction (National Lime Association, 2004).

In a recent testing of a highly expansive soil treated with varying lime percentages for the soil (0 - 20%), studies showed that for all percentages used, the unconfined compressive strength (qu) increased in time with high lime percentages. At low lime percentage addition (2 and 4%); strength kept developing up to 28 days curing. The qu of the specimens treated with 6% lime was found to be 5 times higher after 28 days curing and 6 times after 90 days curing compared to the qu of the untreated specimen. With 20% lime addition, qu increased by 8 times for 28 days curing and 17 times after 90 days curing compared to the qu of the untreated specimen (Cherian and Arnepalli, 2015). Note the very high percentages of lime used for this soil, which contradicts 1 - 3% when using quicklime according to Bell (1996) and statement made by Basma and Tuncer (1991) that 2 to 8% of hydrated lime would be enough. This shows the strong dependence of the necessary lime percentage on the type of soil.

2.5.4 Mellowing time

Finally, the unconfined compressive strength qu was found to be dependent on the compaction delays. This delay is called in the literature the ‘mellowing’ time, as opposed to ‘curing’ time, which refers to times after compaction. The (up to three hours) strength characteristics of lateritic soil treated with a maximum of 8% lime (by dry weight) were introduced by Osinubi (1998a). These indicated that the compaction and strength properties of lime treated soil, declined with an increase in mellowing time. Note that the studied compaction delay (mellowing) times in Osinubi (1998a) are indeed quite short. The British Standards specify a minimum of 12 hours for untreated London Clay samples and between 24 and 48 hours duration for lime treated samples to allow for mellowing (Zoheir, 2015).

For a clay content greater than 25 - 30%, volumetric change greater than 20 - 30% and PI greater than 15-18 which are the physical properties of high clay content soil, lime is used on the priority basis (Arora and Aydilek, 2005). Soil with clay content > 25% and plasticity index PI > 10 fulfils the criteria for soil to be suitable for lime stabilisation (Adnan *et al.,* 2019). Lime usually reacts with the most of soils with plasticity index ranging from 10 – 50 (Bell, 1996). Improvement in CBR values was also observed for laterite-polyethylene mixture, resulting in maximum CBR value of 13.18% under soaked condition. This value falls within the range of 10 to 25% CBR value as it was specified for sub grade soils by the Nigerian Highway Design Manual, Federal Ministry of Works and Housing (Ojuri, 2016)**.** Basically, the benefits of stabilising soil include the following;

i. Improves quality of pavement and consolidation settlement of structures,
ii. Eliminates the handling and hauling quantity of excavation material,
iii. Gives higher resistance value,
iv. Reduce the swelling characteristics and plasticity of clayey soil (Ratna *et al*., 2018).

This study involves the evaluation of the effect of using lime-plastic blend for soil stabilisation. In the present study investigations were made according to the BS 1377 part 2: 1992 of the British Standard code.

CHAPTER THREE

3.0 MATERIALS AND METHODS

This chapter covers the methods used for collection of materials, laboratory testing and analysis of the data obtained for the untreated and treated soil samples. Conventional tests for evaluation of soil suitability for engineering purposes were carried out on the samples collected from the site and were treated with the stabilisers subsequently.

3.1 Materials

The materials used for this research include; soil, lime, plastic waste materials and water. The soil was sourced for around Minna, Niger state while the lime and plastic materials were gotten from Kaduna state for the treatment of the soil in this research.

3.1.1 Natural soil

The soil sample used for this research is a reddish - pinkish brown soil and is shown in Plate I a. This was obtained from Dibbo borrow pit used by Dantata and Sawoe construction company for construction of the Minna – Bida road in Minna Niger state. The sample was collected in their disturbed state from a sufficient depth of 1.5 m below the ground surface. The geotechnical properties of the soil were determined in the laboratory which include; sieve analysis, consistency limits and strength tests in accordance with BS 1377 (1992). X-Ray Diffraction (XRD) and X-Ray Fluorescence techniques were also used to determine the chemical oxide and mineralogical composition of the natural soil.

3.1.2 Lime

The lime used as a chemical additive for this study is hydrated lime and is shown in Plate I c. This was supplied from Bicaj Investment Company Limited in Kaduna State. X-Ray Diffraction (XRD) and X-Ray Fluorescence technique was also used to determine the mineralogical composition of the lime. The proportion of lime added to the soil was 2, 4, 6 and 8%.

3.1.3 Plastic material

The increase in use of one-time usable plastic material in day-to-day life is producing large volumes of plastic waste and thus becoming a nuisance to the environment. Usually, some amount of these plastic wastes is recycled and the remaining waste is incinerated for the production of thermal energy. The plastic waste material used for the research was obtained from a recycling factory in Kaduna state and is shown in Plate I b. The plastic material was analysed for their chemical composition using Scanning Electron Microscopy – Energy Dispersion Spectroscopy (SEM-EDS). The plastic was combined with the required percentage of lime and used as additive for the treatment of the soil. The proportion of plastic added to the soil was 1, 2, 3 and 4%.

Plate I a: Lateritic soil

Plate I b: Plastic waste

Plate I c: Hydrated lime

3.2 Methods

3.2.1 Sample collection and laboratory analysis

The soil sample was collected at a depth of 1.5 m from Dibbo borrow pit used by Dantata and Sowoe Construction Company in the suburbs of Minna while the lime and plastic waste additives were obtained from Kaduna state. The soil sample collected was oven dried and necessary laboratory procedures were followed in the determination of the natural moisture content of the sample and other tests required.

3.2.2 Laboratory tests

In the present study investigations are made according to the BS 1377 part 2: 1992 of the British Standard code. The mineralogical characterization of the natural soil and lime additive were conducted using the X-Ray Diffraction (XRD) and X-Ray Fluorescence technique. The plastics were analysed for their morphological properties and elements in the plastics structure using Scanning Electron Microscopy – Energy Dispersion Spectroscopy (SEM-EDS). These properties of materials used were determined at Spectral Laboratory Services located at No.14 Forte Oil Station Polytechnic Road, Kaduna south, Kaduna state.

The blend of lime and plastic materials was added to the soil in dry condition, mixed thoroughly to get a uniform mixture. Then the required amount of water was added, mixed, moulded and tested. The geotechnical experiments carried out in this study is presented in Table 3.1.

Table 3.1: Laboratory Tests Conducted on the Soil Samples

Untreated soil samples	Soil Samples treated with 2, 4, 6 and 8% lime and 1, 2, 3 and 4% plastic.
1)Natural Moisture Content	1) Atterberg limits
2)Specific Gravity	2) Standard Compaction test
3) Grain size distribution	3) California Bearing Ratio (CBR) test
4) Atterberg limits	4) Unconfined Compression (UCS) test
5) Standard Compaction test	
6) California Bearing Ratio (CBR) test	
7) Unconfined compression (UCS) test	

3.2.3 Index properties of the soil

Preliminary tests (such as particle size distribution analysis and Atterberg limit tests) for the purpose of determining the soil's index properties were carried out on the sample according to the BS 1377 part 2: 1992 of the British Standard code. The lime was first used to stabilise the soil, then the selected percentage of lime was then combined with different percentages of plastic wastes for stabilising the soil.

3.2.3.1 *Determination of natural moisture content*

This is the determination of the natural moisture content present within the soil samples. The procedure is as follows;

i. Immediately after the soil sample dug from the borrow pit was collected, a little of the samples (about 50g – 70g) were collected into four (4) containers (that has been weighed already),
ii. The samples collected with the cans were weighed in the laboratory and then oven dried for 24 hours
iii. After the samples were oven dry for 24 hours, they were reweighed and recorded

The readings recorded were then tabulated and the percentage moisture content MC (%) calculated as shown in equation 3.1;

$$\text{Moisture Content MC (\%)} = \frac{\text{weight of water}}{\text{weight of the dry sample}} \text{ x } 100 = \frac{Wc}{DS} \text{ x } 100 \qquad (3.1)$$

This was repeated for trials 2, 3 and 4 and the average moisture content taken to get the natural moisture content of the soil. Amount of water that the soil can hold defines the size of soil particles as fines holds more water over time and allows water rise from underlying layers (capillarity). Soils with more pores between particles hold more water. Soil with more water has low strength, because more water between soil particles reduces shear strength of soil and affects dry density also. This property of soil is especially useful in compaction of the slopes of embankments

3.2.3.2 *Specific gravity test*

For this test the procedure is as follows;

i. The samples were first screened through a BS sieve (5.0 mm) thoroughly to remove deleterious materials.
ii. The weight of the empty bottle was recorded as M_1. Then the sample was filled into the density bottle and weighed; the weight was recorded as M_2 (weight of bottle + dry sample).
iii. The density bottle was gradually filled with distilled water to gauge mark and allowed to soak. At the end of soaking, air entrapped and bubbles on the surface of the aggregate sample were removed by shaking the density bottle and the weight was recorded as M_3 (weight of bottle + dry sample + distilled water).
iv. After which the bottle was emptied and dried. The density bottle was then filled with only distilled water to the gauge mark and weighed as M_4 (weight of bottle + distilled water).

The equation used to determine the specific gravity of the aggregate is given in equation 3.2;

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

$$\text{Specific Gravity (Gs) of soil} = \frac{\text{M2} - \text{M1}}{(\text{M4} - \text{M1}) - (\text{M3} - \text{M2})} \tag{3.2}$$

3.2.3.3 *Grain size distribution*

For this test, the clay content of the soil sample was first washed out in order to determine the percentage clay content of the soil. The remaining soil sample was dried and used for the analysis of the different particle sizes present in the soil sample by passing it through a set of sieve sizes according to the specification in BS 1377 part 2: 1992 of the British standard code. The particle size distribution curve was plotted and used for the classification of the soil. The percentage weight retained in each sieve size and their percentage (%) passing is then calculated for using equations 3.3 and 3.4;

$$\% \text{ Retained} = \frac{\text{weight of soil retained}}{\text{total weight of soil sample}} \times 100 \qquad (3.3)$$

$$\% \text{ Passing} = 100 - \% \text{ cumulative weight retained} \qquad (3.4)$$

The graph of percentage passing is plotted against grain size on a semi-log graph with the particle size as the X-axis with logarithmic axis and the percentage passing as the Y-axis which gives a clear idea of the particle size distribution of the soil. The percentage clay content, coefficient of curvature and coefficient of uniformity are calculated using equations 3.5, 3.6 and 3.7 respectively.

$$\% \text{ of clay} = \frac{\text{weight of clay}}{\text{total weight of soil}} \times 100 \qquad (3.5)$$

$$\text{Coefficient of curvature Cc} = \frac{(D30)2}{D10xD60} \qquad (3.6)$$

$$\text{Coefficient of Uniformity Cu} = \frac{D60}{D10} \qquad (3.7)$$

For a soil to be well graded, it must be within the range 1< Cc < 3 else it is considered a poorly graded soil, as shown in equation 3.6. A Cu greater than 4 to 6 classifies the soil as well graded. When Cu is less than 4, it is classified as poorly graded or uniformly graded soil. For any single sized soil mass, the value of both Cu and Cc is 1. Any material of which more than 35% passes the No.75 mm sieve is considered unsuitable depending on the plasticity characteristics of the material and must be replaced provided good quality fill materials are available; otherwise, such material may be modified for use.

3.2.3.4 *Atterberg limits*

For the Atterberg limits test, the data obtained from the laboratory test is used to determine the plasticity index (PI) of the soil. The procedures used for determining the liquid limits and plastic limits of the soil are presented as follows;

Liquid limit (LL) test: The liquid limit can be determined using the cone penetrometer or the casangrande apparatus (BS 1377: 1992: part 2, clause 4.3, 4.5). In this experiment the cone penetrometer method was adopted and the procedures for the liquid limit is;

i. 500 ± of air-dried soil is pulverised and passed through the sieve size 425µm.
ii. 80 ± ml of water is added to the soil and mixed as thorough as possible.
iii. 300 g of the sample is taken from the paste prepared and placed on the glass plate.
iv. The paste is then mixed for at least 10 minutes using two palette knives. If necessary, more distilled water is added so that the first cone penetration reading is about 15 mm
v. A portion of the mixed soil is pushed into the cup with a palette knife, taking care not to trap air.
vi. The penetration cone is locked in the raised position and the supporting assembly is lowered, so that the tip of the cone just touches the surface of the soil. When the cone is in the correct position, a slight movement of the cup will just mark the soil surface. The stem of the dial gauge is lowered to contact the cone shaft and the reading of the dial gauge is recorded to the nearest 0.1 mm
vii. The cone is released for a period of 5 ± 1s, after which it is locked in position and the stem of the dial gauge is lowered to contact the cone shaft, to record the reading of the dial gauge to the nearest 0.1 mm. The difference between the beginning and end of the drop of cone penetration is recorded.

viii. The cone is lifted up and cleaned carefully to avoid scratching.

ix. A moisture content of the sample of about 10 g is taken from the area penetrated by the cone and the moisture content is determined.

x. Step v-ix is repeated at least three more times, using the same sample of soil to which further increment of distilled water have been added. Proceeded from the drier to the wetter condition of the soil.

xi. At any time during the above procedure, the soil has to be left for a while on the glass plate. The soil was covered with evaporating dish or a damp cloth to prevent the soil drying out (Abdul, 2014).

xii. The percentage moisture content of the samples tested is then calculated for and the result of penetration data obtained was also recorded.

xiii. Then a graph of penetration vs. moisture content (%) was plotted and the percentage moisture content obtained at 200 penetrations from the graph was taken as the liquid limit of the soil.

Plastic Limit (PL) test: Plastic limit (PL) can be defined as the maximum moisture at which the soil can be rolled into a thread of 3 mm diameter without breaking up. It is the percentage moisture content at which the soil-water paste changes from a semi-solid to a plastic consistency as it is rolled into a 3.175 mm diameter thread. The procedure for this test is as follows;

i. About 20 g of the dried soil samples, all passing the 0.425 mm sieve were mixed with distilled water and moulded into ball. The balls of soil were rolled by hand on a glass plate with sufficient pressure to form thread. When the diameter of the resulting thread becomes 3 mm, the soil is threaded together and then rolled out again. This process continues until the thread crumbles.

ii. These pieces are collected in cans and weighed. The weights are recorded and then oven dried.
iii. The oven dried samples collected are then recorded and used to determine the percentage moisture content of the soil sample.
iv. The whole procedure was carried out twice and the average value of moisture content was taken as the plastic limit of the soil.

The plasticity index of the soil is calculated by using the liquid limit (LL) obtained from the graph of penetration vs. moisture content (%) and the plastic limit (PL) obtained from the analysis of the data from the plastic limit test. The percentage moisture content MC (%) which was plotted with the penetration was calculated for using equation 3.1

The Liquid Limit (LL) = MC (%) at 20 mm penetration

The plasticity index of the soil was calculated as shown in equation 3.8:

Plasticity index (PI) = Liquid Limit (LL) – Plastic Limit (PL) (3.8)

Based on the plasticity of the soil, it can be classified according to the British Standard (BS) system of description and classification, based on part of the geotechnical reference package by John (2000) which is presented in Table F1. The specification for Liquid Limit LL and plasticity index PI are LL $\leq$ 35% and PI $\leq$ 12%. At the Engineer Representative's discretion, materials with liquid limit greater than 35% and a plasticity index greater than 12 may be disallowed for use as sub-base course or base course material (Federal Ministry of Works, 1997).

The result of the liquid limit and the plasticity index obtained from the Atterberg limit test was used for the classification of the soil. The soil was classified as a clayey soil of low plasticity (CL) from the Unified Soil Classification System (USCS) system of classification and clayey of intermediate plasticity (CI) based on the British Soil Classification System. According to the American Association of State Highway and Transportation Officials (AASHTO) system of classification, it is classified as A-7-6 clayey soil. A plasticity index greater than 10 and percentage passing sieve No. 200 greater than 35% implies a weak soil, hence the need for stabilisation of the soil to reduce its plasticity and increase its strength. The degree of plasticity related to the clay content is called the **"activity of soil"** and is given by equation 3.9. The recommended standard for soil classification is the British Soil Classification System which is presented in Table F2 of the Appendix.

$$\text{Activity (A) of soil} = \frac{\text{Plasticity Index(PI)}}{\text{\% clay particles}} \qquad (3.9)$$

3.2.4 Engineering properties of the soil

3.2.4.1 *Compaction test*

Compaction is the densification of unsaturated soil, by the reduction in volume of voids filled with air, while the volume of solids and water content remains the same. The major aim of compaction of the soil is to increase shear strength, decrease compressibility, reduce permeability, and to control swelling and shrinkage of soil. Increasing the moisture content beyond the OMC does not increase dry density any further but starts replacing the soil particles as the unit weight of water is less than the soil particles, hence the dry density starts decreasing (CivilSeek, 2019). The degree of compaction is measured in terms of its dry density and the maximum dry density (MDD) occurs at the optimum moisture content (OMC) (CivilSeek, 2020).

Compaction of the sample was conducted in accordance with the guidelines specified in BS 1377 (1992) to compute for the dry density and moisture content. British Standard Heavy (BSH)/Modified compaction was adopted for this test. The apparatus used consists of a cylindrical metal mould of a capacity of 1000 cm^3, an internal diameter of 101.6 mm, and an effective height of 116.43 mm with a detachable base plate, a collar 50 mm deep that fits into the top of the mould, a rammer having 50 mm diameter circular face and weighs 4.5 kg. The rammer is contained in a cylindrical sleeve designed such that the rammer falls into the soil through a height of 450 mm for each blow. The soil sample was compacted at 27 blows for 5 layers. The procedure involved in this test is as follows;

i. 3000 g of dry soil sample was weighed out and placed in a large tray. The sample was pulverised and 6% of water was added to the soil.

ii. The mould was then filled in layers with each layer being subjected to 27 blows, using the specified rammer for the test after which it was weighed.

iii. The soil was then removed from the mould; sample of the soil was taken from the top and bottom of the specimen for moisture content determination. This was used for the calculation of the dry density.

iv. The test was repeated with the same soil after pulverization of the remoulded specimen at different moisture contents. 3% of water was added subsequently after each test, until at least two successive tests show a decrease in bulk density/wet density. This is calculated as shown in equation 3.11

v. A graph of dry density against moisture content was then plotted and used to determine the maximum dry density and the optimum moisture content. The percentage moisture content was obtained from equation 3.1 and the dry density was calculated as shown in equation 3.10;

$$\text{Dry Density DD (g/cm}^3) = \frac{\text{wet density of soil Dw}}{1+\text{Moisture Content}} = \frac{\text{Dw}}{1+\text{MC}} \tag{3.10}$$

$$\text{Wet density Dw (g/cm}^3) = \frac{\text{weight of soil sample}}{\text{volume of soil sample}} = \frac{\text{Ws}}{\text{Vs}} \tag{3.11}$$

The optimum moisture content and the maximum dry density are obtained from the graph of dry density (g/cm^3) vs. moisture content (%). An OMC (%) of less than 18% is the Nigeria Specification for road and bridge materials for general filling and embankment (Nwadike and Nweke, 2016).

3.2.4.2 *California bearing ratio (CBR) test*

The California bearing ratio (CBR) test was conducted to determine the bearing capacity of the soil. It was carried out according to the BS 1377 part 2: 1992 of the British standard code. This is a penetration test used to assess the strength of the soil for construction purpose. 6000 grams of the soil sample was measured and compacted in 5 layers, 62 blows with a 4.5 kg rammer at a drop height of 450 mm at the optimum moisture content of the soil and tested in unsoaked condition. The CBR values at 2.5 and 5.0 mm penetration were computed for by using the formulas in equations 3.12 and 3.13 respectively.

$$\text{CBR} = \frac{\text{Test load (kN)}}{\text{Standard load}} = \frac{\text{load guage reading x proving ring constant}}{\text{Standard load}} \text{ x } 100 \tag{3.12}$$

$$\text{Where Test load (kN)} = \frac{\text{load guage reading x 41.6}}{1000}$$

$$\text{\%CBR at 2.5 mm penetration} = \frac{\text{load guage reading x 0.0416}}{13.24} \text{ x100} \tag{3.12a}$$

$$\%CBR \text{ at } 5.0 \text{ mm penetration} = \frac{\text{load guage reading x } 0.0416}{19.96} \text{x}100 \qquad (3.12b)$$

The higher value of the two CBR value obtained is selected as the CBR value obtained for the soil. Federal Ministry of works and housing recommendation for soils for use as: subgrade, sub-base and base materials are: ≤ 10%, ≤ 30% and ≤ 80% respectively for unsoaked condition (Layade and Ogunkoya, 2018).

Normally the minimum strength of base course material shall not be less than 80% CBR value. However, where the Engineer's Representative considers it necessary on account of perched water-table or any other reasons, he may specify that a CBR value of 80% be obtained after at least 24 hours soaking (Federal Ministry of Works, 1997).

3.2.4.3 *Unconfined compressive strength (UCS) test*

The test was conducted according to the procedure described in BS, 1377: (1992). The specimens were passed through 4.75mm sieve and were prepared at optimum moisture contents (OMC) obtained from the compaction test and compacted at 10 blows and 3 layers. Samples used had a height to diameter ratio of 2:1. Cylindrical test specimens of diameter 38mm with a height of 76 mm was prepared using remoulded samples for the test. The samples were then cured for 7, 14 and 28 days. The dial gauge readings were taken as the specimen was been loaded until it failed at the peak load Data obtained were recorded and used for the computation of the unconfined compressive strength.

The undrained shear strength (Su) (basically equal to the cohesion (c)) of a cohesive soil is equal to one-half the unconfined compression strength (qu) when the soil is under the f = 0 condition (f = the angle of internal friction). This is expressed in equations 3.13 to 3.18;

Shear Strength $S_u = c = \frac{qu}{2}$ (3.13)

q_u = peak axial stress

Axial Stress (kN/m^2) $Sc = \frac{\text{Load kN}}{\text{Anew}} = \frac{P}{A''}$ (3.14)

Load (kN) $= \frac{\text{load guage reading x 7.14}}{1000}$ (3.15)

$$A'' = \frac{A0}{1-e} \quad (3.16)$$

Where A_0 is the old cross-sectional area of the specimen before it was loaded

e is the strain obtained from the deformation of the specimen after loading

A" is the new cross-sectional area of the specimen after loading

$A_0 = \frac{\pi}{4} \text{ x } d^2$ (3.17)

Where d is the diameter of the sample tested

Axial Strain e (%) $= \frac{\text{sample deformation after loading } \Delta L \text{ (mm)}}{\text{original length of sample before loading Lo}} \text{ x } 100 = \frac{\Delta L}{Lo} \text{ x } 100$ (3.18)

The condition to be met for UCS value by the specification of the Nigeria General Specifications for Highways is 750 – 1500 kN/m^2 for use as sub-base material for light trafficked highways (Opeyemi and Grace, 2018). From the Nominal strength classification of materials in the design catalogue in the Federal Ministry of Works highway manual presented in Table F3 in the Appendix., Unconfined Compression Strength (UCS) for sub-base after curing for 7 days is 750 kN/m^2 – 1500 kN/m^2 at 100% modified AASHTO density (NB: 1 MPa = 1000 kN/m^2).

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

The results of the untreated and treated soil sample, obtained from the data gotten from the laboratory tests are presented in this section of the study. Also, the analyses of the microstructural properties of the soil and additives used in this study are also presented in this section.

4.1 Result of the Untreated Natural Soil Sample

Experimental studies are required in order to know the effect of addition of waste plastic on index and engineering properties of the soils. The results obtained for the untreated and treated samples in this study are presented in this section of the study.

4.1.1 Index properties

Result obtained for the index properties of the soil is presented in this section.

4.1.1.1 *Natural moisture content of the soil*

For the soil samples, the Moisture content is obtained from equation 3.1. The result of the moisture content for the trials and the average percentage moisture content is presented in Table A1. The average natural moisture content of the soil sample obtained is 22.46%. Since the optimum moisture content of this soil is 15.3%, this implies that the natural soil has more moisture content than the optimum water content of the soil which can decrease the strength of soil and dry density. Hence there is need for stabilisation of this soil to increase the dry density and strength especially for compaction of slopes of embankments.

4.1.1.2 *Specific gravity*

The specific gravity of the soil and the additives used for the test were determined. For the natural soil sample, the data and result of the test obtained from equation 3.2 is presented in Table A2. The specific gravity of the soil for trial 1 is 2.59 and the average specific gravity of the soil for the three trials is 2.63. From Tables A.3 and A.4 in Appendix A the average specific gravity of the lime used and the plastic waste materials obtained using equation 3.2 are 2.4 and 1.05 respectively.

4.1.1.3 *Sieve analysis*

Result of sieve analysis test carried out is presented in Table A5 and Figure 4.1. From equations 3.3 and 3.4, the percentages weight retained and passing were calculated for respectively.

% Retained = $\frac{\text{weight of soil retained}}{\text{total weight of soil sample}}$ x 100 = % Retained = $\frac{21.1}{300}$ x 100 = 7.03%

From equation 3.4, the percentage passing is given as;

% passing = 100 - % cumulative weight retained = 100-7.03 = 92.97%

The amount of clay content obtained from the soil after washing it is 178.2 g. The percentage clay content is obtained from equation 3.5 as;

% of Clay = $\frac{\text{weight of clay}}{\text{total weight of soil}}\ x$ 100 = % of clay = $\frac{178.2}{300}$ x100 = 59.4%

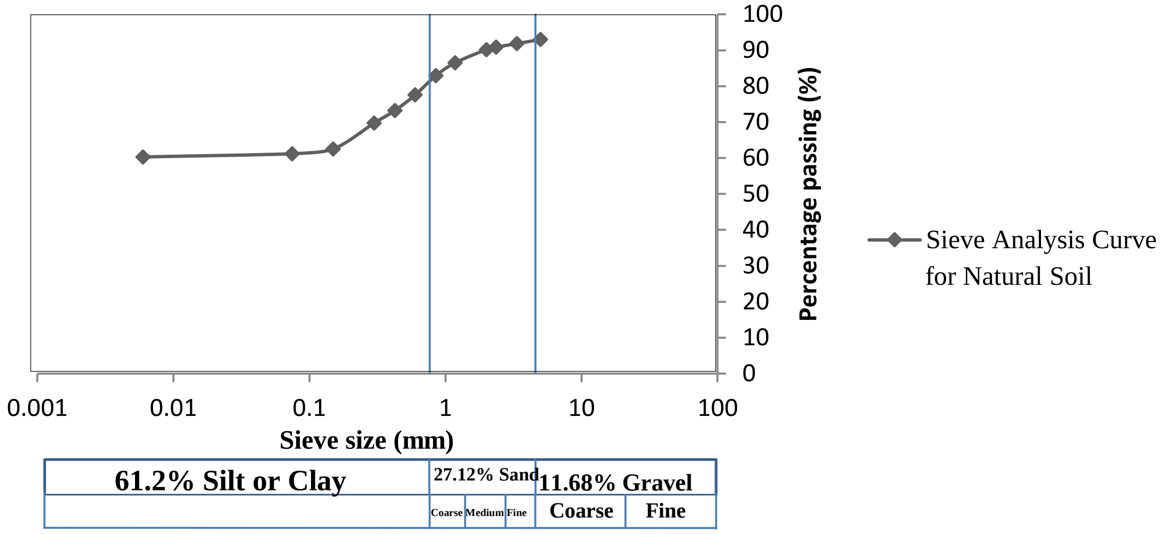

Figure 4.1: Particle size distribution curve of the natural soil

The Coefficient of curvature Cc and Coefficient of Uniformity Cu are obtained from equations 3.6 and 3.7 as; Cu = 30 and Cc = 0.41, where D_{10} = 0.0002, D_{30}=0.0007 and D_{60}=0.006; Hence based on the USCS flow chart for classification of coarse-grained soil, the soil is a poorly graded clayey sand and a gravelly lean clay with sand from the chart in Figure F4, Appendix F. Based on the AASHTO classification, the soil is classified as an A-7-6b clayey from the chart in Figure F2, Appendix F.

4.1.1.4 *Atterberg limits*

The data obtained for the Liquid Limit (LL) and Plastic Limit (PL) of the natural soil samples are presented in Tables B1. The results were obtained from equation (3.1) for the percentage Moisture Content MC (%)

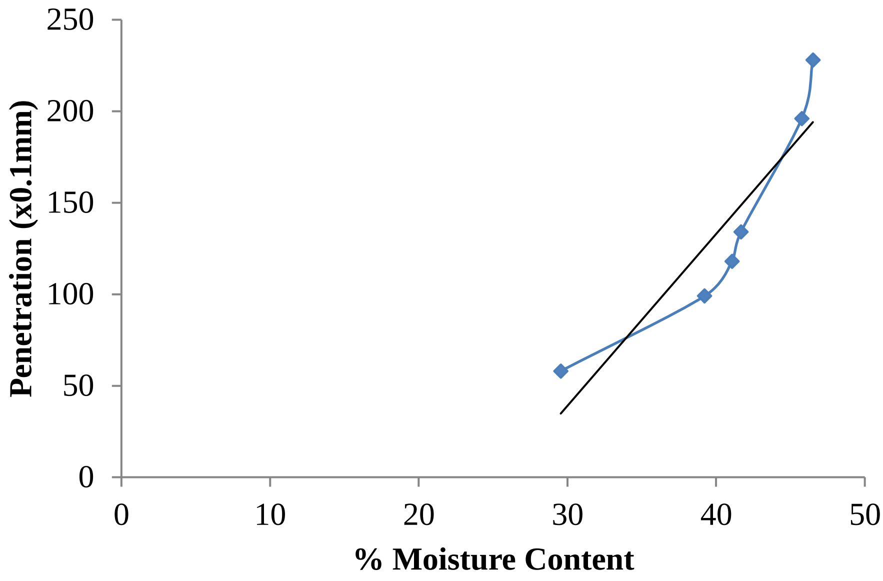

Figure 4.2: Penetration vs. % Moisture Content curve for the Natural Soil.

From the graph, the Liquid Limit (LL) = 47% which is the % Moisture Content at 200 penetration, Plastic Limit = 26.14 and from equation 3.8, Plasticity Index (PI) = 20 From equation 3.9 the activity of the soil is given as;

$$\text{Activity (A) of soil} = \frac{\text{PI}}{\text{\% clay particles}} = \frac{20.9}{59.4} = 0.35.$$

The activity value 0.35 is less than 1.25 the limit for active soil which shows that the soil is an inactive soil type, and does not belong to the swelling type of soil according to Skempton's classification of soil base on its activity value. Since it falls between 0.3 and 0.5 then this implies that the soil contains predominantly the kaolinite mineral whose activity value lies within this range as shown in Table 2.1.

4.1.1.5 *Compaction characteristics*

The percentage moisture content and the dry density were calculated from equations 3.1 and 3.10 respectively. The dry density and the corresponding moisture content for the natural soil is shown in Table C1 Appendix C. The Maximum Dry Density of 1.91 g/cm^3

and optimum moisture content (OMC) of 15.3% was also obtained from the graph of the dry density vs. moisture content as shown in Figure 4.3.

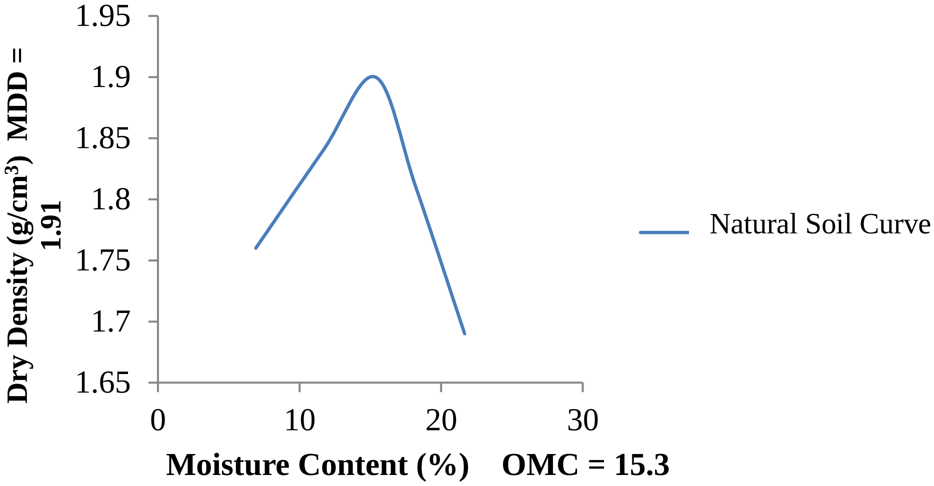

Figure 4.3: Compaction curve for the natural soil sample.

4.1.1.6 *California bearing ratio (CBR) test*

Data obtained for the natural soil sample are presented in Table D1. The data obtained were calculated from equation 3.12 for CBR test. The graph of Load vs. Penetration curve for the natural soil test conducted is presented in Figure 4.4.

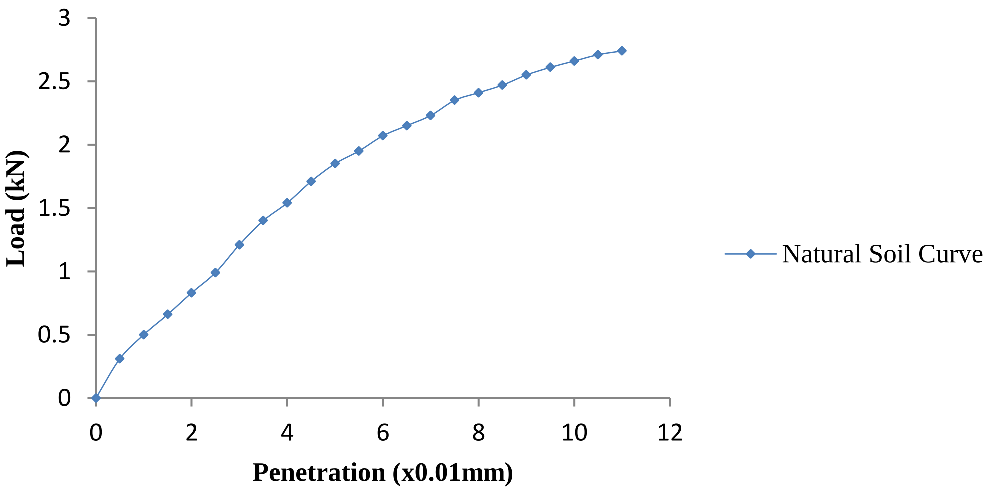

Figure 4.4: Load vs. penetration curve for natural soil

From equations 3.12a and 3.12b, the CBR values at 2.5 mm penetration and 5.0 mm penetration were calculated to be 5.34% and 6.044% respectively. The higher % CBR value was selected which is 6.04% at 5.0 mm penetration for the natural soil.

4.1.1.7 *Unconfined compressive strength (UCS)*

The graph of axial stress vs. axial strain obtained for the untreated soil sample from the result of unconfined compressive strength test conducted in the laboratory is presented in Figure 4.5. From equations 3.14 to 3.18, the Unconfined Compressive Strength (UCS) is calculated for. The results of the values obtained at each penetration are presented in Table E1 for the natural soil samples being tested.

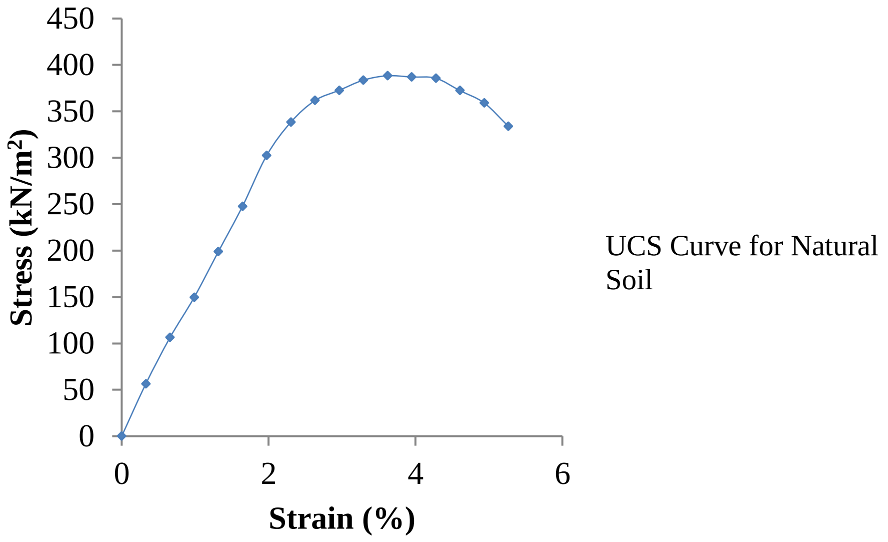

Figure 4.5: Axial stress vs. Axial strain curve for the natural soil

The maximum stress at which the sample failed was 388.4 kN/m^2. Hence the peak axial stress which gives the unconfined compressive strength (q_u) is 388.4 kN/m^2.

The summary of result obtained for the untreated soil sample is presented in Table 4.1. This gives the basic geotechnical properties of the untreated lateritic soil. The specific gravity of the plastic material obtained from analysis is 1.05 while that of the hydrated lime is 2.4 and the natural soil sample is 2.63.

Table 4.1: Summary of the Properties of the Natural Soil

Characteristics	Description
Natural moisture content (%)	22.46
% Passing B.S sieve No. 200	61.2
% of Silt and Clay, Sand and Gravel	61.2, 27.12 and 11.68
% Clay	59.4
Liquid Limit LL	47
Plastic Limit PL	26.14
Plasticity Index PI	20.86
Activity (A) of soil	0.35
AASHTO Classification	A-7-6
USCS Classification	CL
MDD (g/cm3)	1.91
OMC (%)	15.3
Specific Gravity	2.63
CBR (%)	6.04
Unconfined Compressive stress q_u (kN/m^2)	388.4
Colour	Reddish-Pinkish brown

4.2 Result of the Soil Sample Treated with Lime

4.2.1 Plasticity characteristics

The addition of lime to the soil resulted in the decrease of plasticity index from 20.86 to 18 at 6% lime as presented in Table B4. There was an overall increase in the plastic limits and liquid limits.

The graph of variation of the liquid limit, plastic limit and plasticity index of sample with different lime content is presented in Figure 4.6.

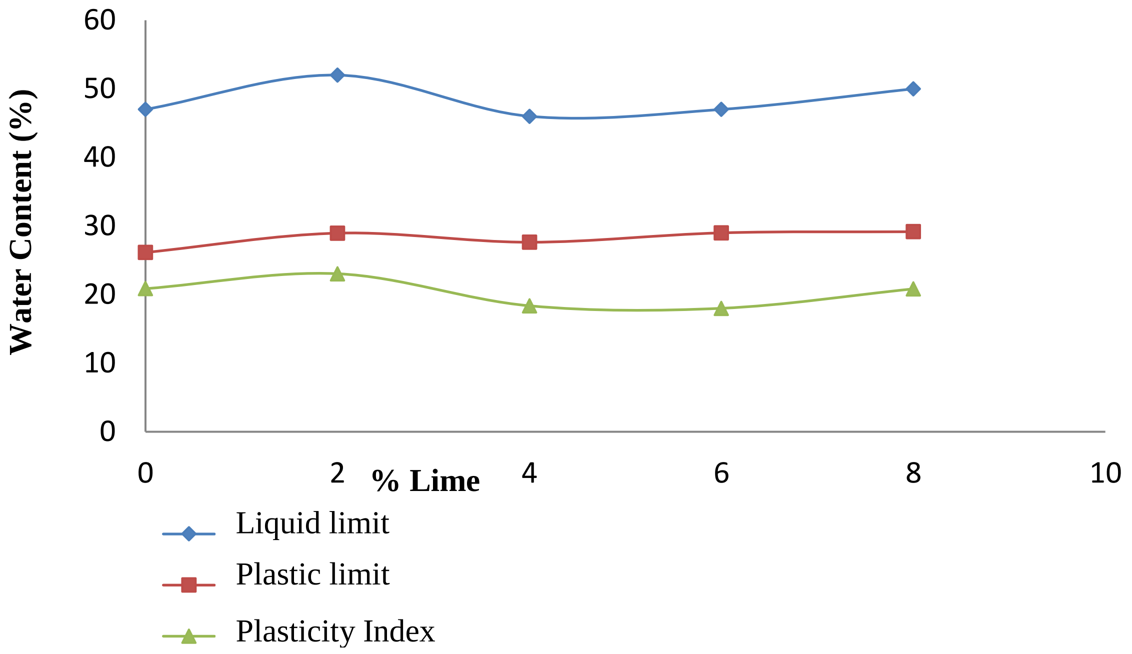

Figure 4.6: Variation of Atterberg limits of treated soil with % Lime

The plasticity index is seen to decrease up to 6% addition of lime to the soil. Further addition of lime resulted in increase of the plasticity index. From the graph in Figure 4.6, it is seen that there was an overall decrease in the plasticity index of the soil.

4.2.2 Compaction characteristics

Compaction result for soil with lime of different combinations is presented in Figure C1. The result obtained from the compaction test indicates that, the optimum moisture content of the soil increased from 15.3% to 16% with the increase in lime content from 0% to 8% while there was an overall decrease in the maximum dry density value from 1.915g/cm^3 to 1.825 g/cm^3 at 8% lime. These variation values are presented in Table C6. The variation of the percentage lime content on the soil is shown in Figures 4.7 and 4.8 for the optimum moisture content (OMC) and the maximum dry density (MDD) respectively.

The optimum moisture content is seen to decrease with increase in lime up to 2% lime, and continued to increase on further addition of lime as presented in Figure 4.7. This agrees with the work carried out by Bello and Adekoge (2013) where the OMC decreased up to 5% lime content and then continued to increase. As the lime increases, more hydrated calcium ions are released into the solution resulting in more flocculation and agglomeration of clay particles (Francis *et al.*, 2013). The reduction in the maximum dry density according to Bell (1996) could be due to an immediate formation of cementitious products which reduce compatibility and hence the density of the treated soil.

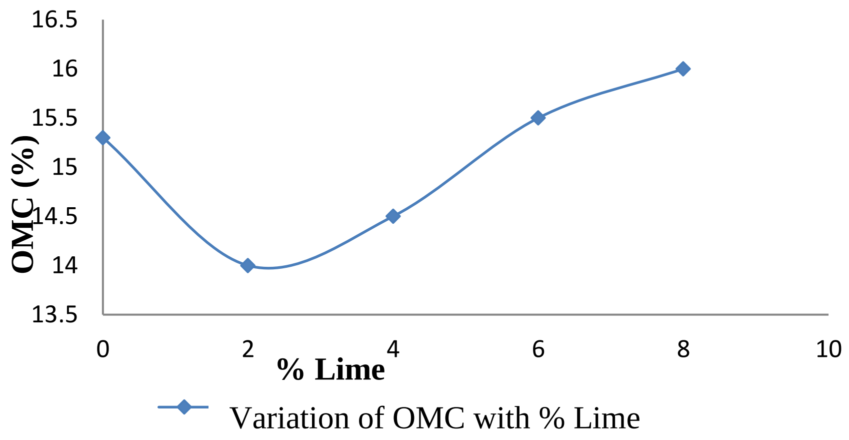

Figure 4.7: Variation of optimum moisture content with different % lime content

The MDD decreases until it gets to 2% lime, after which it increases up till 6% lime and then continues decrease. This is presented in Figure 4.8. This agrees with studies carried out by Bello and Adekoge (2013) where the MDD decreased to 3% lime and then increased up to 5% lime after which it continued to decrease.

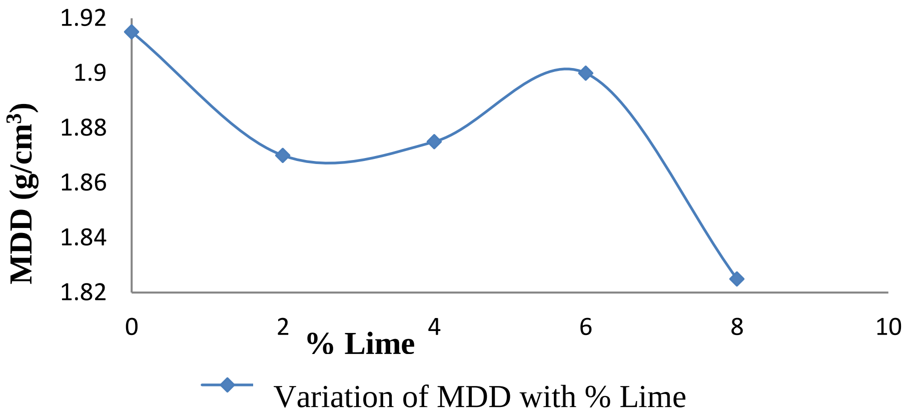

Figure 4.8: Variation of maximum dry density values with different % lime content

Also, Sujit and Monowar (2015) observed MDD reduces till 3% of lime after which it continues to increase. From this study 5% lime and above was recommended for better compaction density. This agrees with findings by Osinubi and Nwaiwu (2006). Hence for this study an optimum lime content of 6% or more recommended to obtain better compaction density.

4.2.3 California bearing ratio (CBR) characteristics

From the data obtained from the laboratory, the load vs. penetration curves were drawn for various percentage of lime in the soil. The graph of CBR is presented in Figure D1 Appendix D, while the variation of the CBR values with increase in the lime content, are tabulated in Table D3 and plotted in Figure 4.9.

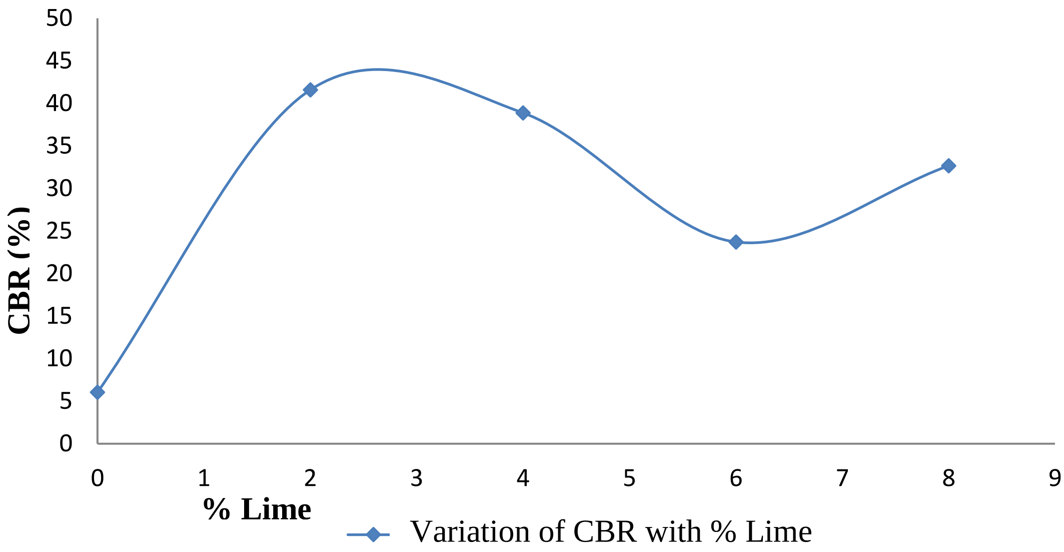

Figure 4.9: Variation of CBR values of soil with % lime.

From the result obtained the CBR value increases from 6.04% to 32.67% at 8% lime with a maximum value of 41.58% obtained at 2% Lime as shown in Table D3 which was taken as the optimum lime content for the soil sample being tested. According to Chulmin and Anthonio (2008) for long term pozzolanic reaction to take place, a higher amount of the additive should be used for stabilisation which in this case is 4% instead of 2%.

4.2.4 Unconfined compressive strength (UCS) characteristics

Graph of the Results obtained from the unconfined compressive strength test is presented in Figure E1, Appendix E. The variation of the UCS value with percentage of lime content in the soil is presented in Figure 4.10 and tabulated in Table E3. This shows that, as the percentage of lime increased up to 8%, there was an overall increase in the UCS value. This increase in strength must have resulted from increased bonding and adhesion of intermolecular particles by the hydration process of the lime.

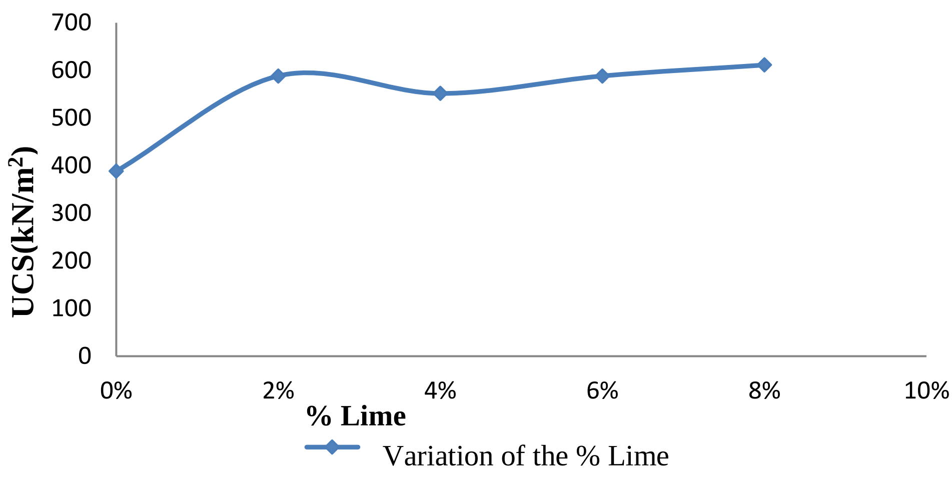

Figure 4.10: Variation of UCS values of soil with % lime

From the result obtained, the UCS value increased from 388.38 kN/m^2 to 588.29 kN/m^2 at 2% lime content, decreased at 4% and then continues to increase as seen in Figure 4.10.

For long term pozzolanic reaction and permanent development of strength to occur, usually the amount of additive to use must be above the optimum. Hence lime additive above 2% lime is recommended (Chulmin and Anthonio, 2008). For this study, 4% lime instead of the optimum content of 2% was combined with the plastic waste for further testing of the unconfined compressive strength of the soil. The Lime fixation point (Lm) value of approximately 3% as calculated from equation 2.8 which is above 2% supports with the use of 4% lime as optimum.

4.3 Result of the Soil Sample Treated with Lime – Plastic Waste (PW) Blend

4.3.1 Plasticity characteristics

The addition of lime-plastic blend to the soil resulted in an overall increase in the liquid limit from 47% to 51% at 6% Lime + 4% Plastic blend as presented in Table B6 to Table B9, Appendix B. There was an increase in the plastic limit from 26.14 to 50.346% and decrease in the plasticity index from 20.86 to 1% at 6% Lime + 3% Plastic as seen in

Table B8. Also, the plastic limit dropped to 49.2% and the plasticity index increased to 2.2% at 6% Lime + 4% Plastic as presented in Table B9.

The graph of variation of the liquid limit, plastic limit and plasticity index of sample with different lime-plastic content is presented in Figure 4.11.

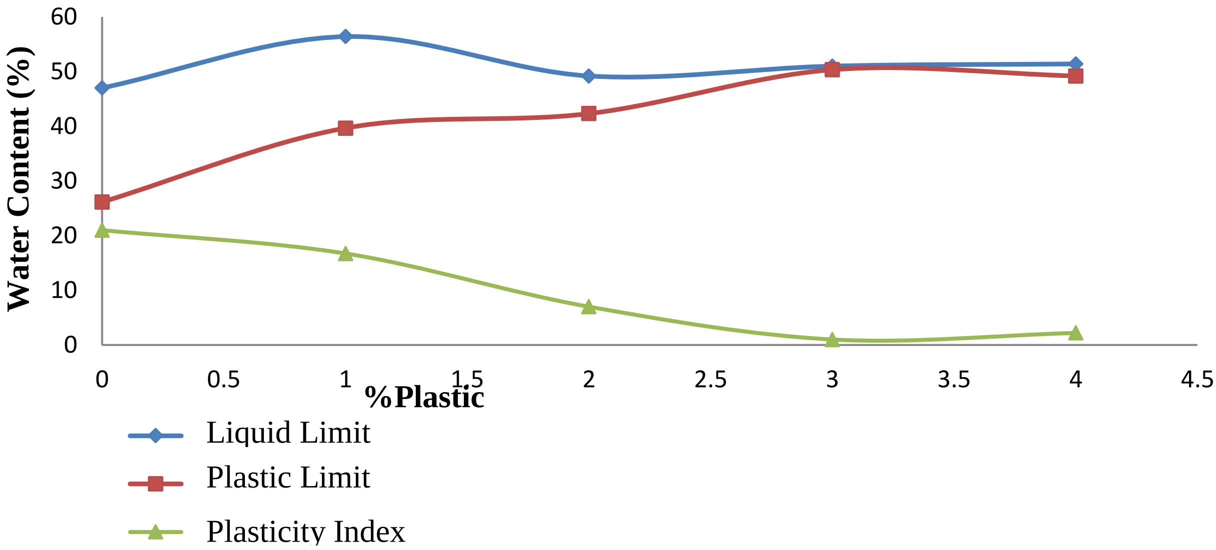

Figure 4.11: Variation of Atterberg limits of soil with % lime and %plastic blend

The plasticity index was seen to decrease up to 6% lime + 3% plastic which were the amount of the blend that gave the best reduction in the soil plasticity and hence is recommended for the soil in this study. Further addition of lime resulted in increase of the plasticity index. From the graph in Figure 4.11, it is seen that there was an overall decrease in the plasticity index of the soil. The reduction in the plasticity is attributed to the change in the soil's nature (its granular nature after flocculation and agglomeration) and the modified soil is as crumbly as silt soil, which is characterized by a low surface area and a low liquid limit because of the plastic nature of the lime (Ibtehaj *et al.*, 2014). According to Bello *et al.* (2015), these results indicate that the clay is of intermediate plasticity in nature. High plasticity is an indicator of potential swelling. Clay is prone to large volume changes if PI is greater than or equal to 30% (Amu *et al.*, 2005). The plasticity of the soil was decreased which gave an indication of a more stable soil with marked increased workability.

4.3.2 Compaction characteristics

Compaction result for the soil with lime of different combinations is presented in Figure C2. The result obtained from the compaction test indicates that, the optimum moisture content of the soil increases from 15.3% to 19.5% with the increase of lime-plastic content to 6% lime + 4% plastic. There was an overall decrease in the maximum dry density values from 1.915g/cm^3 to 1.7132g/cm^3 at 6% lime + 4% plastic. These values are presented in Table C11. The effect of variation of the percentage lime content + plastic waste on soil is shown in Figures 4.12a and 4.12b for the optimum moisture content and the maximum dry density respectively. The decrease in the dry unit weight may be due to the lower specific gravity of the lime and plastic, while an increase in the optimum moisture content may be as a result of water needed to be hydrated (Wajid *et al*., 2016).

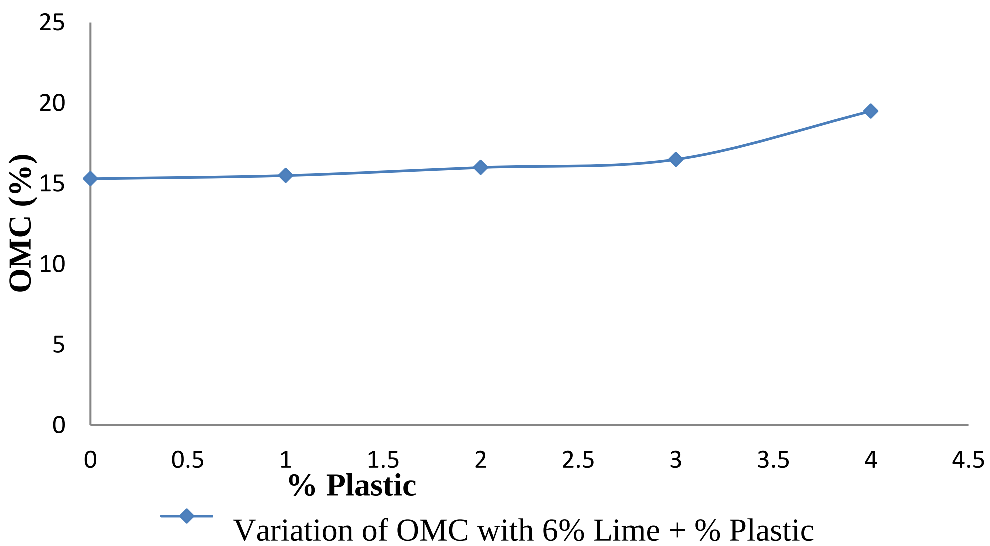

Figure 4.12a: Variation of optimum moisture content with different % plastic

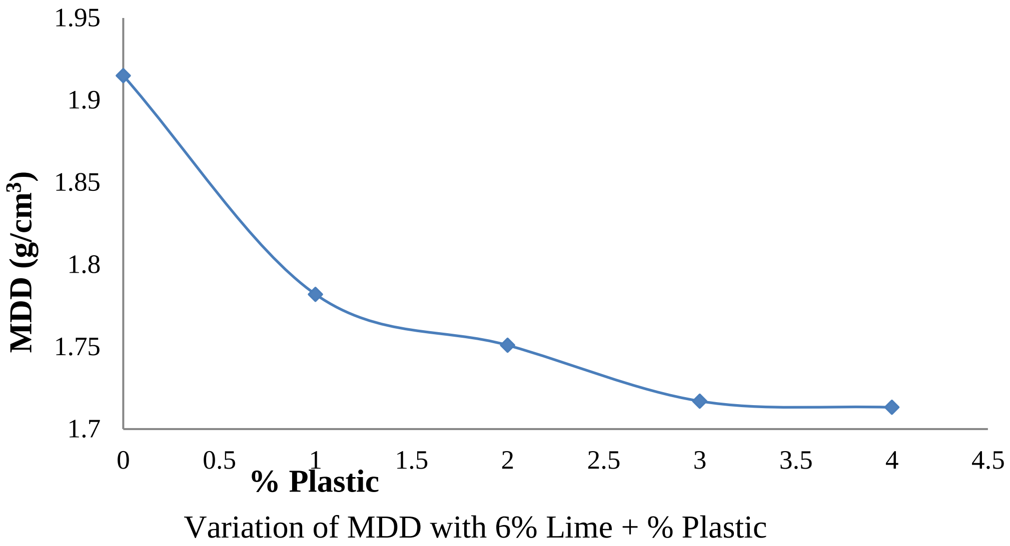

Figure 4.12b: Variation of maximum dry density values with varied % plastic

This decrease in MDD may also be attributed to the flocculation and agglomeration of clay particles, due to cation exchange leading to corresponding decrease in dry density. The increase in optimum moisture content (OMC) may be due to increased demand for water, which commensurate with higher amount of lime required for hydration reaction and dissociation needed for cation exchange reaction (Sadeeq *et al.*, 2015).

4.3.3 California bearing ratio (CBR) of test samples

From the laboratory results obtained, the load vs. penetration curves were drawn for various percentages of lime + plastics in the soil and are presented in Figure D2 in Appendix D. The variation of the CBR values with increase in the lime and plastic blend content are tabulated in Table D4 and plotted in Figure 4.13.

Increase in the CBR may be due to the pozzolanic effect of the additive in the reaction. Decrease after optimum value is attributed to the low strength exhibited by the additive (Wajid *et al.*, 2016).

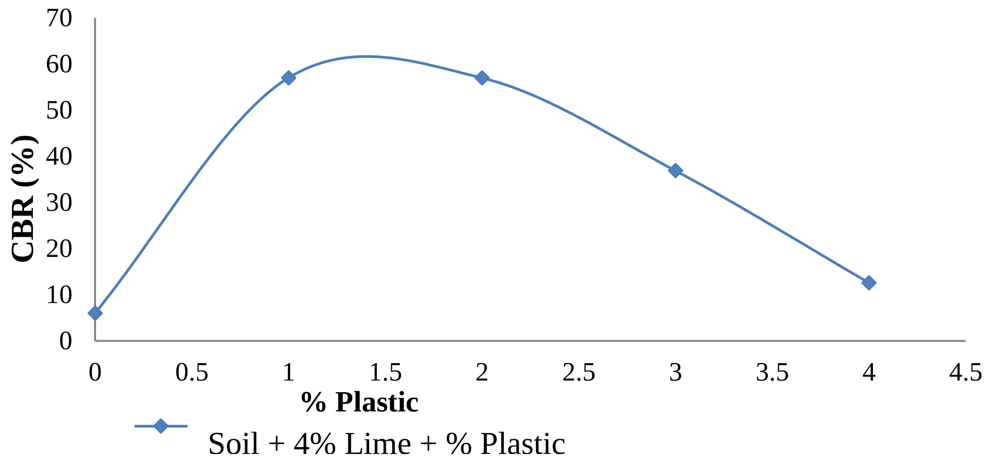

Figure 4.13: Variation of CBR values of soil with varied % Plastic

From Table D4, the unsoaked CBR value of the natural soil sample was 6.04% and the CBR value increased to 57.03% with the addition of 1% plastic + 4% lime. The increase in strength can be attributed to the formation of cementing materials or binders. Unsoaked CBR values of sample at optimum states are 38.8% on addition of 4% lime alone and 57.03% on addition of the 4% lime + 1% plastic blend. These values adequately meet the requirements for sub base by the Federal Ministry of Works and Housing (1997) which states that it is 30%. Increase in values of CBR on addition of the lime-plastic blend, when compared to stabilising with lime of a specific gravity of 2.4 alone, may be attributed to the additional specific gravity of the plastics which is 1.05 (Anand and Vageesh, 2017). Also, same can be seen for the case with the UCS result obtained after the test which were 551.59kN/m^2 at 4% lime and 792.2 kN/m^2 at 4% lime + 1% plastic.

4.3.4 Unconfined compressive strength (UCS) of test samples

Graph of the Results obtained from the unconfined compressive strength test is presented in Figure E2 in Appendix E. It can be seen from this figure that as the percentage of lime-plastic blend increased there was an overall increase in the UCS value. The maximum UCS value of 792.24 kN/m^2 was obtained at 4% lime + 1% plastic additive which is higher than at of the optimum lime content of 551.59kN/m^2. The variation of the UCS values with percentages of the lime-plastic blend content in the soil shows there was an overall increase in strength. This increase in strength must have resulted from increased bonding and adhesion of intermolecular particle by the hydration process of lime. This is presented in Figure 4.14 and tabulated in Table E5

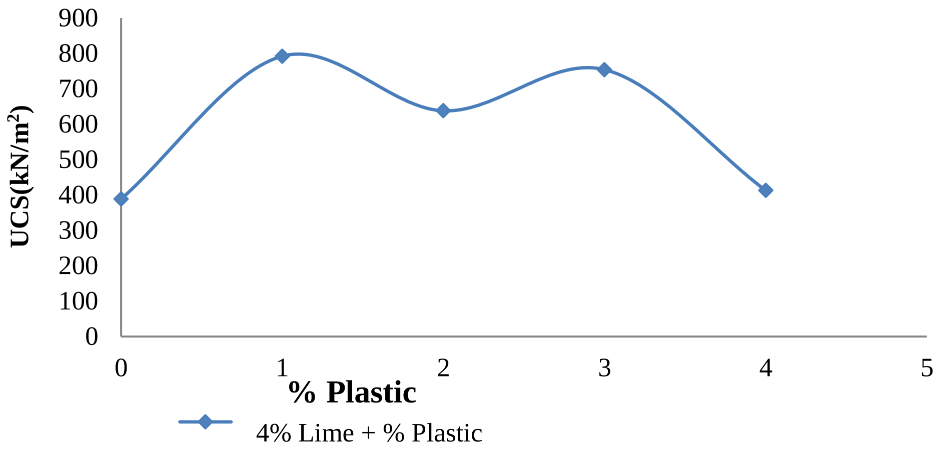

Figure 4.14: Variation of UCS values of treated soil with 4% Lime + %Plastic

4.4 Result of UCS for Different Curing Days

The result of the unconfined compressive strength test obtained after curing the samples for 7, 14 and 28 days and the strength developed are presented in this section of the study. The optimum of 4% lime was combined with the various percentages (1, 2, 3 and 4%) of plastic and used for the test. The graph of Axial Stress vs. Axial Strain for varied percentage of lime + Plastic Waste (PW) for the various curing days are presented in Figures E3 to E5 in Appendix E. The variation of the UCS values with different percentages of Plastic Waste (PW) and Lime Blend for the different curing days is presented in Table E9 and Figure 4.25.

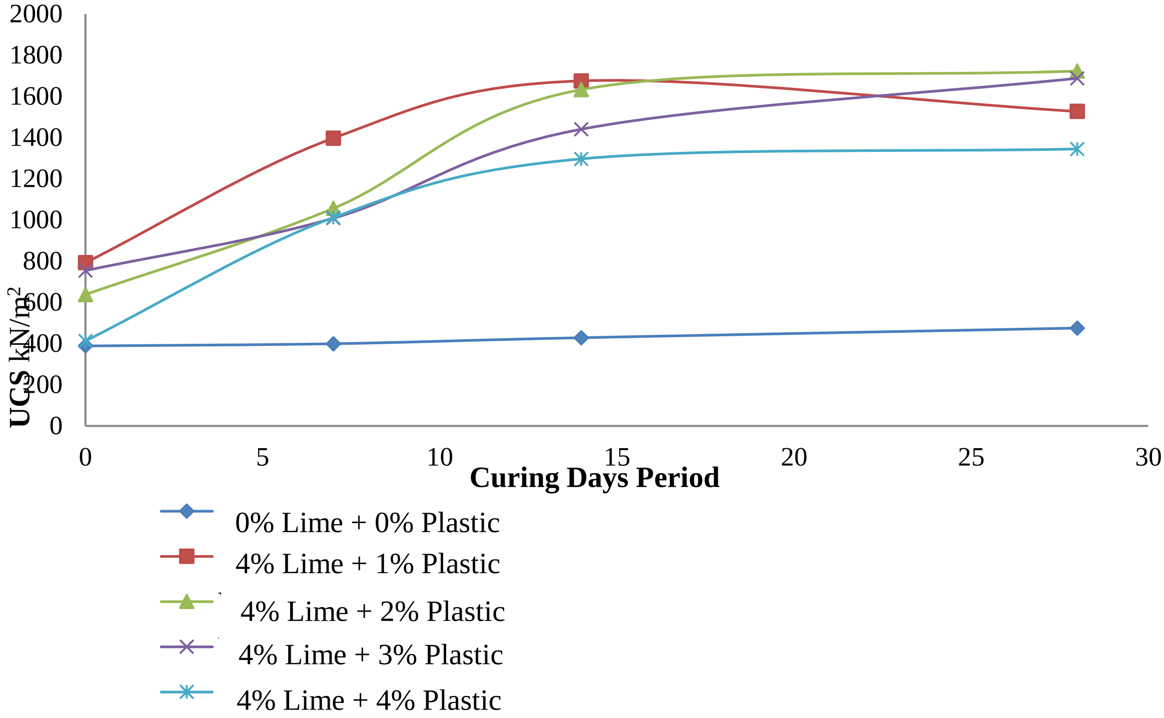

Figure 4.15: Variation of UCS with days of curing for different percentages of lime + plastic blend

The increase in the UCS values was mainly due to the formation of some compounds such as Calcium Silicates Hydrates (CSH) and Calcium Aluminate Hydrates (CAH) which are the major compounds responsible for strength gain (Sadeeq *et al.*, 2015).

The Unconfined Compression Strength increased from 388.38 to 1397.24 kN/m^2 for the treated soil at 4% lime + 1% plastic waste after curing for 7 days which satisfies the specification of 750 kN/m^2 – 1500 kN/m^2 in the Federal Ministry of Works Highway manual. From the Nominal strength classification of materials in the design catalogue in Table F3 in Appendix, UCS for sub-base after curing for 7 days is 0.75 – 1.5 MPa which when converted is equivalent to 750 kN/m^2 – 1500 kN/m^2 at 100% modified AASHTO density. UCS value was increased from 388.38 kN/m^2 to 1723.16 kN/m^2 after curing for 28 days on addition of 4% lime + 2% plastic blend. This was taken as the optimum and recommended from this study for stabilising the soil.

4.5 Result of the Microstructural Properties of the Soil and the Additives

Results obtained from X-RD for the mineralogy of the soil shows that the predominant clay minerals in the soil are Kaolinite, Illite and Quartz as presented in Figure 4.16 a. The result obtained for lime shows that it contains Portlandite and Calcite as shown in Figure 4.16 b. The result showing the chemical composition of the soil from the chemical analysis using the X-Ray Fluorescence technique conducted on the soil and lime is presented in Table 4.2 showing their percentage oxide composition. The major percentage oxide composition of the soil is Fe_2O_3 of about 20.726%, Al_2O_3 of 16.29 and 58.75% of SiO_2.

Table 4.2: Oxide Composition (%) for the Natural Soil and the Lime

Oxide	Concentration (%)	
	Soil	Hydrated Lime
Fe_2O_3	20.726	0.286
MnO	0.053	0.012
Cr_2O_3	0.06	0.0099
TiO_2	2.164	0
CaO	0.89	95.33
Al_2O_3	16.29	1.451
MgO	1.03	0
ZnO	0.013	0.001
SiO_2	58.75	2.91

From equation (2.1), the Silica - Sesquioxide Ratio (S/R) of this soil is 3.38 which make it to be considered as a Non – Lateritic tropically weathered soils because it is greater than the limit of 2 for a lateritic soil.

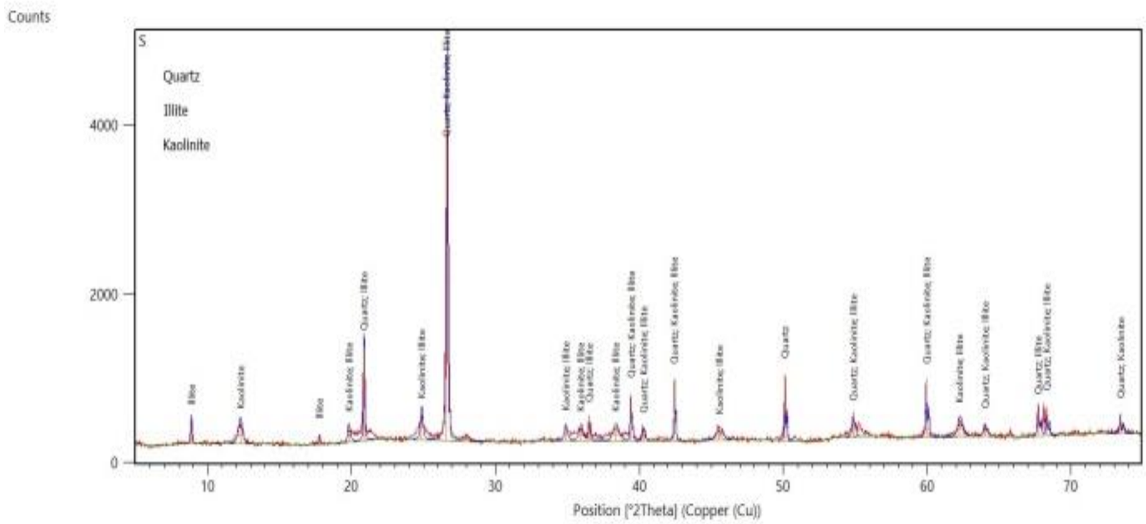

Figure 4.16 a: X-Ray Diffractogram of the natural soil

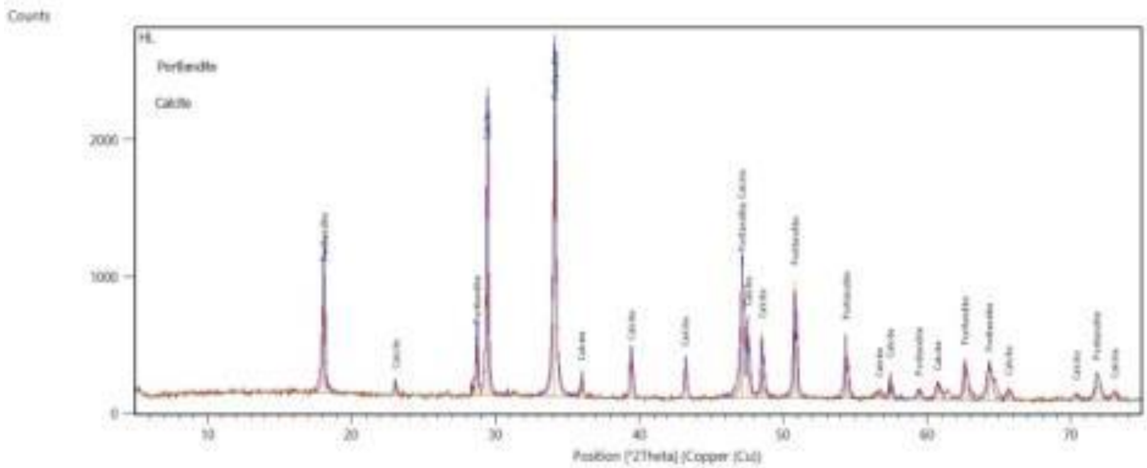

Figure 4.16 b: X-Ray Diffractogram of the hydrated Lime

The chemical composition of the plastics using Scanning Electron Microscopy – Energy Dispersion Spectroscopy (SEM -EDS) of the plastic materials are presented in Table 4.3 and the image of plastic material obtained from the Scanning Electron Microscopy is also presented in Plate II while the weight concentration of the elements present in the plastic material is presented in Figure 4.17.

Table 4.3: Properties of Plastic Material

Element Number	Element Name	Weight Concentration
6	Carbon (C)	58.05
14	Silicon (Si)	8.55
20	Calcium (Ca)	6.38
26	Iron (Fe)	6.21
13	Aluminium (Al)	5.73
11	Sodium (Na)	2.43
12	Magnesium (Mg)	2.29
47	Silver (Ag)	1.86
16	Sulphur (S)	1.59
19	Potassium (K)	1.58
22	Titanium (Ti)	1.38
17	Chlorine (Cl)	1.38
15	Phosphorus (P)	1.18
30	Zinc (Zn)	0.98
7	Nitrogen (N)	0.41

The specific gravity of the plastic material is 1.06

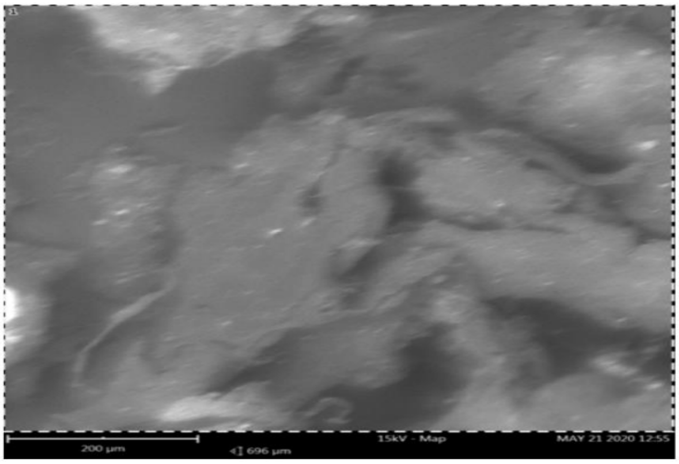

Plate II: Image of the material obtained from the Scanning Electron Microscopy

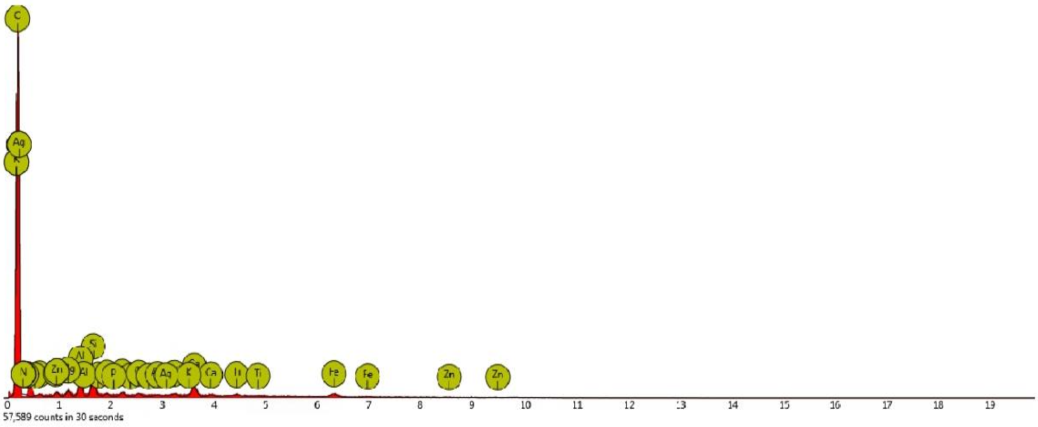

Figure 4.17: Weight concentration of the elements present in the plastic material

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

In this study, the lime and plastic waste blend improved the index and engineering properties of the soil. The use of plastic and lime blend was found to be useful in improving the properties of the soil as a stabiliser and thus reducing nuisance it causes to the environment.

For the index properties of the soil, result obtained shows that the soil is an A-7-6b soil, clay of low plasticity (CL) with a Plasticity index (PI) of 20.86. When treated with the blend the PI reduced to as low as 1 at 6% lime + 3% plastic hence improving the soil workability and stability. The compaction test result shows that increase in the stabilising agent resulted in decrease in maximum dry density (MDD) of the soil and increase in the optimum moisture content (OMC) of the soil.

The strength of the stabilised soil obtained from the UCS and CBR tests showed appreciable increase from 388.4 kN/m2 to 792.2 kN/m^2 and 6.04 to 57.03% respectively. This meet the minmum requirement of 750 kN/m^2 for UCS and 30% for CBR value for sub-base in pavement base on the specification in Federal Ministry of Works Highway manual. The UCS value increased from 388.38 to 1397.24 kN/m^2 at 4% lime + 1% plastic after curing for 7 days, which satisfied the specification in Federal Ministry of Works Highway manual of 750 kN/m^2 to 1500 kN/m^2. An optimum of 1723.2 kN/m^2 was obtained at 4% lime + 2% Plastic after curing for 28 days. For long term pozzolanic reaction and permanent development of strength, usually the amount of additive to use should be above the optimum. The optimum blend selected for the overall stabilisation and improvement of the engineering properties of the lateritic soil is 6% lime + 3% plastics. This is recommended for effective stabilisation and improvement of the soil PI, MDD, CBR and UCS. The long-term reaction continues for years, and the strength and/or stiffness of the lime-treated soil increases with time (Chulmin and Anthonio, 2008).

The major clay minerals present in the soil from the X-Ray Diffraction test include Kaolinite, Illite and Quartz with Kaolinite being the predominant mineral. Also the soil has a reddish – pinkish brown colour. From the X-Ray Fluorescence test, the major percentage oxide compositions of the soil are Fe_2O_3 of about 20.726%, Al_2O_3 of 16.29 and 58.75% of SiO_2.

From this study, the blend of lime and plastics improved and stabilised the lateritic soil tested. Studies have shown that waste plastics combined with lime acceptably acts as a cheap stabilising material, reducing the nuisance caused by plastic wastes in the environment. Hence this makes it a good alternative to reduce construction cost particularly in the rural areas of developing countries.

5.2 Recommendations

From the investigation of the work carried out, the following recommendations were obtained

1. For improvement of the strength properties of the treated soil, a blend of the additive (4% lime + 2% plastics) should be added to the soil in road construction.
2. The optimum blend of lime and plastic recommended from this study which satisfies for the overall stabilisation of the lateritic soil is 6% lime + 3% plastics for construction purposes generally.

3. Since the optimum plastic was obtained at 1% plastics from the UCS and CBR tests result, hence lower percentages of plastics (0.5, 1, 1.5 and 2%) combine with the optimum lime can be investigated for further studies on the effect of the lime- plastic blend on the strength properties of the soil.

5.3 Contribution to Knowledge

The study established that the strength of the lateritic soil stabilised with a blend of hydrated lime and waste plastic granules increase the Unconfined Compressive Strength and California Bearing Ratio of the composite from 388.4 kN/m^2 to 1723.2 kN/m^2 (77.5%) and 6.04 to 57.03% (89.4%) respectively. It also established a productive and economic use of use of a hitherto waste material in soil stabilisation.

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APPENDICES

APPENDIX A: Natural Moisture, Specific Gravity and Sieve Analysis Test Data

Table A1: Natural Moisture Content of Soil

No. of trial	1	2	3	4
Can No.	G42	AA5	M7	H2
Wt. of can (g)	37.7	38.3	38.4	38.2
Wt. of can + WS (g)	86.2	86.6	103.1	107.1
Wt. of can + DS (g)	77.4	78	91.2	94
Wt. of Water (g)	8.8	8.6	11.9	13.1
Wt. of DS (g)	39.7	39.7	52.8	55.8
% Moisture content	22.17	21.66	22.54	23.48
Average Moisture Content (%)		22.46		

Table A2: Specific Gravity of Soil

No. of trial	1	2	3
Mass of empty pycnometer M1 (g)	48.8	45.9	43.5
Mass of empty pycnometer + dry soil M2 (g)	87.6	84.1	80.8
Mass of empty pycnometer + dry soil + water M3 (g)	171.4	168.4	165.4
Mass of empty pycnometer + water M4 (g)	147.6	144.7	142.1
Specific Gravity Gs	2.59	2.63	2.66
Average Specific Gravity Gs		2.63	

Table A3: Specific Gravity of Lime

No. of trial	1	2	3
Mass of empty pycnometer M1 (g)	76.8	66.8	43.5
Mass of empty pycnometer + dry soil M2 (g)	109.5	98.5	63.7
Mass of empty pycnometer + dry soil + water M3 (g)	293.6	281.7	154.1
Mass of empty pycnometer + water M4 (g)	274.4	263.4	142.4
Specific Gravity Gs	2.42	2.366	2.38
Average Specific Gravity Gs		2.4	

Table A4: Specific Gravity of Plastic Material

No. of trial	1	2	3
Mass of empty pycnometer M1 (g)	76.7	66.8	-
Mass of empty pycnometer + dry soil M2 (g)	93.1	81.4	-
Mass of empty pycnometer + dry soil + water M3 (g)	275.5	263.4	-
Mass of empty pycnometer + water M4 (g)	274.6	262.7	-
Specific Gravity Gs	1.06	1.05	-
Average Specific Gravity Gs		1.05	

Table A5: Sieve Analysis Data obtained from the Laboratory

Sieve size (mm)	Weight of soil retained (g)	% Weight retained (%)	%Cumulative weight retained	% Passing (%)
5.0	21.1	7.03	7.03	92.97
3.35	3.4	1.13	8.16	91.84
2.36	3	1	9.16	90.84
2.0	2	0.67	9.83	90.17
1.18	11	3.67	13.5	86.5
0.85	10.8	3.6	17.1	82.9
0.6	16	5.33	22.43	77.57
0.425	13.1	4.37	26.8	73.2
0.3	10.4	3.47	30.27	69.73
0.15	21.6	7.2	37.47	62.53
0.075	4	1.33	38.8	61.2
0.006	2.7	0.9	39.7	60.3
pan	180.9	60.3	100	0
Total weight	300			

APPENDIX B: Atterberg Limit Tests Result

The data obtained for the LL and PL were analysed from the equation 3.1 for %Moisture Content.

Table B1: Atterberg Limits for the Natural Soil Sample

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	58	99	118	134	196	228		
Can No.	B6	A4	B1	A9	A8	B2	A6	N2
Wt. of can (g)	21.9	19.9	21.6	22.1	22.5	18.2	21.9	39.1
Wt. of can + WS (g)	27.6	27	29.5	27.2	31.1	30.8	24.7	41.6
Wt. of can + DS (g)	26.3	25	27.2	25.7	28.4	26.8	24.1	41.1
Wt. of water (g)	1.3	2	2.3	1.5	2.7	4	0.6	0.5
Wt. of DS (g)	4.4	5.1	5.6	3.6	5.9	8.6	2.2	2
% Moisture content	29.55	39.22	41.07	41.67	45.76	46.51	27.27	25
LL (%)			47				Average PL=26.14	
PI			20.86					

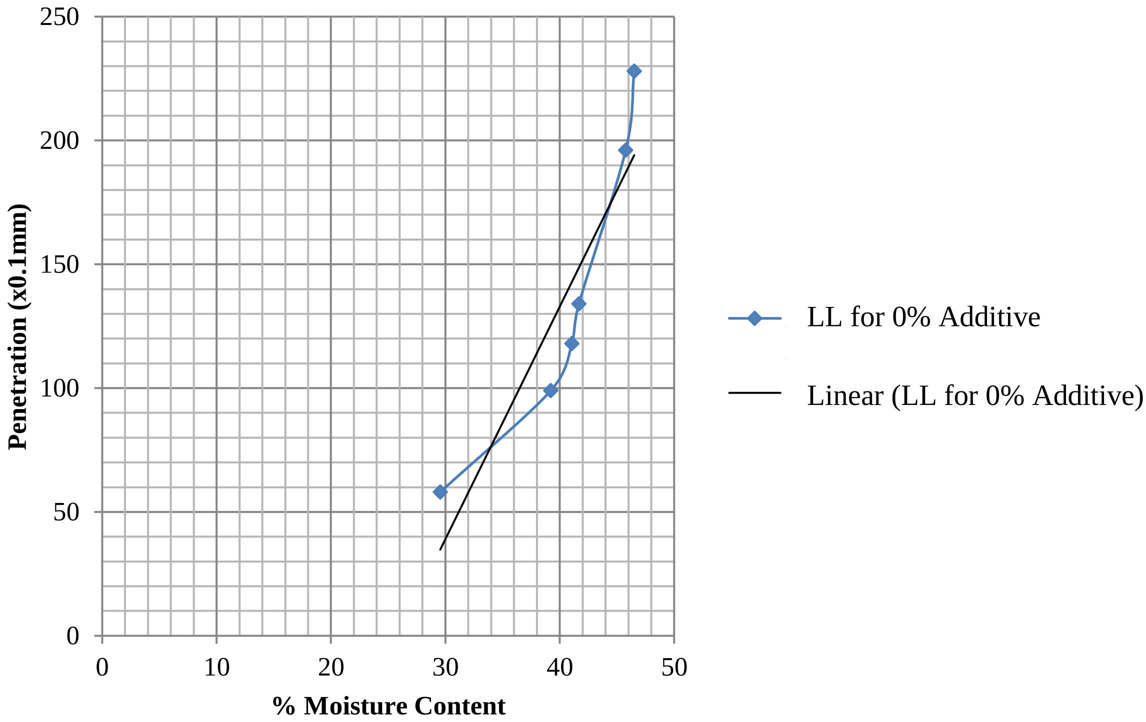

Figure B1: Liquid Limit curve for the untreated natural soil

Table B2: Atterberg Limits for soil treated with 2% Lime

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	43	100	163	220	265	-		
Can No.	11S	Eo	F11	DO7A	Z90	-	P21	T2
Wt. of can (g)	24.9	24	24.4	24.3	25.3	-	38.9	38.2
Wt. of can + WS (g)	33.8	31.4	36.2	38.3	37.4	-	41.0	40.1
Wt. of can + DS (g)	31.7	29.3	32.4	33.4	33	-	40.5	39.7
Wt. of water (g)	2.1	2.1	3.8	4.9	4.4	-	0.5	0.4
Wt. of DS (g)	6.8	5.3	8	9.1	7.7	-	1.6	1.5
% Moisture content	30.88	39.62	47.5	53.85	57.14	-	31.25	26.67
LL (%)			52				Average PL = 28.96	
PI			23.04					

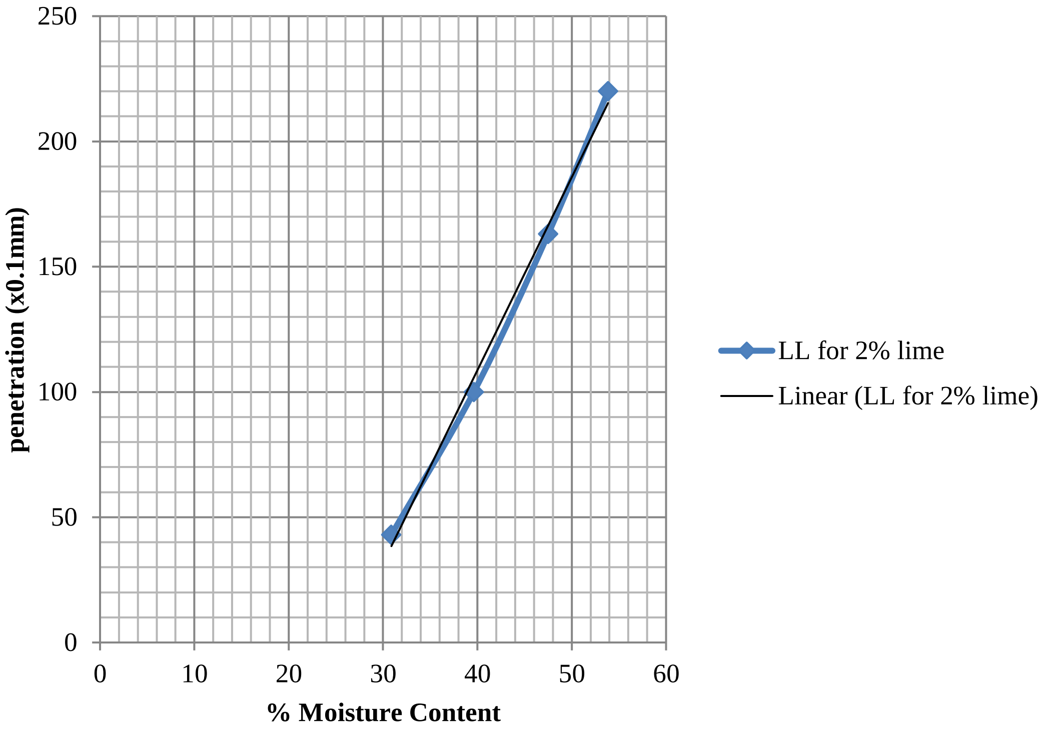

Figure B2: Liquid Limit curve for soil treated with 2%lime

Table B3: Atterberg Limits for soil treated with 4% Lime

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	66	157	190	237	-	-		
Can No.	M17	M7	N6	D16	-	-	PQ4	NH3
Wt. of can (g)	38.4	38.6	19.9	39.7	-	-	38.2	38.6
Wt. of can + WS (g)	51.5	56	33.4	54.8	-	-	41	41.8
Wt. of can + DS (g)	47.8	50.7	29.2	49.9	-	-	40.4	41.1
Wt. of water (g)	3.7	5.3	4.2	4.9	-	-	0.6	0.7
Wt. of DS (g)	9.4	12.1	9.3	10.2	-	-	2.2	2.5
%Moisture content	39.36	43.80	45.16	48.04	-	-	27.27	28
LL (%)			46				Average PL = 27.64	
PI			18.36					

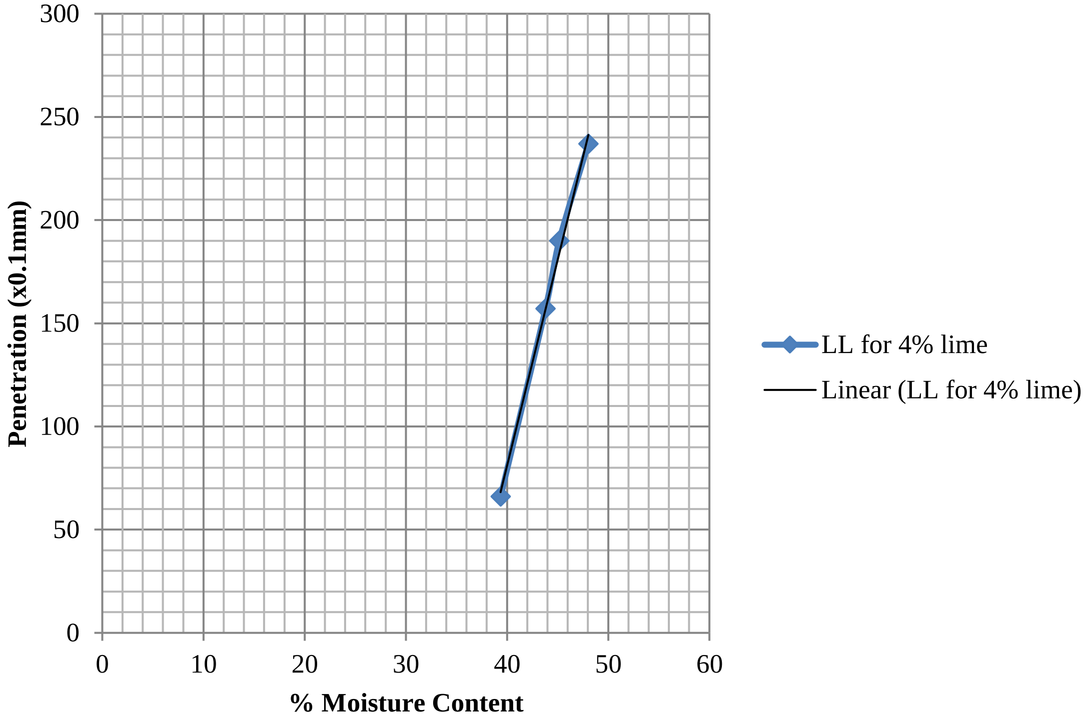

Figure B3: Liquid Limit curve for soil treated with 4%lime

Table B4: Atterberg Limits for soil treated with 6% Lime

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	35	50	131	144	211	-		
Can No.	T45	B31	E09	C9	G8	-	G7	Q31
Wt. of can (g)	24.5	24.6	24.6	25.1	25	-	24.7	35.3
Wt. of can + WS (g)	34.4	32.9	42.3	39.4	41.4	-	28.1	38.6
Wt. of can + DS (g)	32.1	30.8	37.3	35.1	36.1	-	27.4	37.8
Wt. of water (g)	2.3	2.1	5	4.3	5.3	-	0.7	0.8
Wt. of DS (g)	7.6	6.2	12.7	10	11.1	-	2.7	2.5
%Moisture content	30.26	33.87	39.37	43	47.75	-	25.93	32
LL (%)			47				Average PL = 28.96	
PI			18					

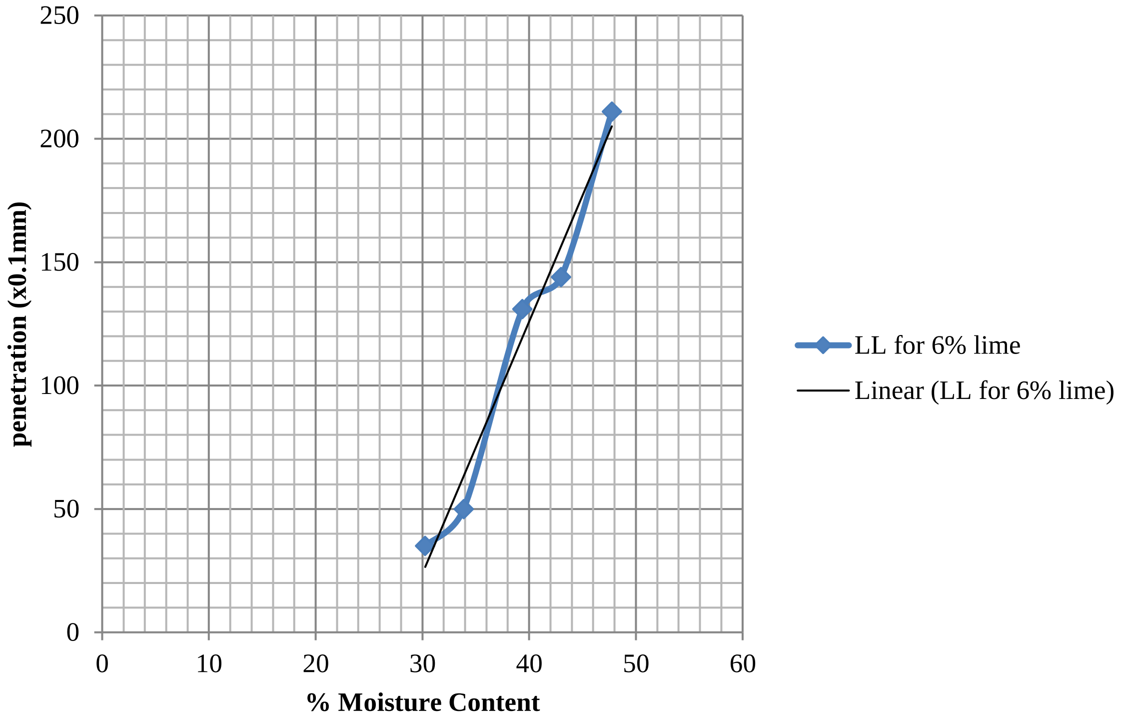

Figure B4: Liquid Limit curve for soil treated with 6%lime

Table B5: Atterberg Limits for soil treated with 8% Lime

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	20	65	80	118	196	223		
Can No.	T2A	CO91	M4	H2	B10	E8		
Wt. of can (g)	38.2	35.2	34.8	38.7	27.2	24.5	25.2	25.5
Wt. of can + WS (g)	49.5	47.9	48	57.8	43.6	40.9	26.7	26.7
Wt. of can + DS (g)	47.4	44.8	44.2	51.8	38.6	35.7	26.4	26.4
Wt. of water (g)	2.1	3.1	3.8	6	5	5.2	0.3	0.3
Wt. of DS (g)	9.2	9.6	9.4	13.1	11.4	11.2	1.2	0.9
%Moisture content	22.83	32.29	40.43	43.86	45.80	46.43	25	33.33
LL (%)			50				Average PL = 29.17	
PI			20.83					

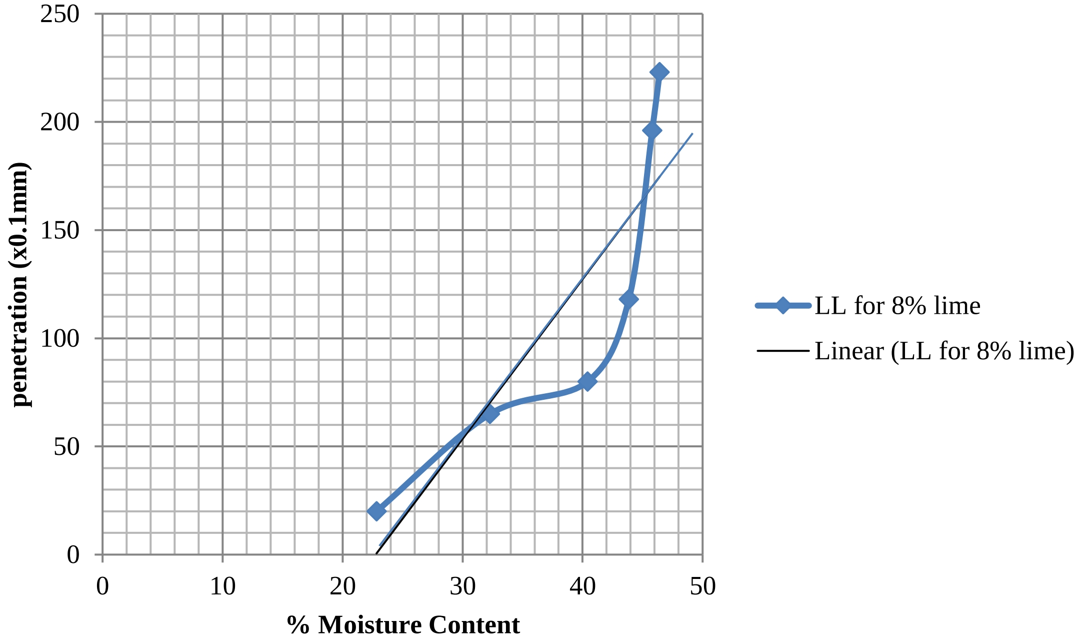

Figure B5: Liquid Limit curve for soil treated with 8%lime

The data obtained for the LL and PL were analysed from the equation 3.1 for %moisture content of 6% Lime + % Plastic Waste (PW).

Table B6: Atterberg Limits for soil treated with 1% PW + 6% Lime

	LL							PL	
No. of trial	1	2	3	4	5	6	7	1	2
Penetration (x0.1mm)	60	106	142	171	187.5	192	253		
Can No.	D16	T3	N2/5	M7	M10	Q7	Q3	P2	M3
Wt. of can (g)	39.72	38.21	39.15	38.72	38.31	38.14	35.35	38.37	38.73
Wt. of can + WS (g)	50.15	50.28	50.14	49.92	55.28	48.89	51.45	39.87	39.92
Wt. of can + DS (g)	47.3	44.9	46.72	45.96	49.26	44.94	45.27	39.4	39.62
Wt. of water (g)	2.85	5.38	3.42	3.96	6.02	3.95	6.18	0.47	0.30
Wt. of DS (g)	7.58	6.69	7.57	7.24	10.95	6.8	9.92	1.03	0.89
%Moisture content	37.6	40.42	45.18	54.7	54.98	58.09	62.3	45.36	33.71
LL (%)	56.4							Average PL =39.67	
PI	16.73								

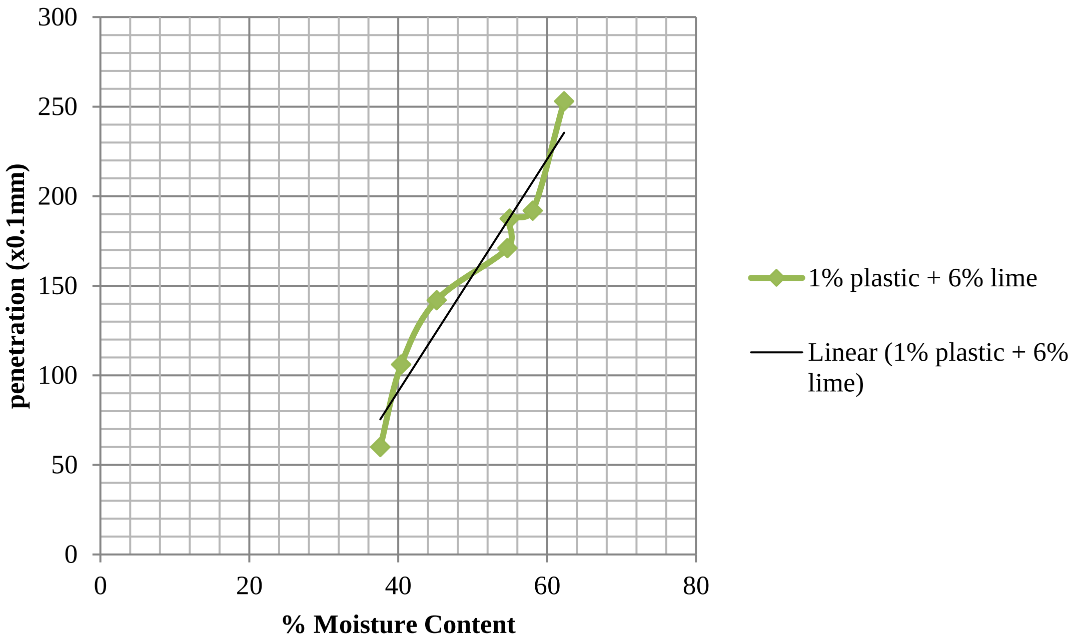

Figure B6: Liquid Limit curve for soil treated with 6%lime + 1% plastic

Table B7: Atterberg Limits for soil treated with 2% PW + 6% Lime

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	76	132	186	250				
Can No.	RM42	RM17	RM36	RM20	-	-	R37	RM8
Wt. of can (g)	23.03	24.31	22.90	24.25	-	-	24.19	23.38
Wt. of can + WS (g)	32.63	34.20	33.26	37.57	-	-	25.47	25.05
Wt. of can + DS (g)	29.70	31.23	30.12	33.06	-	-	25.10	24.54
Wt. of water (g)	2.93	2.97	3.14	4.51	-	-	0.37	0.51
Wt. of DS (g)	6.67	6.92	7.22	8.81	-	-	0.91	1.16
%Moisture content	43.49	45.92	48.93	51.19	-	-	40.66	43.96
LL (%)			49.2				Average PL = 42.31	
PI			7					

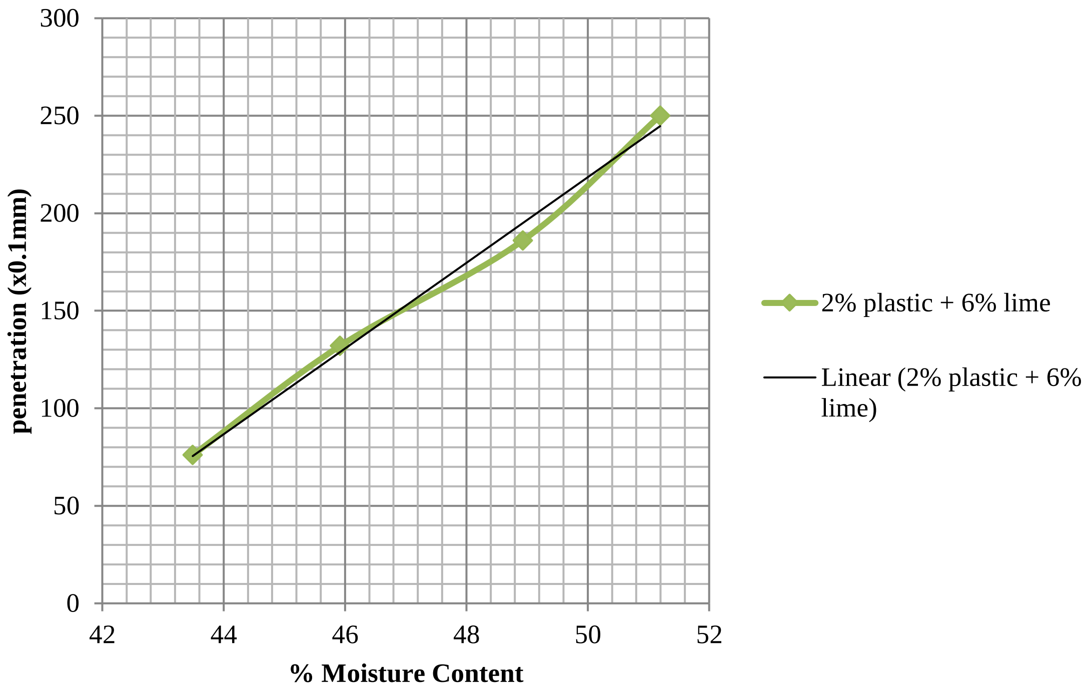

Figure B7: Liquid Limit curve for soil treated with 6%lime + 2% plastic

Table B8: Atterberg Limits for soil treated with 3% PW + 6% Lime

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	45	60	98	192	240	265		
Can No.	S4	PQ4	YY	T2A	ST2	M4	8K	ZZ
Wt. of can (g)	38.79	38.19	39.11	38.32	39.05	34.79	24.75	23.35
Wt. of can + WS (g)	45.75	47.31	48.77	49.53	53.48	48.53	26.38	24.95
Wt. of can + DS (g)	44	46.7	45.6	45.8	48.5	43.8	25.8	24.45
Wt. of water (g)	1.75	0.16	3.17	3.73	4.98	4.73	0.58	0.5
Wt. of DS (g)	5.21	8.51	6.49	7.48	9.45	9.01	1.05	1.1
%Moisture content	33.59	37.17	48.84	49.87	52.7	52.5	55.24	45.45
LL (%)			51				Average PL = 50.34	
PI			1					

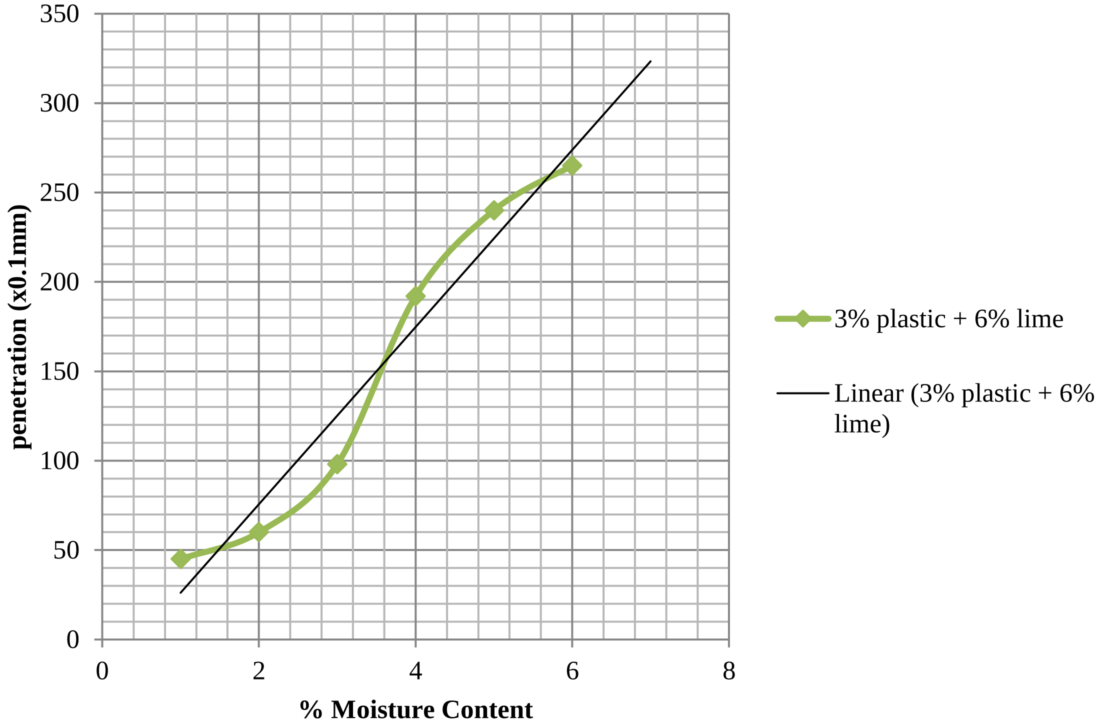

Figure B8: Liquid Limit curve for soil treated with 6%lime + 3% plastic

Table B9: Atterberg Limits for soil treated with 4% PW + 6% Lime

	LL						PL	
No. of trial	1	2	3	4	5	6	1	2
Penetration (x0.1mm)	50	60	105	197	219	255		
Can No.	G7	SI	A7	P21	NH3	AoJ2	K4	KM3
Wt. of can (g)	25.01	25.35	37.88	38.92	38.59	24.77	25.12	24.82
Wt. of can + WS (g)	34	35.52	50.00	55.10	54.19	39.19	27.06	26.48
Wt. of can + DS (g)	31.75	32.46	46.26	49.70	48.88	34.07	26.35	26.0
Wt. of water (g)	2.25	3.06	3.74	5.4	5.31	5.12	0.71	0.48
Wt. of DS (g)	6.74	7.11	8.38	10.78	10.29	9.3	1.23	1.18
%Moisture content	33.38	43.04	44.63	50.09	51.60	55.05	57.72	40.68
LL (%)			51.4				Average PL = 49.2	
PI			2.2					

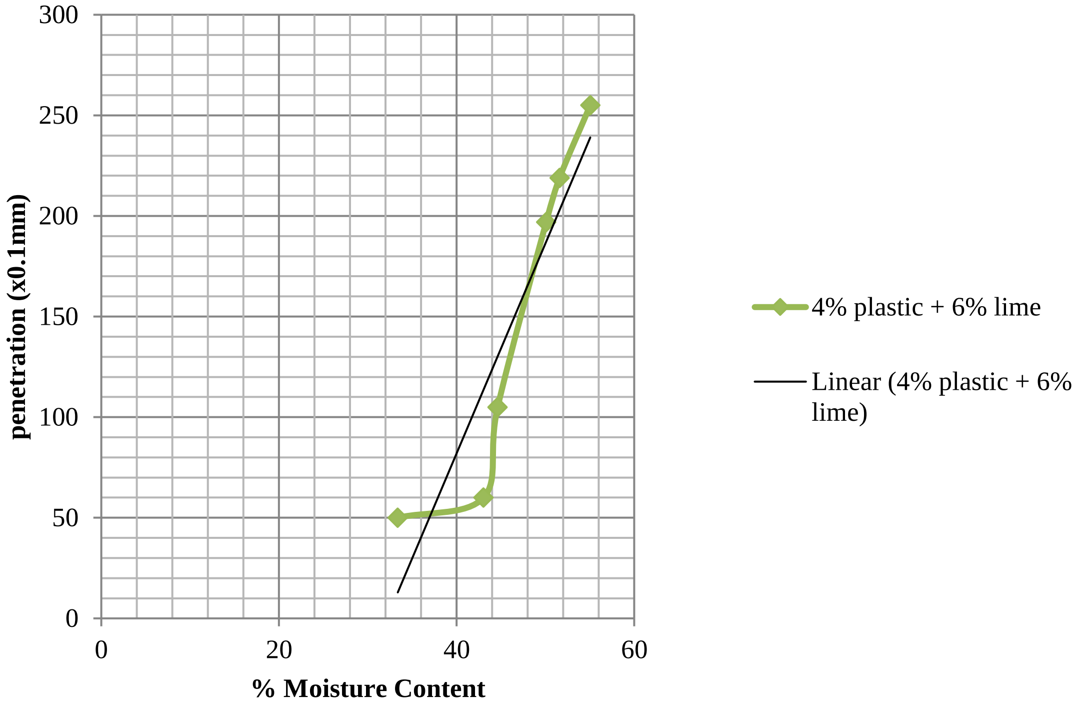

Figure B9: Liquid Limit curve for soil treated with 6%lime + 4% plastic

APPENDIX C: Compaction Test Data Obtained For The Soil Sample

Table C1: Result of Compaction Data for Untreated Soil Sample

Trials	1		2		3		4		5		6
Wt. of mould (g)	3678g		3678g		3678g		3678g		3678g		-
Wt. of mould + WS (g)	5563		5728		5874		5822		5738		-
Wt. of WS (g)	1885		2050		2196		2144		2060		-
Can no	B2	B6	A9	B1	A6	A8	A4	N2	QK	M4	-
Wt. of can (g)	18.2	21.9	22.2	21.6	21.9	22.5	19.9	39	38	34.8	-
Can +Ws (g)	32.2	42.7	48.4	41.4	49.5	48.3	48.7	73.1	76.8	68	-
Can + Ds (g)	31.2	41.5	45.7	39.3	45.9	44.8	44.3	67.8	70	62	-
Wt. of water (g)	1	1.2	2.7	2.1	3.6	3.5	4.4	5.3	6.8	6	-
Wt. of Ds (g)	13	19.6	23.5	17.7	24	22.3	24.4	28.8	32	27.2	-
% MC	7.7	6.1	11.5	11.9	15	15.7	18.03	18.4	21.3	22.1	-
Average % MC (%)	6.91		11.67		15.35		18.21		21.66		-
Dry Density (g/cm^3)	1.76		1.84		1.91		1.81		1.69		-

Table C2: Result of Compaction Data for Soil treated with 2% Lime

Trials	1		2		3		4		5		6
Wt. of mould (g)	3685		3685		3685		3685		3685		-
Wt. of mould + WS (g)	5535		5642		5815		5782		5705		-
Wt. of WS (g)	1850		1957		2130		2097		2020		-
Can no	ZZ	SD	Ao	DO4	7N	A61	C16	XXO2	S01	T01	-
Wt. of can (g)	23.39	19.9	24.6	24.8	24.6	24.5	25	24.8	24.7	25.2	-
Can +Ws (g)	41.3	37	45.1	42.3	46.2	39.7	51.5	46.3	55.9	55.9	-
Can + Ds (g)	16.9	16.2	18.8	15.9	19	13.3	22.3	18.1	25.4	24.9	-
Wt. of water (g)	1.1	0.9	1.7	1.6	2.6	1.9	4.2	3.4	5.8	5.8	-
Wt. of Ds (g)	13	19.6	23.5	17.7	24	22.3	24.4	28.8	32	27.2	-
% MC	6.51	5.55	9.04	10.06	13.68	14.29	18.83	18.78	22.83	23.29	-
Average % MC (%)	6.03		9.55		13.98		18.81		23.06		-
Dry Density (g/cm^3)	1.74		1.78		1.87		1.76		1.64		-

Table C3: Result of Compaction Data for Soil treated with 4% Lime

Trials	1		2		3		4		5		6	
Wt. of mould (g)	3685		3685		3685		3685		3685		3685	
Wt. of mould + WS (g)	5520		5594		5763		5792		5713		5663	
Wt. of WS (g)	1835		1909		2078		2107		2028		1978	
Can no	A14	B4	1X	Po1	S16	FO	AG2	K1	A1	Ao	FOS	10B
Wt. of can (g)	24.6	24.4	24.4	24.6	24.5	24.4	25.3	24.7	24.5	20.3	24.8	20.1
Can +Ws (g)	47.9	42.4	46.2	37.6	47.4	50.9	52.1	52.3	54.5	49.1	57.2	39.6
Can + Ds (g)	46.6	41.2	44.3	36.4	44.6	48.1	48.2	48.3	49.1	43.8	50.5	35.6
Wt. of water (g)	1.3	1.2	1.9	1.2	2.8	2.8	3.9	4	5.4	5.3	6.7	4
Wt. of Ds (g)	22	16.8	19.9	11.8	20.1	23.7	22.9	23.6	24.6	23.5	25.7	15.5
% MC	5.91	7.14	9.55	10.17	13.93	11.81	17.03	16.88	21.95	22.55	26.07	25.81
Average % MC (%)	6.53		9.86		13.87		18.95		22.25		25.94	
Dry Density (g/cm^3)	1.73		1.74		1.87		1.81		1.66		1.57	

Table C4: Result of Compaction Data for Soil treated with 6% Lime

Trials	1		2		3		4		5		6
Wt. of mould (g)	3685		3685		3685		3685		3685		-
Wt. of mould +	5553		5685		5892		5716		5632		-
Wt. of WS (g)	1868		2000		2207		2031		1947		-
Can no	S10	SI	D2	S4	C9	C4	S6	5A	B1	3H	-
Wt. of can (g)	22.8	23.1	25.9	22.7	20.5	25.2	24.4	23.2	21.3	21.5	-
Can +Ws (g)	56.2	47.8	63.6	47.8	59.3	64.6	75.1	83.8	68.0	71	-
Can + Ds (g)	54.2	46.3	59.6	45.1	54.1	59.1	66.6	74.1	58.8	61.1	-
Wt. of water (g)	2	1.5	4	2.7	5.2	5.5	8.5	9.7	9.2	9.9	-
Wt. of Ds (g)	31.4	23.2	33.7	22.4	33.6	33.9	42.2	50.9	37.5	39.6	-
% MC	6.37	6.47	11.87	12.05	15.48	16.22	20.14	19.06	24.53	25	-
Average % MC (%)	6.42		11.96		15.85		19.8		24.77		-
Dry Density (g/cm^3)	1.76		1.79		1.91		1.70		1.57		-

Table C5: Result of Compaction Data for Soil treated with 8% Lime

Trials	1		2		3		4		5		6
Wt. of mould (g)	3685		3685		3685		3685		3685		-
Wt. of mould + WS (g)	5562		5690		5804		5679		5656		-
Wt. of WS (g)	1877		2005		2119		1994		1971		-
Can no	T3	F10	RM18	RM36	RM17	RM46	RM42	TF	RM20	Z18	-
Wt. of can (g)	24.5	24.7	24.5	22.8	24.3	22.6	23	23.3	24.2	27.5	-
Can +Ws (g)	56.1	51.0	54.4	46.7	55.9	53.4	70.8	68.8	66.9	68.8	-
Can + Ds (g)	53.9	49.2	51.2	44.1	51.4	48.9	62.1	60.6	58.6	60.9	-
Wt. of water (g)	2.2	1.8	3.2	2.6	4.5	4.5	8.7	8.2	8.3	7.9	-
Wt. of Ds (g)	29.4	24.5	26.7	21.3	27.1	26.3	39.1	37.3	34.4	33.4	-
% MC	7.84	7.35	11.98	12.21	16.61	17.11	22.25	21.98	24.13	23.65	-
Average % MC (%)	7.42		12.09		16.86		22.12		23.89		-
Dry Density (g/cm^3)	1.75		1.79		1.82		1.64		1.59		-

Table C6: Variation of MDD and OMC of Soil with Different Combinations of Lime

Material	Maximum Dry Density MDD (g/cm^3)	O.M.C (%)
Soil + 0% lime	1.915	15.3
Soil + 2% lime	1.87	14
Soil+4% lime	1.875	14.5
Soil + 6% lime	1.9	15.5
Soil + 8% lime	1.825	16

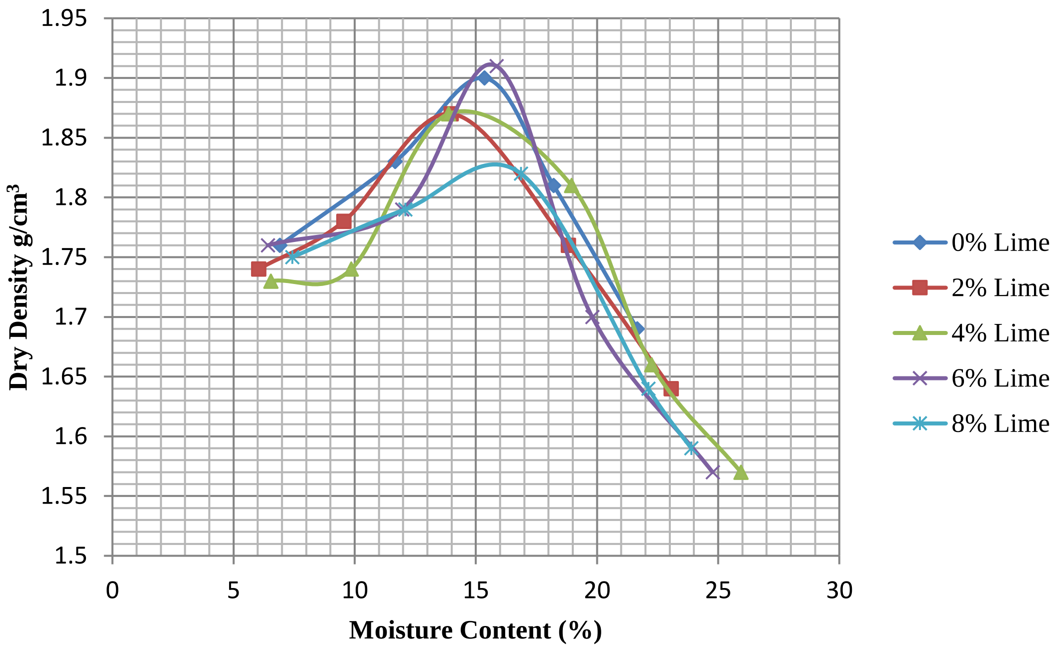

Figure C1: Compaction curve values of soil treated with different % lime

Result of compaction data of soil with 6% Lime + % Plastic waste

Table C7: Result of Compaction Data for Soil treated with 6% Lime + 1% Plastic Waste

Trials	1		2		3		4		5	6
Wt. of mould (g)	3870		3870		3870		3870		-	-
Wt. of mould	5696		5928		5890		5817		-	-
Wt. of WS (g)	1826		2058		2020		1947		-	-
Can no	RM 36	RM 37	RM17	RM3	RM42	RM8	RM25	ZZ	-	-
Wt. of can (g)	22.8	24.1	24.2	24.7	22.9	23.3	24.1	23.3	-	-
Can +Ws (g)	45.1	54.2	56.7	50.8	47.1	50.4	63.2	54	-	-
Can + Ds (g)	43.2	51.7	52.5	47.2	42.9	45.6	55.4	48	-	-
Wt. of water (g)	1.9	2.5	4.2	3.6	4.2	4.8	7.8	6	-	-
Wt. of Ds (g)	20.4	27.6	28.3	22.5	20	22.3	31.3	24.7	-	-
% MC	9.31	9.06	14.84	16	21	21.52	24.92	24.29	-	-
Average % MC (%)	9.18		15.42		21.26		24.61		-	-
Dry Density (g/cm^3)	1.67		1.78		1.66		1.56		-	-

Table C8: Result of Compaction Data for Soil treated with 6% Lime + 2% Plastic Waste

Trials	1		2		3		4		5		6	
Wt. of mould (g)	3870		3870		3870		3870		3870		-	
Wt. of mould + WS (g)	5637		5812		5916		5870		5818		-	
Wt. of WS (g)	1767		1942		2046		2000		1948		-	
Can no	AC	RM18	RM46	Z1	P25	X22	Q7	G42	Bp	M7	-	-
Wt. of can (g)	25.1	24.4	22.6	25.2	38.7	38.4	38	37.9	38	38.4	-	-
Can +Ws (g)	48.1	54.8	46.4	50.4	64.8	60.4	76.5	78.3	73.6	76.5	-	-
Can + Ds (g)	46.2	52.7	43.6	47.4	61.0	57.2	69.8	71.2	66.2	68.8	-	-
Wt. of water (g)	1.9	2.1	2.8	3	3.8	3.2	6.7	7.1	7.4	7.7	-	-
Wt. of Ds (g)	21.1	28.3	21	22.2	22.3	18.8	31.8	33.3	28.2	30.4	-	-
% MC	9.0	7.42	13.33	13.51	17.04	17.02	21.07	21.32	26.24	25.33	-	-
Average % MC (%)	8.21		13.42		17.03		21.19		25.78		-	
Dry Density (g/cm^3)	1.63		1.71		1.75		1.65		1.55		-	

Table C9: Result of Compaction Data for Soil treated with 6% Lime + 3% Plastic Waste

Trials	1		2		3		4		5		6
Wt. of mould (g)	3870		3870		3870		3870		3870		-
Wt. of mould + WS (g)	5595		5795		5889		5807		5734		-
Wt. of WS (g)	1725		1925		2019		1937		1864		-
Can no	T2A	Q51	Ao	P4	Q3	H2	M3	M19	AA5	AA6	-
Wt. of can (g)	38.1	35.4	24.6	37.9	35.4	38.4	38.5	38.6	38.8	38.3	-
Can +Ws (g)	68.8	63.4	60.7	65.8	72.6	70.9	71.1	78.3	84.9	87.4	-
Can + Ds (g)	66.5	61.2	56.6	62.5	66.7	65.9	64.6	70.5	74.7	76.1	-
Wt. of water (g)	2.3	2.2	4.1	3.3	5.9	5	6.5	7.8	10.23	11.3	-
Wt. of Ds (g)	28.4	25.8	32	24.6	31.3	27.5	26.1	31.9	35.9	37.8	-
% MC	8.1	8.53	12.81	13.41	18.85	18.18	24.90	24.45	28.41	29.89	-
Average % MC (%)	8.21		13.11		18.51		24.68		29.15		-
DryDensity (g/cm^3)	1.59		1.702		1.704		1.55		1.44		-

Table C10: Result of Compaction Data for Soil treated with 6% Lime + 4% Plastic Waste

Trials	1		2		3		4		5		6	
Wt. of mould (g)	3870		3870		3870		3870		3870		3870	
Wt. of mould + WS (g)	5540		5635		5810		5920		5867		5775	
Wt. of WS (g)	1670		1765		1940		2050		1997		1905	
Can no	RM25	RM3	RM36	RM18	Q3	M3	T2A	AA5	X22	P4	Z1	RM8
Wt. of can (g)	24.2	24.8	22.8	24.4	35.4	38.5	38.3	38.9	38.4	38	24.9	23.2
Can +Ws (g)	51.5	55.5	58.4	49.8	69.5	70.1	73.6	77.2	90.5	81	61.1	57
Can + Ds (g)	49.7	53.5	55.1	47.4	64	65.8	67.9	70.8	81.2	72	53	49.4
Wt. of water (g)	1.8	2	3.3	2.4	5.5	4.3	5.7	6.4	9.3	8.3	8.1	7.6
Wt. of Ds (g)	25.5	28.7	32.3	23	28.6	27.3	29.6	31.9	42.8	34.7	28.1	26.2
% MC	7.06	6.97	10.22	10.43	19.23	15.75	19.26	20.06	21.72	23.92	28.82	29.01
Average % MC (%)	7.01		10.32		17.49		19.66		22.82		28.92	
Dry Density (g/cm^3)	1.56		1.6		1.65		1.71		1.63		1.48	

Table C11: Variation of MDD and OMC of Soil with 6% Lime + % Plastics

Soil + % Additive	MDD (g/cm^3)	OMC (%)
Soil + 0% Additive	1.915	15.3
Soil + 6% Lime + 1% Plastic	1.782	15.5
Soil + 6% Lime + 2% Plastic	1.751	16
Soil + 6% Lime + 3% Plastic	1.717	16.5
Soil + 6% Lime + 4% Plastic	1.7132	19.5

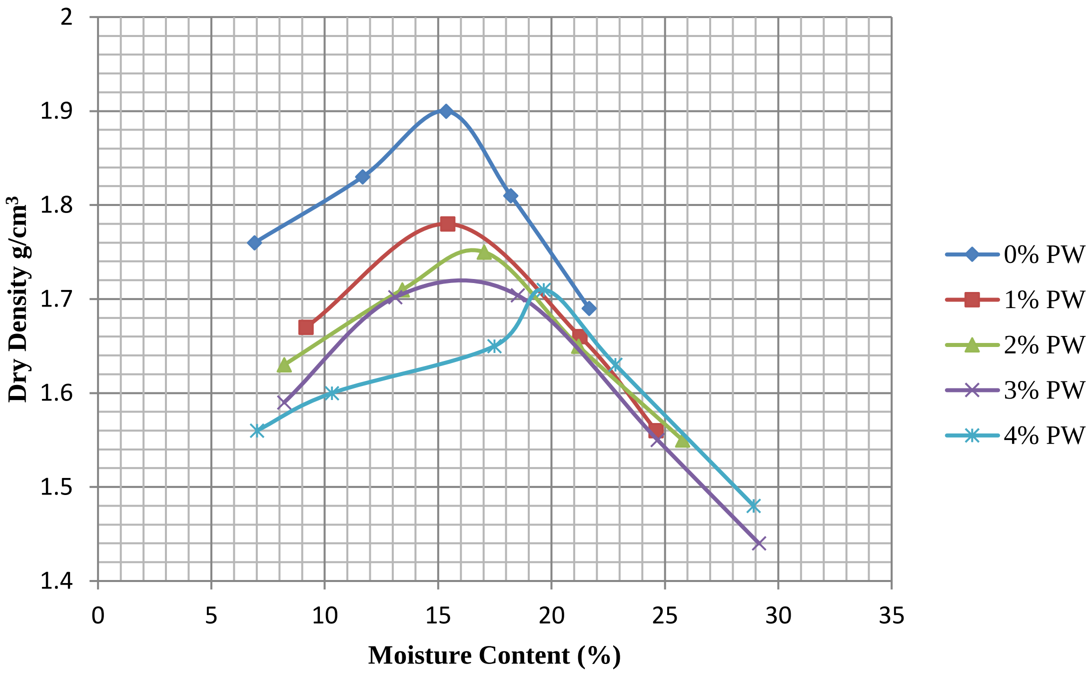

Figure C2: Compaction curve values for soil treated with 6% lime + % Plastic Waste (PW)

APPENDIX D: Result of California Bearing Ratio (CBR) test data

Table D1: Result of CBR test obtained from the laboratory for the Natural Soil Sample

Penetration (mm)	Top dial load reading	Bottom dial load reading	Average load gauge reading	Load (kN)
0	0	0	0	0
0.50	4	3	3.5	0.15
1.00	6	6	6	0.25
1.50	8	14	11	0.46
2.00	11	15	13	0.54
2.50	12	22	17	0.71
3.00	15	23	19	0.79
3.50	19	23	21	0.87
4.00	22	25	23.5	0.98
4.50	25	27	26	1.08
5.00	28	30	29	1.21
5.50	32	32	32	1.33
6.00	36	35	35.5	1.48
6.50	40	38	39	1.62
7.00	43	40	41.5	1.73
7.50	47	43	45	1.87
8.00	51	45	48	1.99
8.50	55	46	50.5	2.10
9.00	58	48	53	2.20
9.50	59	51	55	2.29
10.00	60	53	56.5	2.35
10.50	61	55	58	2.41

Table D2a: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Lime from 0 – 200 penetrations

	0% Lime				2% Lime				4% Lime				6% Lime				8% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	3	1	2	0.0832	12.0	4.91	8.47	0.352	5.4	4.3	4.86	0.202	1.90	2.38	2.142	0.089	6.2	6.74	6.48	0.26
50	4	3	3.5	0.1456	26.2	18.0	22.13	0.920	14.5	25	19.98	0.831	4.76	6.66	5.71	0.237	13	20.7	16.86	0.701
75	5	4	4.5	0.1872	45.3	37.1	41.26	1.716	27	54.5	40.77	1.696	9.5	11.9	10.71	0.445	23.3	31.1	27.24	1.133
100	6	6	6	0.2496	57.3	55.7	56.57	2.353	44.2	74.5	59.4	2.471	16.2	27.6	21.89	0.910	38.4	41.5	39.9	1.662
125	7	10	8.5	0.3536	71.0	69.9	70.51	2.933	61.5	87.4	74.52	3.100	23.3	41.4	32.36	1.346	52.9	51.9	52.4	2.180
150	8	14	11	0.4576	82.5	83.6	83.08	3.456	78.3	96	87.21	3.628	29.0	53.3	41.17	1.712	66.4	59.6	63.05	2.623
175	9	14	11.5	0.4784	96.2	96.2	96.20	4.001	93.9	105	99.36	4.133	34.3	61.8	48.07	2.000	77.8	68.5	73.17	3.044
200	10	16	13	0.5408	107.68	106.0	106.86	4.445	105	112	108.54	4.515	40.5	69.5	54.97	2.287	87.7	76.3	82.00	3.411

Table D2b: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Lime from 225 – 325 penetrations

	0% Lime				2% Lime				4% Lime				6% Lime				8% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
225	11	18	14.5	0.6032	116.42	115.87	116.15	4.831	114	118	116.37	4.841	46.6	75.2	60.92	2.534	98	83.0	90.56	3.767
250	12	22	17	0.7072	125.71	125.71	125.71	5.229	123	124	123.66	5.144	51.4	80.9	66.16	2.752	106	89.8	98.09	4.080
275	14	22	18	0.7488	133.91	135.55	134.73	5.605	132	131	131.49	5.47	57.1	86.6	71.87	2.990	115	96.5	105.87	4.404
300	15	23	19	0.7904	142.11	144.30	143.20	5.957	138	137	137.97	5.74	62.8	91.9	77.35	3.217	123	103	113.14	4.706
325	16	24	20	0.832	147.58	151.40	149.49	6.218	145	143	143.64	5.975	68.5	97.1	82.82	3.445	132	108.4	120.14	4.998

Table D2c: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Lime from 350 – 600 penetrations

	0% Lime				2% Lime				4% Lime				6% Lime				8% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
350	17	25	21	0.8736	155.23	159.06	157.14	6.537	151	166	158.49	6.593	73.8	101.3	87.58	3.643	140	113.6	126.63	5.268
375	19	26	22.5	0.936	162.34	164.52	163.43	6.798	157	171	163.89	6.818	79.5	106.1	92.82	3.861	145	120	132.60	5.516
400	20	27	23.5	0.9776	168.35	171.63	169.99	7.071	162	175	168.48	7.009	84.3	110	97.34	4.049	150.5	124.5	137.53	5.721
425	21	28	24.5	1.0192	173.81	177.64	175.73	7.310	167	179	173.34	7.211	89.0	114.2	101.62	4.227	157	129.2	142.98	5.94
450	22	30	26	1.0816	178.73	184.75	181.74	7.560	172	184	177.66	7.391	93.3	118.5	105.91	4.406	162.4	133.3	147.91	6.153
475	23	32	27.5	1.144	184.75	202.24	193.49	8.04	176	187	181.98	7.570	97.6	122.3	109.95	4.574	168	137	152.32	6.336
500	25	33	29	1.2064	189.67	209.34	199.50	8.299	182	191	186.57	7.761	101.9	125.67	113.76	4.732	172	141	156.73	6.520
525	27	34	30.5	1.2688	194.04	214.81	204.42	8.504	186	194	190.35	7.918	106.1	129.5	117.81	4.901	176	144	160.63	6.682
550	29	35	32	1.3312	198.96	219.73	209.34	8.708	190	195	192.78	8.02	110.4	133.3	121.85	5.069	181	148.4	164.78	6.854
575	31	37	34	1.4144	203.88	224.10	213.99	8.902	194	202	198.18	8.244	114.2	136.6	125.42	5.217	185	152	168.41	7.006
600	33	38	35.5	1.4768	208.80	227.93	218.36	9.084	198	206	201.96	8.401	118.05	140	128.99	5.366	189	155.7	172.30	7.168

Table D2d: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Lime from 625 – 875 penetrations

	0% Lime				2% Lime				4% Lime				6% Lime				8% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
625	35	40	37.5	1.56	213.17	231.21	222.19	9.243	201	208	204.93	8.525	122	143	132.56	5.514	192.5	158.8	175.68	7.30
650	37	41	39	1.6224	217.54	238.31	227.93	9.481	205	212	208.44	8.671	125.2	146	135.66	5.645	195.6	162	178.79	7.437
675	38	42	40	1.664	221.91	242.14	232.03	9.652	208	215	211.68	8.806	128	149	138.51	5.762	199	164	181.65	7.55
700	40	43	41.5	1.7264	225.74	245.42	235.58	9.800	212	219	215.46	8.963	131	152	141.37	5.881	202	166.59	184.50	7.675
725	42	44	43	1.7888	227.38	247.60	237.49	9.879	214	221	217.89	9.064	134	154	144.23	6.000	205.5	169.71	187.61	7.80
750	45	45	45	1.872	231.75	250.88	241.32	10.03	217	225	220.86	9.188	136.6	157	146.84	6.108	208.6	171.78	190.21	7.912
775	47	46	46.5	1.9344	235.03	256.90	245.97	10.23	220	227	223.56	9.300	140	159	149.70	6.227	211	173.86	192.54	8.010
800	49	47	48	1.9968	238.31	264.55	251.43	10.45	224	229.5	226.53	9.424	143	162	152.56	6.346	214	175.94	194.88	8.107
825	52	47.5	49.75	2.0696	241.59	271.66	256.62	10.67	226	233	229.5	9.547	145.2	165	154.94	6.445	216.4	178.53	197.47	8.215
850	53	48	50.5	2.1008	245.42	274.39	259.90	10.81	229	243	235.98	9.817	148	167.1	157.55	6.554	219	180.61	199.81	8.312
875	55	49.5	52.25	2.1736	248.15	276.03	262.09	10.90	232	247	239.49	9.963	150	169.45	159.93	6.653	221	182.68	201.89	8.398

Table D2e: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Lime from 900 – 1100 penetrations

	0% Lime				2% Lime				4% Lime				6% Lime				8% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
900	56	50	53	2.2048	251.43	277.12	264.28	10.99	235	248	241.65	10.05	153	171.36	162.08	6.742	224	184.76	204.48	8.506
925	57	51	54	2.2464	255.80	278.76	267.28	11.11	238	251	244.35	10.16	155	173.74	164.46	6.841	226	186.32	206.3025	8.582
950	58	52	55	2.288	259.08	279.85	269.47	11.21	240	253	246.51	10.25	157	175.64	166.36	6.920	229	188.4	208.63	8.679
975	58	53	55.5	2.3088	264.00	283.13	273.57	11.38	242	256	249.21	10.37	159.4	178.02	168.74	7.019	230.4	190	210.19	8.744
1000	59	54	56.5	2.3504	0	285.87	285.87	11.89	245	258	251.64	10.47	161.3	179.93	170.64	7.099	232.5	192.03	212.27	8.830
1025	59.5	55	57.25	2.3816	0	288.60	288.60	12.00	247	260	253.8	10.55	163.7	181.83	172.79	7.188	234.5	194	214.34	8.916
1050	60	56	58	2.4128	0	291.88	291.88	12.14	249	262	255.96	10.65	165.6	184.21	174.93	7.277	236	197	216.68	9.013
1075	60	56	58	2.4128	0	294.07	294.07	12.23	251	264	258.12	10.74	167.5	186.11	176.83	7.356	238.2	199	218.75	9.100
1100	61	57	59	2.4544	0	296.80	296.80	12.34	254	267	260.28	10.83	169.45	188.02	178.74	7.435	240	201	220.31	9.165

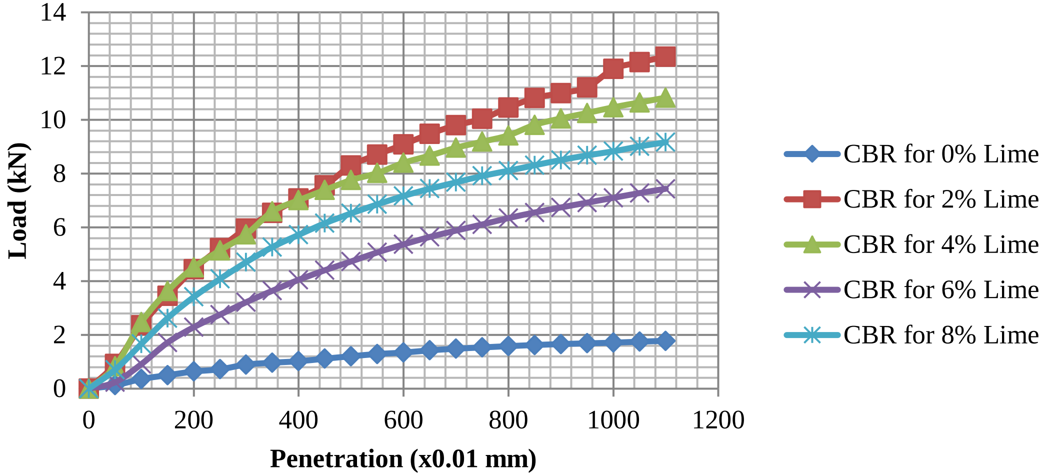

Figure D1: Load vs. Penetration curve for soil treated with lime

Table D3: Variation of CBR values of Soil with Different Percentages of Lime

Soil + % Additive	CBR (%)
Soil + 0% Lime	6.04
Soil + 2% Lime	41.58
Soil + 4% Lime	38.88
Soil + 6% Lime	23.71
Soil + 8% Lime	32.67

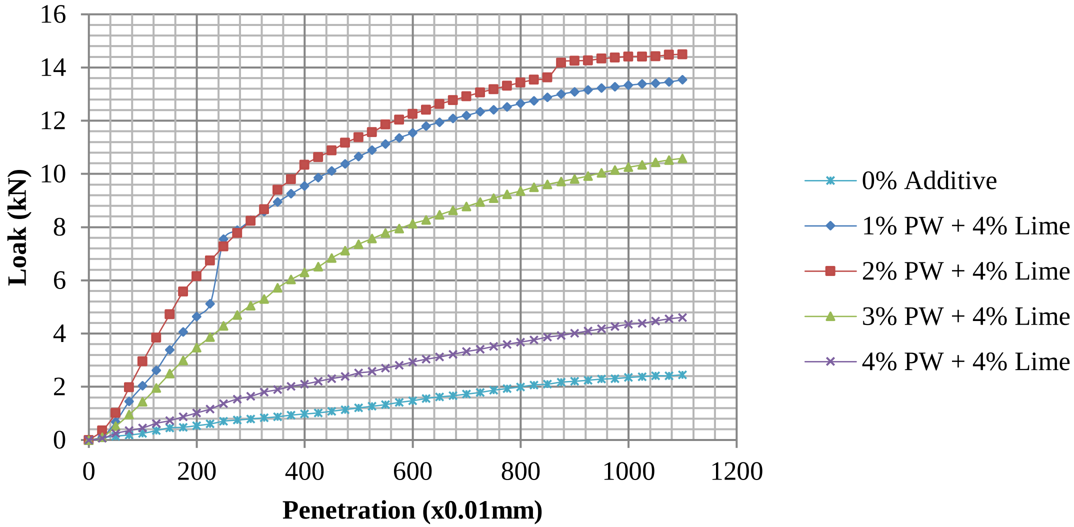

Figure D2: Load vs. Penetration curve for soil treated with lime + plastic

Table D4: Variation of CBR values of Soil with Different Percentages of Plastic +Lime

Soil + % Additive	CBR (%)
Soil + 0% Additive	6.04
Soil + 4% Lime + 1% Plastic	57.03
Soil + 4% Lime + 2% Plastic	57.01
Soil + 4% Lime + 3% Plastic	36.89
Soil + 4% Lime + 4% Plastic	12.61

Table D5a: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Plastic Waste –Lime Blend from 0 – 325 penetrations

	0% PW + 0% Lime				1% PW + 4% Lime				2% PW + 4% Lime				3% PW + 4% Lime				4% PW + 4% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25	3	1	2	0.0832	3	2	2.5	0.104	4	13	8.5	0.3536	3	2	2.5	0.104	1	2	1.5	0.0624
50	4	3	3.5	0.1456	8	25	16.5	0.6864	7	42	24.5	1.0192	6	19	12.5	0.52	3	9	6	0.2496
75	5	4	4.5	0.1872	16	54	35	1.456	22	73	47.5	1.976	11	35	23	0.9568	5	12	8.5	0.3536
100	6	6	6	0.2496	25	73	49	2.0384	44	98	71	2.9536	15	54	34.5	1.4352	7	15	11	0.4576
125	7	10	8.5	0.3536	36	90	63	2.6208	65	120	92.5	3.848	24	70	47	1.9552	11	19	15	0.624
150	8	14	11	0.4576	52	111	81.5	3.3904	86	141	113.5	4.7216	35	85	60	2.496	13	22	17.5	0.728
175	9	14	11.5	0.4784	66	129	97.5	4.056	111	157	134	5.5744	46	98	72	2.9952	16	26	21	0.8736
200	10	16	13	0.5408	78	145	111.5	4.6384	131	165	148	6.1568	57	110	83.5	3.4736	19	30	24.5	1.0192
225	11	18	14.5	0.6032	89	157	123	5.1168	149	175	162	6.7392	66	120	93	3.8688	22	34	28	1.1648
250	12	22	17	0.7072	199	164	181.5	7.5504	165	185	175	7.28	75	131	103	4.2848	27	39	33	1.3728
275	14	22	18	0.7488	210	169	189.5	7.8832	179	195	187	7.7792	86	140	113	4.7008	31	43	37	1.5392
300	15	23	19	0.7904	220	177	198.5	8.2576	191	205	198	8.2368	94	149	121.5	5.0544	36	43	39.5	1.6432
325	16	24	20	0.832	230	183	206.5	8.5904	202	215	208.5	8.6736	102	153	127.5	5.304	43	43.5	43.25	1.7992

Table D5b: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Plastic Waste – Lime Blend from 350 – 750 penetrations

	0% PW + 0% Lime				1% PW + 4% Lime				2% PW + 4% Lime				3% PW + 4% Lime				4% PW + 4% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
350	17	25	21	0.8736	240	190	215	8.944	212	240	226	9.4016	111	164	137.5	5.72	47	44	45.5	1.8928
375	19	26	22.5	0.936	247	198	222.5	9.256	221	250	235.5	9.7968	120	170	145	6.032	52	45	48.5	2.0176
400	20	27	23.5	0.9776	255	204	229.5	9.5472	229	268	248.5	10.3376	128	175	151.5	6.3024	55	46	50.5	2.1008
425	21	28	24.5	1.0192	263	211	237	9.8592	238	273	255.5	10.6288	131	182	156.5	6.5104	59	47	53	2.2048
450	22	30	26	1.0816	270	216	243	10.1088	245	278	261.5	10.8784	144	185	164.5	6.8432	62	49	55.5	2.3088
475	23	32	27.5	1.144	278	221	249.5	10.3792	253	284	268.5	11.1696	150	192	171	7.1136	65	50	57.5	2.392
500	25	33	29	1.2064	286	226	256	10.6496	259	288	273.5	11.3776	156	198	177	7.3632	69	52	60.5	2.5168
525	27	34	30.5	1.2688	293	231	262	10.8992	263	293	278	11.5648	161	203	182	7.5712	71	53	62	2.5792
550	29	35	32	1.3312	299	236	267.5	11.128	272	298	285	11.856	166	208	187	7.7792	74	56	65	2.704
575	31	37	34	1.4144	306	240	273	11.3568	278	301	289.5	12.0432	170	212	191	7.9456	77	58	67.5	2.808
600	33	38	35.5	1.4768	311	244	277.5	11.544	283	306	294.5	12.2512	175	216	195.5	8.1328	81	60	70.5	2.9328
625	35	40	37.5	1.56	318	249	283.5	11.7936	288	309	298.5	12.4176	179	219	199	8.2784	85	61	73	3.0368
650	37	41	39	1.6224	322	252	287	11.9392	294	313	303.5	12.6256	184	223	203.5	8.4656	87	63	75	3.12
675	38	42	40	1.664	327	254	290.5	12.0848	298	316	307	12.7712	188	227	207.5	8.632	89	66	77.5	3.224
700	40	43	41.5	1.7264	332	254.5	293.25	12.1992	302	319	310.5	12.9168	192	230	211	8.7776	92	68	80	3.328
725	42	44	43	1.7888	337	256	296.5	12.3344	306	322	314	13.0624	196	234	215	8.944	95	69	82	3.4112
750	45	45	45	1.872	340	257	298.5	12.4176	310	324	317	13.1872	199	238	218.5	9.0896	98	71	84.5	3.5152

Table D5c: Result of data obtained from the CBR test conducted in the laboratory for different percentages of Plastic Waste – Lime Blend from 775 – 1100 penetrations

	0% PW + 0% Lime				1% PW + 4% Lime				2% PW + 4% Lime				3% PW + 4% Lime				4% PW + 4% Lime			
Penetration (x0.01mm)	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN	T	B	AVG	Load kN
775	47	46	46.5	1.9344	344	258	301	12.5216	314	326	320	13.312	203	241	222	9.2352	100	73	86.5	3.5984
800	49	47	48	1.9968	348	260	304	12.6464	318	328	323	13.4368	206	244	225	9.36	103	74	88.5	3.6816
825	52	47.5	49.75	2.0696	350	263	306.5	12.7504	321	330	325.5	13.5408	210	247	228.5	9.5056	105	76	90.5	3.7648
850	53	48	50.5	2.1008	354	265	309.5	12.8752	324	331	327.5	13.624	212	250	231	9.6096	108	78	93	3.8688
875	55	49.5	52.25	2.1736	357	268	312.5	13	350	332	341	14.1856	215	252	233.5	9.7136	110	79	94.5	3.9312
900	56	50	53	2.2048	359	270	314.5	13.0832	353	332	342.5	14.248	217	255	236	9.8176	112	81	96.5	4.0144
925	57	51	54	2.2464	361.5	271	316.25	13.156	354	332	343	14.2688	220	257	238.5	9.9216	114	83	98.5	4.0976
950	58	52	55	2.288	363	273	318	13.2288	357	332	344.5	14.3312	223	260	241.5	10.0464	116	85	100.5	4.1808
975	58	53	55.5	2.3088	364.5	274	319.25	13.2808	359	332	345.5	14.3728	226	262	244	10.1504	118	87	102.5	4.264
1000	59	54	56.5	2.3504	367	274	320.5	13.3328	361	332	346.5	14.4144	229	264	246.5	10.2544	120	89	104.5	4.3472
1025	59.5	55	57.25	2.3816	369	274.5	321.75	13.3848	362	331	346.5	14.4144	231	266	248.5	10.3376	121	90	105.5	4.3888
1050	60	56	58	2.4128	369.5	275	322.25	13.4056	363.5	330	346.75	14.4248	234	268	251	10.4416	123	92	107.5	4.472
1075	60	56	58	2.4128	370	277	323.5	13.4576	367	329	348	14.4768	236	270	253	10.5248	125	94	109.5	4.5552
1100	61	57	59	2.4544	372	279	325.5	13.5408	369	328	348.5	14.4976	238	271	254.5	10.5872	126.5	95	110.75	4.6072

APPENDIX E: Result of Unconfined Compressive Strength (UCS) test data

Table E1: Result of UCS test obtained from the Laboratory for the Natural Soil

Deformation dial reading (mm)	Strain e (%)	Load Dial reading	Load (kN)	Stress (kN/m^2)
0	0	0	0	0
0.25	0.33	9	0.064	56.48
0.5	0.66	17	0.121	106.33
0.75	0.99	24	0.171	149.62
1	1.31	32	0.228	198.83
1.25	1.64	40	0.285	247.71
1.5	1.97	49	0.350	302.43
1.75	2.3	55	0.392	338.32
2.0	2.63	59	0.421	361.71
2.25	2.96	61	0.435	372.70
2.5	3.29	63	0.450	383.62
2.75	3.62	64	0.457	388.38
3.0	3.95	64	0.457	387.06
3.25	4.28	64	0.457	385.73
3.5	4.61	62	0.443	372.39
3.75	4.93	60	0.428	359.14
4.0	5.26	56	0. 400	334.04

The result of UCS data obtained for soil treated with different percentages of Lime.

Table E2a: Soil Sample treated with different percentages of Lime from 0 – 150 penetrations

				0% Lime			2% Lime			4% Lime			6% Lime			8% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
0	0	0	0.001134	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.328947	25	0.0032895	0.0011377743	9	0.06426	56.48026	19	0.13566	119.2349	5	0.0357	31.37759	9	0.06426	56.47967	7	0.04998	43.92863
0.657895	50	0.0065789	0.00114151	17	0.12138	106.3328	45	0.3213	281.4633	26	0.18564	162.6233	31	0.22134	193.897	17	0.12138	106.3306
0.986842	75	0.0098684	0.0011455302	24	0.17136	149.6199	66	0.47124	411.4416	37	0.26418	230.6566	54	0.38556	336.634	31	0.22134	193.2529
1.315789	100	0.0131579	0.00114912	32	0.22848	198.8304	88	0.62832	546.8157	49	0.34986	304.4769	70	0.4998	434.967	48	0.34272	298.2631
1.644737	125	0.0164474	0.0011552963	40	0.2856	247.7096	95	0.6783	588.3385	57	0.40698	353.0031	81	0.57834	501.636	61	0.43554	377.7753
1.973684	150	0.0197368	0.0011566832	49	0.34986	302.4293	80	0.5712	493.7606	78	0.55692	481.4166	92	0.65688	567.8247	76	0.54264	469.0726

Table E2b: Soil Sample treated with different percentages of Lime from 175 – 325 penetrations

				0% Lime			2% Lime			4% Lime			6% Lime			8% Lime		
Strain (%)	Penetration (x0.01 mm)	Strain e	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
2.302632	175	0.0230263	0.001160727	55	0.3927	338.3224				86	0.61404	529.0274	95	0.6783	584.3907	88	0.62832	541.3304
2.631579	200	0.0263158	0.001164649	59	0.42126	361.7057				90	0.6426	551.7633	97	0.69258	594.6783	96	0.68544	588.5476
2.960526	225	0.0296053	0.001168597	61	0.43554	372.7035				90	0.6426	549.8933	91	0.64974	556.0033	100	0.714	610.9926
3.289474	250	0.0328947	0.001172571	63	0.44982	383.6184				86	0.61404	523.6667				60	0.4284	366.5956
3.618421	275	0.0361842	0.001176573	64	0.45696	388.3821												
3.947368	300	0.0394737	0.001180603	64	0.45696	387.0565												
4.276316	325	0.0427632	0.00118466	64	0.45696	385.731												

Table E2c: Soil Sample treated with different percentages of Lime from 350 – 400 penetrations

				0% Lime			2% Lime			4% Lime			6% Lime			8% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
4.605263	350	0.0460526	0.001188745	62	0.44268	372.3928												
4.934211	375	0.0493421	0.001192858	60	0.4284	359.1374												
5.263158	400	0.0526316	0.0011977	56	0.39984	334.0351												

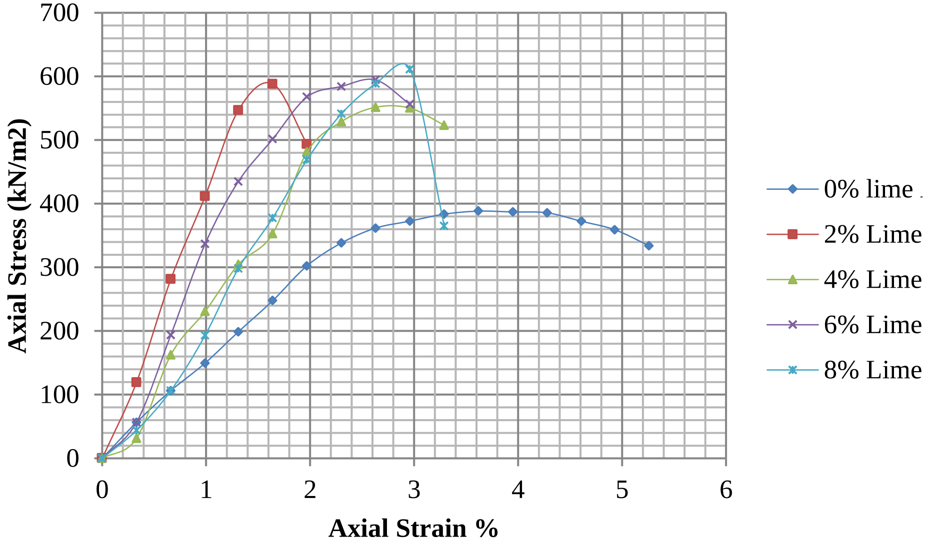

Figure E1: Axial stress vs. Axial stress for lime treated soil.

Table E3: Variation of UCS values of Soil with Different Percentages of Lime

Soil + % Lime	UCS (kN/m^2)
Soil + 0% lime	388.38
Soil + 2% lime	588.29
Soil + 4% lime	551.59
Soil + 6% lime	588.33
Soil + 8% lime	611.3

The result of UCS data obtained for different percentages of plastic waste grains + Lime blend at different curing days.

Table E4a: 0 Curing Days Period from 0 – 175 penetrations

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
0	0	0	0.001134	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.328947	25	0.0032895	0.0011377743	9	0.06426	56.48026	12	0.08568	75.30702	33	0.23562	207.0943	27	0.19278	169.4408	9	0.06426	56.48026
0.657895	50	0.0065789	0.00114151	17	0.12138	106.3328	43	0.30702	268.9596	63	0.44982	394.057	52	0.37128	325.2534	14	0.09996	87.56823
0.986842	75	0.0098684	0.001145302	24	0.17136	149.6199	74	0.52836	461.328	83	0.59262	517.4354	72	0.51408	448.8596	20	0.1428	124.6832
1.315789	100	0.0131579	0.00114912	32	0.22848	198.8304	78	0.55692	484.6491	99	0.70686	615.1316	84	0.59976	521.9298	27	0.19278	167.7632
1.644737	125	0.0164474	0.001152963	40	0.2856	247.7096	88	0.62832	544.961	103	0.73542	637.8521	88	0.62832	544.961	35	0.2499	216.7459
1.973684	150	0.0197368	0.001156832	49	0.34986	302.4293	96	0.68544	592.5146	95	0.6783	586.3426	94	0.67116	580.1706	43	0.30702	265.3972
2.302632	175	0.0230263	0.001160727	55	0.3927	338.3224	104	0.74256	639.7368				104	0.74256	639.7368	51	0.36414	313.7171

Table E4b: 0 Curing Days Period from 200 – 400 penetrations

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Strain e	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
2.631579	200	0.0263158	0.001164649	59	0.42126	361.7057	108	0.77112	662.1053				123	0.87822	754.0643	57	0.40698	349.4444
2.960526	225	0.0296053	0.001168597	61	0.43554	372.7035	115	0.8211	702.6377				119	0.84966	727.0772	62	0.44268	378.8134
3.289474	250	0.0328947	0.001172571	63	0.44982	383.6184	122	0.87108	742.8801							66	0.47124	401.886
3.618421	275	0.0361842	0.001176573	64	0.45696	388.3821	128	0.91392	776.7641							68	0.48552	412.6559
3.947368	300	0.0394737	0.001180603	64	0.45696	387.0565	131	0.93534	792.2563							68	0.48552	411.2476
4.276316	325	0.0427632	0.00118466	64	0.45696	385.731	110	0.7854	662.9751							65	0.4641	391.758
4.605263	350	0.0460526	0.001188745	62	0.44268	372.3928										59	0.42126	354.3738
4.934211	375	0.0493421	0.001192858	60	0.4284	359.1374												
5.263158	400	0.0526316	0.0011967	56	0.39984	334.0351												

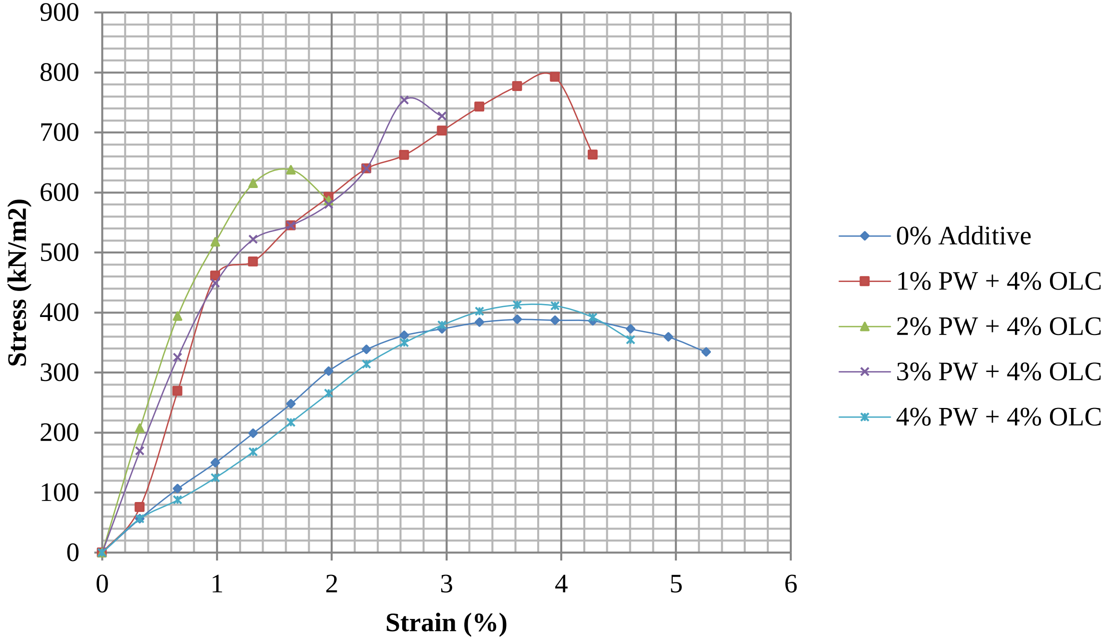

Table E5: Variation of UCS values with % Plast Waste (PW)

% Plastic	UCS (kN/m^2)
0	388.38
1	792.24
2	637.88
3	754.08
4	412.65

Figure E2: Axial stress vs. Axial stress for soil treated with lime + plastic after 0 curing days

Table E6a: 7 Curing Days Period from 0 – 150 penetrations

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
0	0	0	0.001134	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.328947	25	0.0032895	0.001137743	9	0.06426	56.47967	7	0.385	338.3858	15	0.825	725.1124	4	0.22	193.3633	19	1.045	918.4757
0.657895	50	0.0065789	0.00114151	12	0.08568	75.05689	29	1.595	1397.243	21	1.155	1011.796	12	0.66	578.1693	21	1.155	1011.796
0.986842	75	0.0098684	0.001145302	19	0.13566	118.4453	25	1.375	1200.518	22	1.21	1056.456	21	1.155	1008.435	19	1.045	912.3937
1.315789	100	0.0131579	0.00114912	26	0.18564	161.5592				18	0.99	861.5794	21	1.155	1005.176	15	0.825	717.9828
1.644737	125	0.0164474	0.001152963	33	0.23562	204.3702				13	0.715	620.1711	17	0.935	810.9929			
1.973684	150	0.0197368	0.001156832	39	0.27846	240.7083												

Table E6b: 7 Curing Days Period from 175 – 375 penetrations

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
2.302632	175	0.0230263	0.001160727	46	0.32844	282.9681												
2.631579	200	0.0263158	0.001164649	52	0.37128	318.7966												
2.960526	225	0.0296053	0.001168597	58	0.41412	354.3757												
3.289474	250	0.0328947	0.001172571	60	0.4284	365.3489												
3.618421	275	0.0361842	0.001176573	65	0.4641	394.4441												
3.947368	300	0.0394737	0.001180603	66	0.47124	399.1411												
4.276316	325	0.0427632	0.00118466	66	0.47124	397.7851												
4.605263	350	0.0460526	0.001188745	62	0.44268	372.3928												
4.934211	375	0.0493421	0.001192858	56	0.39984	335.1949												

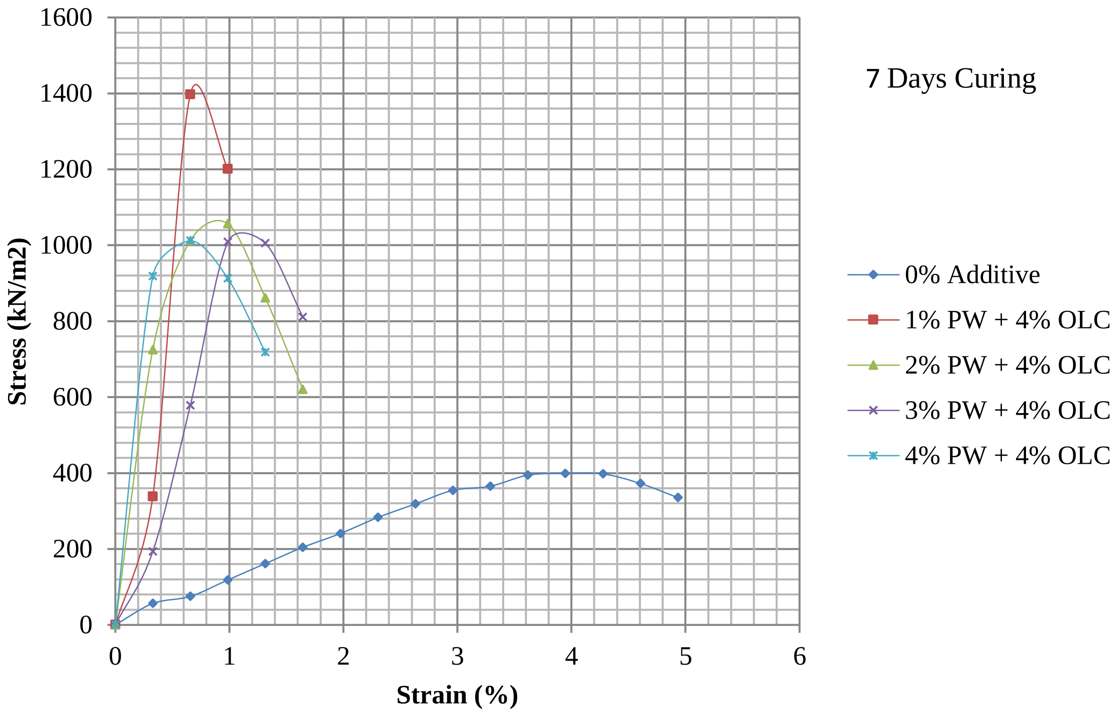

Figure E3: Axial stress vs. Axial stress for lime + plastic treated soil after 7 days curing

Table E7a: 14 Curing Days Period from 0 – 125 penetration

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
0	0	0	0.001134	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.328947	25	0.0032895	0.0011377743	3	0.165	145.0225	10	0.55	483.4083	8	0.44	386.7266	7	0.385	338.3858	14	0.77	676.7716
0.657895	50	0.0065789	0.00114151	4.2	0.231	202.3593	27	1.485	1300.881	30	1.65	1445.423	19	1.045	915.4347	25.5	1.4025	1228.61
0.986842	75	0.0098684	0.001145302	5.5	0.3025	264.114	29	1.595	1392.601	34	1.87	1632.705	30	1.65	1440.622	27	1.485	1296.56
1.315789	100	0.0131579	0.00114912	6.5	0.3575	311.1259	35	1.925	1675.293	30	1.65	1435.966	25	1.375	1196.638	26	1.43	1244.504
1.644737	125	0.0164474	0.001152963	7	0.385	333.9383	30	1.65	1431.164							13	0.715	620.1711

Table E7b: 14 Curing Days Period from 150 – 300 penetration

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
1.973684	150	0.0197368	0.001156832	8	0.44	380.3478												
2.302632	175	0.0230263	0.001160727	8.5	0.4675	402.7756												
2.631579	200	0.0263158	0.001164649	9	0.495	425.0278												
2.960526	225	0.0296053	0.001168597	9.1	0.5005	428.2938												
3.289474	250	0.0328947	0.001172571	9.1	0.5005	426.8373												
3.618421	275	0.0361842	0.001176573	8.8	0.484	411.3573												
3.947368	300	0.0394737	0.001180603	8	0.44	372.6808												

Table E7c: 14 Curing Days Period from 325 – 350 penetration

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m²	Dial reading	Load (kN)	Stress kN/m²	Dial reading	Load (kN)	Stress kN/m²	Dial reading	Load (kN)	Stress kN/m²	Dial reading	Load (kN)	Stress kN/m²
4.276316	325	0.0427632	0.00118466	7.5	0.4125	348.2012												
4.605263	350	0.0460526	0.001188745	7	0.385	323.871												

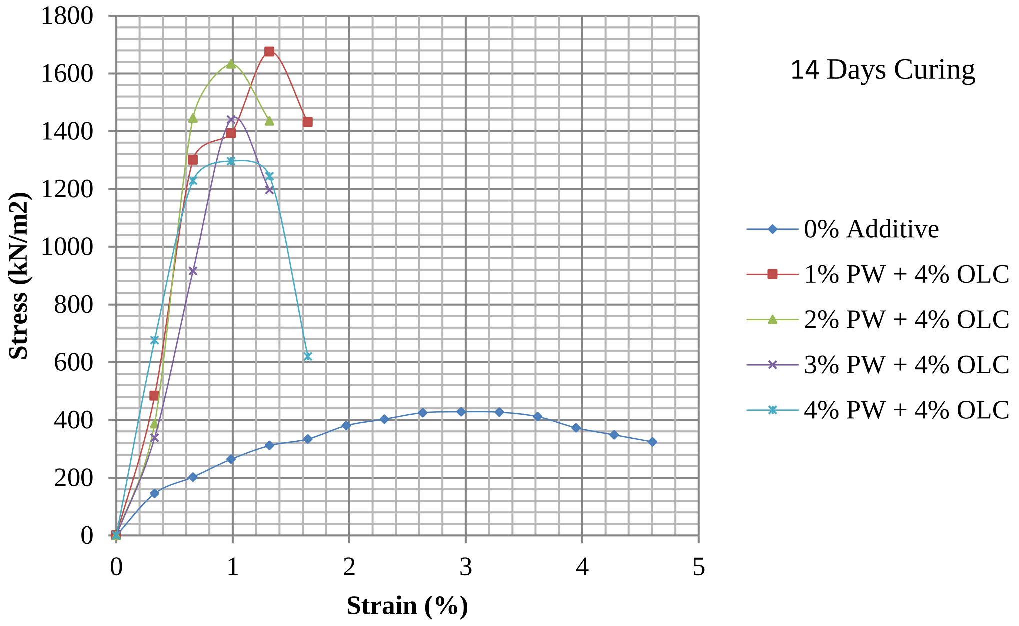

Figure E4: Axial stress vs. Axial stress for lime + plastic treated soil after 14 days curing

Table E8a: 28 Curing Days Period from 0 – 125 penetrations

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
0	0	0	0.001134	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.328947	25	0.0032895	0.001137743	2.7	0.1485	130.5202	6.5	0.3575	314.2154	8	0.44	386.7266	9	0.495	435.0675	10	0.55	483.4083
0.657895	50	0.0065789	0.00114151	4.1	0.2255	197.5412	18	0.99	867.254	21	1.155	1011.796	14	0.77	674.5309	20	1.1	963.6155
0.986842	75	0.0098684	0.001145302	5.3	0.2915	254.5098	25	1.375	1200.518	33	1.815	1584.684	19	1.045	912.3937	28	1.54	1344.58
1.315789	100	0.0131579	0.00114912	6.4	0.352	306.3393	31	1.705	1483.831	36	1.98	1723.159	29	1.595	1388.1	24	1.32	1148.772
1.644737	125	0.0164474	0.001152963	7.5	0.4125	357.791	32	1.76	1526.575	28	1.54	1335.753	33	1.815	1574.28			

Table E8b: 28 Curing Days Period from 150 – 300 penetrations

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
1.973684	150	0.0197368	0.001156832	8	0.44	380.3478	27	1.485	1283.674				35.5	1.9525	1687.793			
2.302632	175	0.0230263	0.001160727	9	0.495	426.4683							29	1.595	1374.175			
2.631579	200	0.0263158	0.001164649	9.3	0.5115	439.1954												
2.960526	225	0.0296053	0.001168597	10	0.55	470.6526												
3.289474	250	0.0328947	0.001172571	10.1	0.5555	473.7425												
3.618421	275	0.0361842	0.001176573	10.1	0.5555	472.126												
3.947368	300	0.0394737	0.001180603	10.2	0.561	475.168												

Table E8c: 28 Curing Days Period from 325 – 425 penetrations

				0% PW + 0% Lime			1% PW + 4% Lime			2% PW + 4% Lime			3% PW + 4% Lime			4% PW + 4% Lime		
Strain (%)	Penetration (x0.01 mm)	Straine	Anew	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2	Dial reading	Load (kN)	Stress kN/m^2
4.276316	325	0.0427632	0.00118466	10	0.55	464.2683												
4.605263	350	0.0460526	0.00118745	9.9	0.5445	458.0462												
4.934211	375	0.0493421	0.001192858	9.5	0.5225	438.0236												
5.263158	400	0.0526316	0.001197	8.9	0.4895	408.939												
5.592105	425	0.0559211	0.001201171	8	0.44	366.3093												

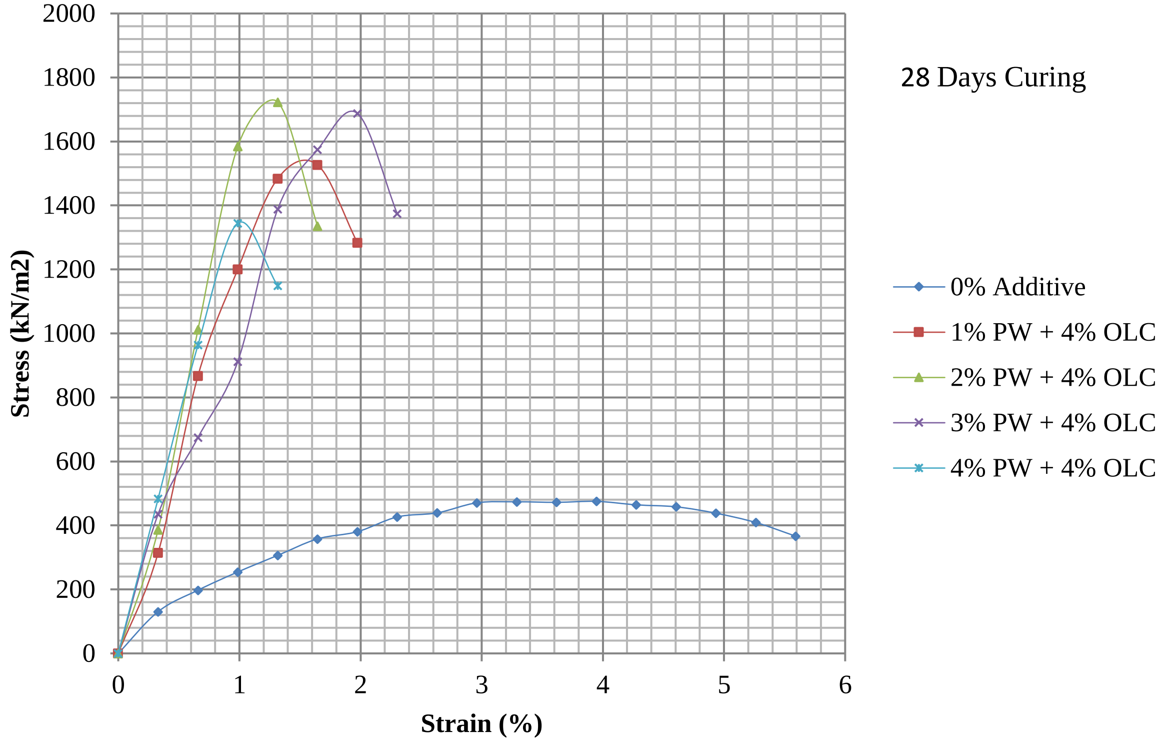

Figure E5: Axial stress vs. Axial stress for lime + plastic treated soil after 28 days curing

Table E9: Variation of UCS values with days of curing for different percentages of lime – plastic blend

Curing days	UCS kN/m^2 at 0% lime + 0% plastic waste	4%lime + 1% plastic waste	4%lime + 2% plastic waste	4%lime + 3% plastic waste	4%lime + 4% plastic waste
0	388.38	792.25	637.88	754.07	412.65
7	399.14	1397.24	1056.5	1008.4	1011.8
14	428.29	1675.29	1632.7	1440.62	1296.56
28	475.17	1526.58	1723.2	1687.79	1344.58

APPENDIX F: Soil Classification Systems

Table F1: Classification of Clay Based on Plasticity

Liquid Limit	Clay classification
LL = < 35%	Clay of Low Plasticity (CL)
LL = 35 – 50%	Clay of Intermediate Plasticity (CI)
LL = 50 – 70%	Clay of High Plasticity (CH)
LL = 70 – 90%	Clay of very High Plasticity (CV)
LL = >90%	Clay of Extremely High Plasticity (CE)

(Source: John, 2000)

Table F2: British Soil Classification for Fine Soils

British Soil Classification System for Fine Grain				
Soil group		Symbol		Recommended name
Fine soils		>35% fines	Liquid limit%	
	M	MG		Gravelly Silt
Silt		MS		Sandy Silt
		ML, MI…		(Plasticity Subdivisions as for Clay)
	C	CG		Gravelly Clay
		CS		Sandy Clay
		CL	<35	Clay of Low Plasticity
Clay		CI	35 – 50	Clay of Intermediate Plasticity
		CH	50 – 70	Clay of High Plasticity
		CV	70 – 90	Clay of Very High Plasticity
		CE	> 90	Clay of Extremely High Plasticity
Organic soils	O			(Add Letter ‘O’ to Group Symbol)
Peat	Pt			(Soil Predominantly Fibrous and Organic)

(Source : John, 2000)

Table F3: Nominal Strength Classification of Materials in the Design Catalogue (satcc)

Layer	Material	Nominal Strength
Base	Granular	Soaked CBR > 80% at 98% modified AASHTO density
	Cemented	7 days UCS*1.5 – 3.0 MPa at 100% modified AASHTO density (1.0 – 1.5MPa at 97% if modified test is followed)
	Bituminous	See specification
Sub-base	Granular	Soaked CBR > 30% at 95% modified AASHTO density
	Cemented	7 days UCS*0.75 – 1.5 MPa at 100% modified AASHTO density (0.5 – 0.75 MPa at 97% if modified test is followed)
Capping/selected	Granular	Soaked CBR > 15% at 93% modified AASHTO density

*7 day unconfined compressive strength

Nominal strength classification of materials in the design catalogue in the Federal Ministry of Works highway manual (2013).

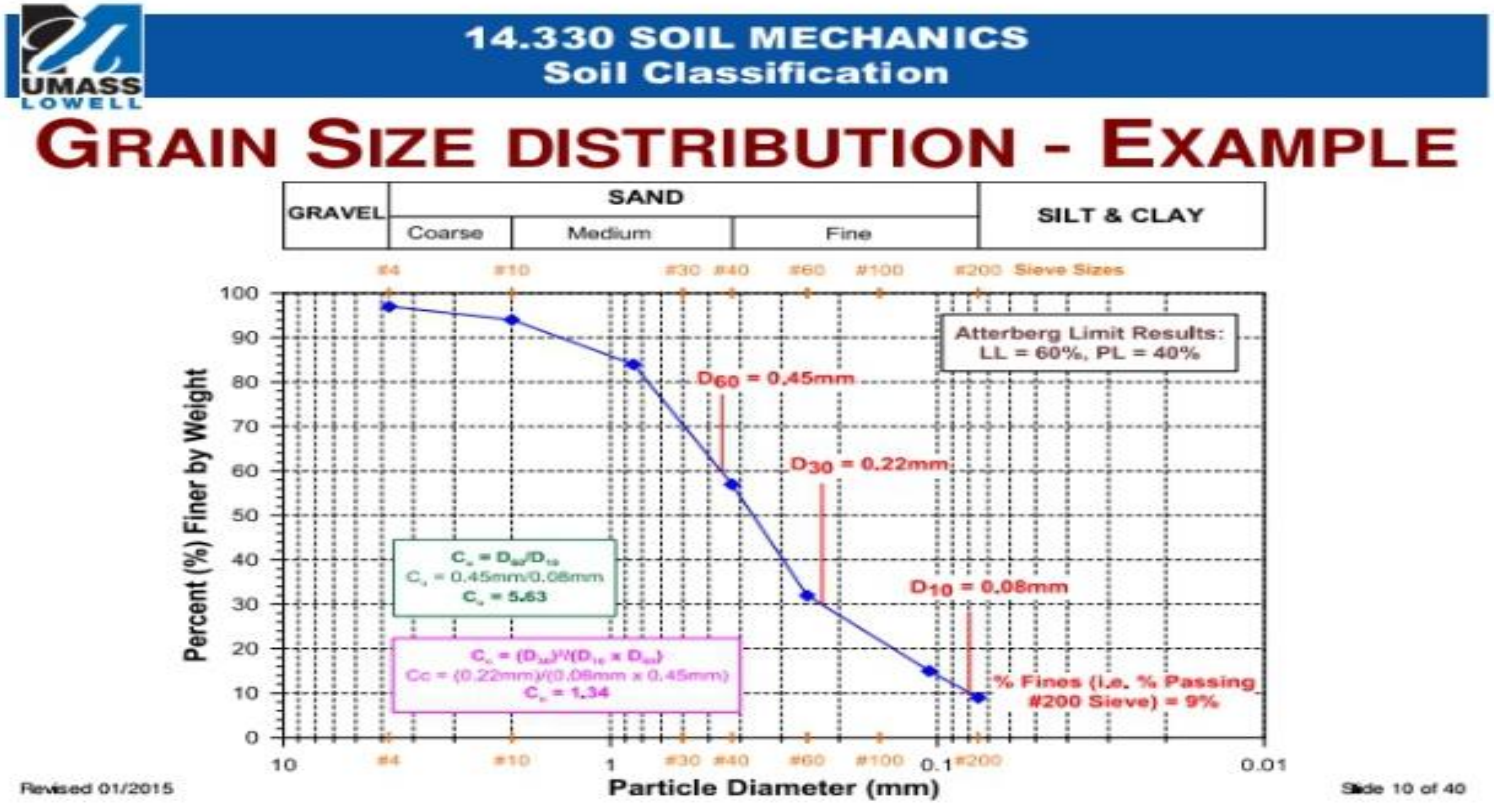

Figure F1: AASHTO (2015).Soil classification basics for grain size distribution

Table 5.1. AASHTO Classification System

General Classification	*Granular materials (35% or less passing No. 200 Sieve (0.075 mm)*							*Silt-clay Materials More than 35% passing No. 200 Sieve (0.075 mm)*			
Group Classification	*A—1*		*A—3*	*A—2*				*A—4*	*A—5*	*A—6*	*A—7*
	A—1—a	*A—1—b*		*A—2—4*	*A—2—5*	*A—2—6*	*A—2—7*				*A—7—5* *A—7—6*
(*a*) Sieve Analysis: Percent Passing											
(*i*) 2.00 mm (No. 10)	50 max										
(*ii*) 0.425 mm (No. 40)	30 max	50 max	51 min								
(*iii*) 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
(*b*) Characteristics of fraction passing 0.425 mm (No. 40)											
(*i*) Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
(*ii*) Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min*
(*c*) Usual types of significant Constituent materials	Stone Fragments Gravel and sand		Fine Sand	Silty or Clayey Gravel Sand				Silty Soils		Clayey Soils	
(*d*) General rating as subgrade.	Excellent to Good							Fair to Poor			

* If plasticity index is equal to or less than (liquid Limit—30), the soil is A—7—5 (*i.e.* PL > 30%)
If plasticity index is greater than (Liquid Limit—30), the soil is A—7—6 (*i.e.* PL < 30%)

Figure F2: AASHTO Soil classification system – AASHTO Chart by Haseeb (2019)

14.330 SOIL MECHANICS
Soil Classification

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO)

TABLE 2 Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 % or less passing No. 200)							Silt-Clay Materials (More than 35 % passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, % passing:											
No. 10 (2.00 mm)	50 max	...	...	...	...	...	...	...	...	...	...
No. 40 (425 μm)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
No. 200 (75 μm)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (425 μm):											
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 (see Fig. 1).
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Go from Left to Right, Process of Elimination

Table 2 from **ASTM D3282-09** *Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes.*

Revised 01/2015

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Figure F3: AASHTO Classification of soils and soil-aggregates-mixtures

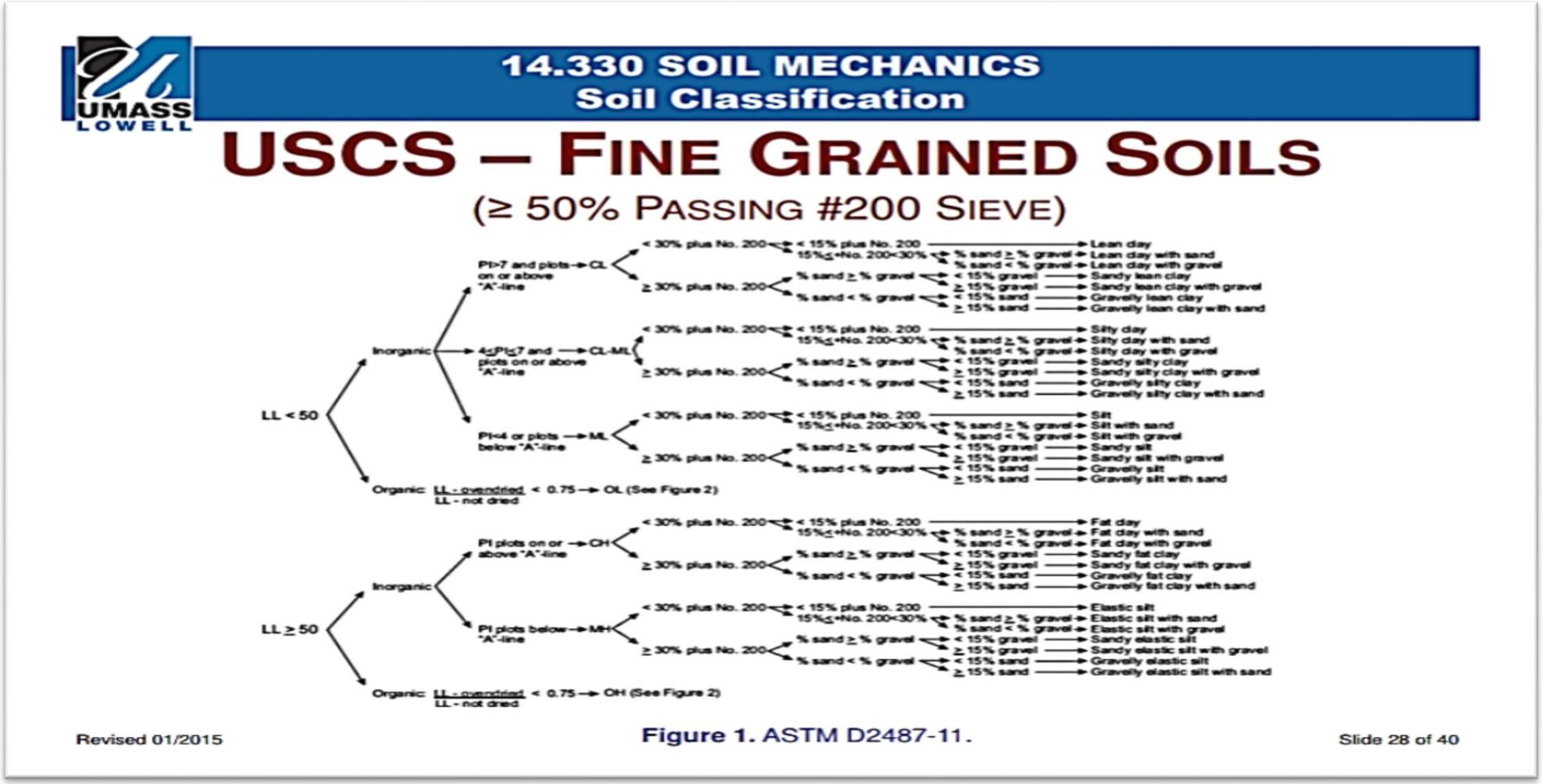

Figure F4: Unified Soil Classification System (USCS) for fine grained soils