STRENGTH PROPERTIES OF LATERITE STABILIZED WITH CEMENT AND SAND

BY

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CERTIFICATION

This is to certify that this project topic titled "Strength Properties of Laterite Stabilized with Cement and Sand" was undertaken by Nzekwabam Chukwudi Anthony with registration number (NAU/2017224032) in the Department of Civil Engineering, Nnamdi Azikiwe University, Awka, Anambra State.

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APPROVAL PAGE

This research work "Strength Properties of Laterite Stabilized with Cement and Sand" has been assessed and approved by department of civil engineering Nnamdi Azikiwe University.

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DEDICATION

This work is dedicated to Almighty God for the gift of life and also for guiding me through school.

ACKNOWLEDGEMENT

Special thanks go to Almighty God for giving me the strength to complete this work and also for His guidance and protection throughout my stay in Nnamdi Azikiwe University.

Also, I will like to express my profound gratitude to my parents; Mr. and Mrs. Samuel Nzekwabam and to my Uncle and wife, Mr. and Mrs. Peter Ewuzie for their moral support, constant prayers throughout my stay in school, special thanks goes also to my siblings Chima, Chidinma, and Chukwunwike for their encouragement during trying times of my academic pursuit.

I want to thank in a very special way, my project supervisor in the person of Engr B.A .Njotea for her time and guidance in the accomplishment of this project. May the lord enlarge your coast and also protect your family.

Special thanks go to the Head of Department of Civil Engineering in the person of Engr. Prof. C. A. Ezeagu. He has been more like a father to us all and I appreciate him so much.

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Finally, I will like to appreciate everyone who has in one way or the other contributed to making me a better person, may the Lord Almighty reward you all greatly

ABSTRACT

The study was undertaken to evaluate the effect of sand and cement on strength properties of laterite. Portland cement known as Dangote 3x cement was used for the study. The cement was added to laterite in an increasing order of 4%, 8%, 12%, 16% and 20% by dry weight of laterite while sand sample was added to laterite in an increasing order of 8%, 16%, 24%, 32% and 40% by dry weight of laterite. The mixtures were subjected to geotechnical testing. Test conducted were sieve analysis test, specific gravity test, atterberg limit test, compaction and CBR test. Results obtained from sieve analysis test revealed that laterite and sand and laterite were classified as A-2-4 and A-2-6 according to AASHTO Soil Classification System, and SM and SC according to Unified Soil Classification System, the specific gravity of laterite increased from 2.65 to 2.74 on addition of sand and cement at 24% and 12% respectively, the liquid limit, plastic limit and plasticity index of laterite decreased on addition of sand and cement to laterite, it was observed that at 12% sand and 24% cement, the mixture became non plastic. Compaction test showed that the maximum dry unit weight of laterite increased on addition of cement up to 12% beyond 12% cement content, the maximum dry unit weight was found to decrease while for laterite to sand mixture, the maximum dry unit weight of laterite increased on consist addition of sand to laterite, while for a blend of sand and cement added to laterite, the maximum dry unit weight of laterite increased up to 12% cement and 24% sand content beyond this point, the maximum dry unit weight was found to decrease. Assessment of the CBR of laterite stabilized with sand and cement revealed that the CBR of laterite was found to increase for all combinations (sand to latertie, cement to laterite and a . It was concluded that on addition of sand and cement to laterite, the laterite samples satisfied the criterion for use as sub-base type 1 and 2 material as recommended by the Federal Ministry of Works and Housing.

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CHAPTER ONE

INTRODUCTION

1.1 Background of Study

Laterite are highly weathered natural materials with a high concentration of hydrated oxides of iron or aluminum as a result of residual accumulation or absolute enrichment caused by the solution movement, and chemical precipitation of aluminum, iron, and manganese Oluyemi, (2019). Lateritic soils, the most prevalent of all tropical soils in Nigeria, are the most commonly used earth resources for highway construction (Hagos,2017). During the last decade, the global demand for indigenous lateritic soil has continued to increase (Osinubi, 2004). This growing demand has generated interest in the use of this red tropical soil as a building and road construction material especially in developing countries like Nigeria.

Lateritic soils are considerably affected by weathering due to the presence of meteorized materials enriched by minerals with poor solubility (e.g., iron and aluminum oxides), known as laterite gravel (Okonkwo, et al, 2022). They typically do not meet the standards required by road and building agencies for high rise building and traffic road pavement and, in certain circumstances, medium to light traffic as well buildings. This can be related to their particle-size characteristics, the type and strength of gravel particles, the degree of compaction, the volume of traffic, the climatic and hydrological regime of the construction site, and the geography of the area (Biswal, et al., 2018).

Because of its swelling nature, lateritic soil is always difficult for engineering projects. When dry, it contracts and when wet, it expands (Setiawan, et al., 2020). Laterites range in color from yellowish to reddish-brown, depending on the amounts of iron and aluminum sesquioxides. Different methods are used to improve the geotechnical characteristics of laterites to meet the criteria for sub-base, base course materials and fill in foundations of buildings. Preloading, soil replacement, the use of recycled concrete aggregates, and the use of soil stabilizing chemicals are among these methods (Eisazadeh, et al., 2012; Huan, et al., 2010; Jitsangiam, et al., 2015; Mengue, 2017; Musec, 2018).

Soil stabilization is any procedure that improves and makes a soil material more stable, resulting in increased bearing capacity and plasticity, increased mechanical strength or stiffness, altered grain size distribution, and durability under severe moisture and stress conditions (Okonkwo, et al., 2022). Soil stabilization can be accomplished mechanically or chemically. Mechanical stabilization entails the addition of one kind of soil to a parent soil or aggregate in order to increase its strength and stability by densifying the soil using mechanical energy (Geremew, 2018).

To enhance the geotechnical qualities of natural soil, one alternative to mechanical stabilization is chemical stabilization, which involves the addition of additives such as lime, cement, fly ash, and bitumen to the soil (Correia, 2016). Sand has been reported to improve the engineering qualities of natural soils. Due to a lack of sand and silt size particles, laterite gravels are gap graded; the addition of sand may enhance the grading curve and compaction properties of the laterites, hence reducing the flexibility of the fines and fines' characteristics (Mahmud, et al., 2016). Soil-cement is a basic yet highly compacted combination of soil, cement, and water (Okonkwo, et al., 2022).

When cement is blended with the other two ingredients, it increases the soil's characteristics, providing the finished material with the durability to handle traffic loading. This is all dependent on the kind of soil used, the amount of cement applied, the amount of moisture present, and the compaction of the mixture Petry and Little, (2002). The use of a cement stabilized foundation to reinforce the base section directly beneath rigid or flexible pavements is very common in highway construction (Okonkwo, et al., 2022). Roads, parking lots, airports, residential streets, and other structures can all benefit from the soil-cement pavement. It's a low-cost pavement base that's recognized for its strength and longevity (Ali, 2018: Kunduri and Mustapha, 2008).

This study will therefore seek to enhance the strength properties of lateritic soils for use sub-base, base course and fill in building through stabilization with Portland cement and sand.

1.2 Statement of Problem

Lateritic soils are considerably affected by weathering due to the presence of meteorized materials enriched by minerals with poor solubility (e.g., iron and aluminium oxides), known as laterite gravel (Okonkwo, et al., 2022). They typically do not meet the standards required by road and building agencies for high rise building and traffic road pavement and, in certain circumstances, medium to light traffic as well buildings. This can be related to their particle-size characteristics, the type and strength of gravel particles, the degree of compaction, the volume of traffic, the climatic and hydrological regime of the construction site, and the geography of the area (Biswal, et al., 2018: Makasa, 2007).

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In other to enhance the geotechnical properties of laterite thereby making them fit for use as fill material in building and at the sub-base and base course level of pavement, this study will investigate the effect of Portland cement and sand on geotechnical properties of laterite.

1.3 Aim and Objectives of Study

The aim of study is to investigate the effect of partial replacement of laterite with cement and sand on strength properties of laterite while the objectives are:

1 To determine the index properties of the natural lateritic soils.
2 To determine the index properties of sand.
3 To study the effect of sand and cement on compaction and strength properties of laterite.
4 To ascertain the maximum amount of sand and cement required for optimum improvement in strength properties of laterite.
5 To draw conclusion and make recommendation in the light of the findings obtained from the study.

1.4 Scope of Study

The study is limited to the use of Portland cement and sand as a partial substitute for enhancement of strength properties of lateritic soils. Laterite used for the experimental study will be partially replaced with cement in increasing percentages of 4, 8, 12, 16 and 20% by weight of laterite while sand will be added to the natural laterite soil in a stepped increase of 8% to 40% by dry weight of the laterite samples. Lateritic soils partially stabilized with cement and sand will be subjected to various testing. These tests are: sieve analysis, specific gravity, atterberg limit (liquid and plastic limit), compaction and California bearing ratio test. Results obtained from compaction and

California ratio test will be used as basis for making evaluative deduction on strength properties of the natural lateritic soils.

1.5 Significance of Study

Key findings obtained from the experimental study on effect of partial replacement of laterite with sand and cement will be significant in the following ways:

1. Serve as cost effective means of enhancing geotechnical properties of natural lateritic soils.
2. Tackle knowledge gap in material selection for cost effective improvement of lateritic soils.
3. Foster infrastructural development through frequent and high level of building and road construction.
4. Valuable as reference material for geotechnical studies and construction works.

CHAPTER TWO

LITERATURE REVIEW

2.1 Origin of Laterite

Lateritic soils are highly weathered and altered residual soils formed by the in-situ weathering and decomposition of parent rocks under tropical and subtropical climatic conditions (Aginam, et al., 2015). This weathering process primarily involves the continuous chemical alteration of minerals, the release of iron and aluminum oxides and the removal of bases and silica in the rocks. Lateritic soils are void or nearly void of bases primarily silicates, but may contain substantial amount of quartz and kaolinite (Alexander and cady, 2013). They are formed in hot, wet tropical regions with an annual rainfall of at least 1200mm and a daily temperature in excess of 25^oC and typically occur in humid tropical climate with 30^{o}N and 30^{o}S of the equator (Madu, 2010). They are also composed entirely of iron and aluminum oxide. They are reddish in colour and are the least soluble of rock weathering in tropical climate (Plummer, et al., 2013). Laterite is also described as a product of in-situ weathering in igneous, sedimentary and metamorphic rocks commonly found under unsaturated conditions (Rhardjo, et al 2014). Lateritic soil is one of the most common and important material used in earth work engineering construction in the tropics and subtropics where it is in abundance.

The name laterite was coined by an English surgeon Francis Buchanan in 1807 in India from a Latin word “later” meaning brick. In the 19th century, He coined the term laterite when he wrote “What I have called indurate clay is one of the most valuable materials for building. It is diffused in immense masses without any appearance of stratification and is placed over the granite that forms the bases of Malayala. It is full of cavities and pores and contains a very large quantity of quartz in the form of yellow and red ochres In the masses, while excluded from the air It is so soft, that any iron instrument readily cut it, and it is dug up in square masses with a pick-axle, and immediately cut into shape wanted with a trowel or large knife. It very soon become as hard as brick, and resists the air and water much better than materials made from bricks. The most proper English name would be laterite, from lateritis, the appellation that may be given to it in sciences”. Since then lot of researches have been carried out on laterite and a lot of terms referring to many soil types have been produced. There is a tendency to apply the term to any red soil and rocks in the tropics (Abebaw, 2014). Nearly all kind of rock can be deeply decomposed by action of high rainfall and elevated temperature. The percolating rainwater causes dissolution of primary rock material and a decrease of soluble elements such as sodium, potassium, calcium and magnesium. As a result, there remain a residual concentration of insoluble element predominantly iron and aluminum. In geosciences, only those weathered products that are most strongly altered geochemically and mineralogically are termed laterite.

2.2 Formation of Laterite

Tuncer and Lohnes, (2014) described the genesis of laterite as the weathering process which involves leaching of silica, formation of colloidal oxide and precipitation of the oxide with increasing crystallinity and dehydration as the soil is weathered. The major processes of weathering are physical, chemical and biological process. The physical weathering is predominant in the dry climate while the extent and rate of chemical weathering is largely controlled by the availability of moisture and temperature (Abebaw, 2014). As the disintegration of underlying rock occurs, the primary element are broken down by the process of physical and chemical weathering to simple ionic form. The silica and bases in the weathered material such as sodium, potassium, calcium and magnesium are washed out by the percolated rain water (verdose water), boxides and hydroxides of sesquioxide are accumulated thereby enriching the soil and giving the soil it's characteristic red colour. This process is termed laterization and it depends on the nature and extent of chemical weathering.

Laterization is the weathering process by which the rock is transformed into laterite. It is a gradual process which must be active for centuries. In tropical countries the "verdose water" is at high temperature and as a result they may contain more carbonic acids, alkaline, carbonates and organic matter. This element explains why rocks that are leached by verdose water are commonly found in tropical countries than in temperate ones. After weathering, dehydration occurs. Dehydration (either partial or complete) alters the composition and distribution of the sesquioxide rich material in a manner which is generally not reversible over wetting (Abebaw, 2014). It leads to the formation of strongly cemented soil with a unique granular soil structure. The topography and drainage of an area also influences the rate of weathering because to some extent, it determines the amount of water available for laterization to occur and the rate at which it moves through the weathering zone. The rate at which weathered material is eroded is also controlled.

Deep weathering cannot occur on steep slopes this is because the surface run-off on steep slopes is greater than the rate of infiltration thereby increasing the rate of erosion. Hence lateritic soils tend to be found on slopes (sometimes locally termed ridge gravel), to a lesser extent on uplands and rarely in poorly drained areas (Jiregna, 2012). The structure of Lateritic soil varies with the type of parent rock from which it was formed, the location (i.e. where it was formed) and also the weathering process that lead to its formation. Studies in some lateritic soils shows that they have porous granular structure consisting of iron impregnated clayey material in minute spherical aggregation (Hamilton, 2013). The aggregation derives its strength from the film found within the micro-joints of the elementary clay particles, which in addition coats the particles (Gidigasu, 1988).Thus the film found the micro joints of the elementary clay particles and as coatings over particles provides the strength of aggregation. Viewing carefully prepared thin sections of laterite under the optical microscope has shown that these soils contain rough materials with sizes tending from silt to fine sand spread throughout the soil with very finely-divided iron oxide, and a porous structure of peds or clay clusters which are usually not cemented by coatings of iron oxide but rather, they are weakly bonded. The surface of laterite soil initially exists as a gelatinous coating. After losing moisture, it becomes denser but retains its non-crystalline structure after which it crystallizes slowly into different forms, which gives them strongly cemented surfaces covered by iron oxides (Sergeyev, et al 2012). The structural development depends on the deposition of iron oxides at different stages of weathering process.

Lateritic soil chemistry and mineralogy as shown by studies greatly influence the geotechnical properties, and in certain circumstances, significantly affects the economic potential in the construction industry (Ogunsanwo, 1995). Studies by (Tuncer and Lohnes, 2014) also revealed that the degree of weathering is very well connected with the mineralogy of laterite, as the kaolinite content is high in the early stage of weathering and decrease with increase in weathering, whereas the amount of sesquioxide increases. The soil profile of laterite is defined as that in which laterite horizon exists or is capable of developing under favorable conditions (Ikiensinma, 2012). The alteration of rock by the processes of chemical weathering take place progressively through a series of events and stages which result in a profile of weathering. Lateritic gravels stand out as low humps in the terrain. They consist of gravel sized concretionary nodules in a matrix of silt and clay. They may take up an area of several hectares and a thickness of between 1 to 5m (Jiregna, 2012).

2.3 Properties of Laterite

2.3.1 Chemical Properties

Mallet, (1983) was perhaps the first to introduce the chemical concept for establishing the ferruginous and aluminum nature of lateritic soils. (Fermor, 2010) defined various forms of lateritic soils on the basis of the relative contents of the so-called lateritic constituents (Iron, Aluminum, Titanium, and Manganese) in relation to silica. Also, (Lacroix, 2014) divided laterite into: -true laterite, silicate laterite, and lateritic clays depending on the relative content of the hydroixides. There are other several attempts by the researchers to classify laterite in terms of their chemical compositions, but (Fox, 1996) has demonstrated that such classification are inadequate, other than in relations to deposits that may be exploited for their mineral content, classification based on chemical composition cannot be used to distinguish between indurate and softer formations.

The high content of the sesquioxides of iron or aluminum relative to other components is a feature of laterite. These essential components are mixed in variable proportions. Some laterite may contain more than 80% of Fe_2O_3 and little of Al_2O_3, While others may contain up to 60% of Al_2O_3 and a little of Fe_2O_3. Although alkali and alkaline bases are almost entirely absent in most cases, this is not an absolute criterion. In particular, some ferruginous tropical soils may contain significant amounts of alkaline bases. Combined silica content is low in sesquioxides. This combined silica is predominantly in the form of Kaolinite, the characteristic clay mineral of most tropical formation.

2.3.2 Physical Properties

The physical properties of residual soils, commonly known as the index properties, vary from region to region due to their heterogeneous nature and highly variable degree of weathering controlled by regional climate and topographic conditions, and the nature of bedrock, (Nnadi, 1988). It also varies with the depth of the soil and can be determined by simple laboratory tests. Studies on the effect of weathering on the physical properties of lateritic soil by (Tuncer et al, 1977 and Rahardjo, 2014) have revealed the following;

1 Pore-size distribution varies with the degree of weathering.

2 Higher pore volume and larger range of pore-size distribution indicates advancement in the weathering stage.

3 Soil classification and Atterberg limits do not show any correlation to weathering.

4 High specific gravity is a good indication of advanced degree of weathering.

5 Soil aggregation increases with increasing weathering.

6 Position in the topographical site, and depth of soil in the profile.

7 Genesis and pedological factors (parent material, climate, vegetation, period of time in which the processes have operated).

2.3.3 Plasticity

Textural lateritic soils are very variable and may contain all fractions sizes; boulders, cobbles, gravel, sand, silt, and clay as well as concretionary rocks. The interaction of the soil particles at the micro scale is reflected in the atterberg limits of the soil at micro scale level. Knowledge of the atterberg limits may provide the following information:-

1. A basis for identification and classification of a given soil texture.
2. Strength and compressibility characteristics swell potential of the soil or the water holding capacity.

Atterberg limit depends on:

1. The clay content**:** plasticity increases with increase in clay content (Piaskowski, 1993).
2. Nature of soil minerals**:** only minerals with sheet-like or plate-like structures exhibit plasticity. This is attributed to the high specific surface areas and hence the increased contact in the shaped particles.
3. Chemical composition of the soil environment**:** the absorptive capacity of the colloidal surface of the actions and water molecules decrease as the ratio of silica to sesquioxides decreases (Baver, 1980).
4. Nature of exchangeable actions**:** this has a considerable influence upon the soil plasticity (Hough, 1989).

Pre-test preparation, degree of molding and time of mixing, dry and re-wetting, and irreversible changes may affect the plasticity of soil. Drying drives off absorbed water, which is not completely regained, on re-wetting (Fookes,1997). Studies on the relationship between the natural moisture content, liquid limits and plastic limits of laterite have shown that generally the natural moisture contents is less than the plastic limit in normal lateritic soils (Vargas,1993). However, the lateritic soil from high rain fall areas may have moisture contents as high as the liquid limit (Hirashima,1979).

2.3.4 Particle Size Distribution

Consequently, great importance has also been accorded to particle-size distribution in dealing with lateritic soils. Recent studies have revealed that lateritic soils are strikingly different from temperate zone soils in terms of genesis and structure. Their concretionary structure as compared to the dispersed temperate zone soils has necessitated modifications to mechanical or grading tests (Remillion,2007). Consistent reports of variations in the particle-size distribution with methods of pretreatment and testing have been widely reported on laterite soils. (Schofield,2012) found out that wet sieving increased the silt and clay fraction from 7 to 20% as compared to the dry sieving. It has been found that sodium hexametaphosphate generally gives better dispersion of the fine fractions. It was also found, for example, that using sodium oxalate on a halloysitic clay from Kenya gave between 20 and 30% clay fraction, while the sodium hexametaphosphate gave as high as between 40 to 50% clay fraction for the same soil (Quinones,2004).

Another factor which has been found to affect the sedimentation test is the method of drying. Oven-dried lateritic soils were found to give the least amount of clay fraction, as compared to air-dried (Mohr and Mazhar,1969). The decrease in the clay content was accompanied by an increase in silt and sand fraction contents as a result of the cementation and coagulation of the clay particles by free iron oxide into clusters (Terzaghi,1958). The variation in the grading of lateritic gravels with the method of manipulation is also widely reported (Novais-Ferreira and Correia 1965 and Nascimento et al., 1998). In the study of the particle-size distribution of lateritic soils, three sources of confusion were noted. The first confusion arises from the belief by some authors, e.g. (Bawa,2008), opined that lateritic soils represent a group of materials that can be defined within a specific range of particle-size distribution. The second source of confusion seems to arise out of attempts by some authors to confine the word laterite to concretionary lateritic gravels. The third source of confusion arises out of the attachment of unnecessary importance to the soil colour. (Nascimento et al.,1998) have suggested an interesting lithological classification of lateritic soils as follows:

Lateritic clays <0.002 mm

Lateritic silts =0.002 - 0.06 mm

Lateritic sands ~0.06- 2 mm

Lateritic gravel =2 - 60 mm

Laterite stones and cuirasse ≥ 60 mm

Studies on lateritic gravels by (de Graft-Johnson et al.,,1969) among others have shown that the grading, though important for identification purposes, cannot alone form the basis for grouping lateritic gravels in terms of mechanical properties. The strength of the aggregates was found to be an important factor. Studies of lateritic aggregates in Nigeria, has also established that the strength of the aggregates is mainly a function of the degree of maturity of the lateritic concretionary particles and the predominant sesquioxide in the aggregates.

2.3.5 Compaction Characteristics

The compaction characteristics of lateritic soils are determined by their grading characteristics and plasticity of fines. Most lateritic soils contain a mixture of quartz and concretionary coarse particles, which may vary from very hard to very soft. The strength of these particles has major implications in terms of field and laboratory compaction results and their subsequent performance in civil engineering construction projects. Placement variables (moisture content, amount of compaction, and type of compaction efforts) also influence the compaction characteristics. Varying each of these placement variables has an effect on permeability, compressibility, strength and stress-strain characteristics of the soil.

2.3.6 Shear Strength Characteristics

Shear strength is a term used in soil mechanics to describe the magnitude of the shear stress that a soil can sustain. The shear strength of a lateritic soil is a function of the friction and interlocking of particles (soil angle of internal friction) and possibly cementation or bonding at particle contact relative to total and effective stress. Due to cohesion, particulate materials may expand or contract in volume as it is subject to shear strains. If soil expands in volume, the density of particles will decrease and the strength will decrease likewise the shear strength.

The cohesion is attributable to the resultant of inter particle forces which are mainly associated with the clay-size particle of soils and will vary with the particle size and the distance separating them. The angle of internal friction included the effect of interlocking. The interlocking effect is affected to some degree by the shape of particles and the grain–size distribution. The two parameters cohesion (**c**) and angle of friction (ø) depends on the grading, particle shape and void ratio factors of the soil. Cohesion also depends on degree of saturation, while angle of internal friction does not (Gidigasu,1988).

The shear strength characteristics of lateritic soils have been found to depend significantly on the parent materials, and the degree of weathering which in turn depends on the position of the sample in the soil profile and compositional factors as well as the pretest preparation of the samples(Lohnes,1988).

2.3.7 Compressibility and Consolidation

When a soil mass is subjected to a compressive force, its volume decreases. The property of the soil due to which it decrease in volume occurs under compressive force is known as the compressibility of soil. The compression of soil can occur due to;

1. Compression of solid particles and water in the void
2. Compression and expulsion of air in the void
3. Expulsion of water in the voids

The compression of saturated soil under a steady static pressure is known as consolidation. It is entirely due to expulsion of water from the voids. The consolidation characteristics of lateritic soils is generally moderate with the modulus of compressibility ranging between 1×10^{-3} to 1×10^{-2} sq. ft./ton.

2.3.8 Specific Gravity

The available data indicate that specific gravities vary not only with the textural soil groups but also within different fractions. In the first place lateritic soils have been found to have very high specific gravities of between 2.6 to 3.4 (De Graft-Johnson and Bhatia, 1969). For the same soil, gravel fractions were found to have higher specific gravities than fine fractions due to the concentration of iron oxide in the gravel fraction while alumina is concentrated in the silt and clay fractions. It is common to find specific gravities reported for the gravel and fines separately. The average of the two values can be assumed to be more representative of the specific gravity for the whole soil.

2.4 Soil Stabilization

Soil stabilization is not new, but man has sought to accomplish it by various means almost since the first roads were built, but it is only in recent years that scientific methods has been applied to soil stabilization (Olugbenga and Adetuberu,2013). Soil stabilization maybe defined as the process of blending and mixing materials with a soil so as to improve certain properties of the soil. A stabilized material may be considered as a combination of binder soil and aggregates preferably obtained at or near the site of stabilization manipulated and treated with or without admixtures, and compacted so that it will remain in its compacted state without detrimental change in shape or volume under applied force or exposure to weather.

Stabilization signifies improvement in both strength and durability which are related to performance. Increase in strength may be expressed quantitatively in terms of compressive strength, shearing strength, or some measure of bearing value or load deflection to indicate the load bearing quality (Olugbenga and Adetuberu,2013). Stabilization is a method of processing available materials for the production of low cost roads and other civil engineering projects. In this type of project, design and construction, emphasis is definitely placed on the effective utilization of local materials, with a view to decrease construction cost. In some areas, naturally occurring aggregates and soil aggregate combinations exists which requires minimum processing for successful stabilization (Olugbenga and Adetuberu,2013). While in other places, the natural soils are of unfavorable character and require modification through the use of suitable components such as gravels, crushed stones, geosynthetics, natural fibers or clay binder. While in other areas, admixtures like bituminous materials, lime or Portland cement must be used for effective stabilization. The type and degree of stabilization is dependent on the availability and cost of the required materials.

2.5 Methods of Soil Stabilization

In road construction projects, soil or gravelly material is used as the road main body in pavement layers. To have required strength against tensile stresses and strains spectrum, the soil used for constructing pavement should have special specification. Through soil stabilization, unbound materials can be stabilized with cementitious materials (cement, lime, fly ash, bitumen or combination of these). The stabilized soil materials have a higher strength, lower permeability and lower compressibility than the native soil (Keller,2014). The method can be achieved in two ways, namely:

1 In-situ soil stabilization
2 Ex-situ soil stabilization

Stabilization is not necessary a magic wand by which every soil properties can be improved for better. The decision to technological usage depends on which soil properties have to be modified. The chief properties of soil which are of interest to engineers are volume stability, strength, compressibility, permeability and durability (Sherwood, 1993: Altabba and Evans, 2015). Some stabilization technique includes mechanical and chemical stabilization.

2.5.1 Mechanical Stabilization

Mechanical Stabilization is the process of improving the properties of the soil by changing its gradation moisture (Onyelowe and Chibuzor,2012). This process includes soil compaction and densification by application of mechanical energy using various sorts of rollers, rammers, vibration techniques and sometime blasting. The stability of the soil in this method relies on the inherent properties of the soil material moisture (Onyelowe and Chibuzor,2012). Two or more types of natural soils are mixed to obtain a composite material which is superior to any of its components. Mechanical stabilization is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification.

2.5.2 Chemical Stabilization

In order to improve the properties of expansive soil, a combination of chemical stabilizers such as cement, fly ash, and lime with chloride or individually can be used. About replacing soil particles to meet more stable soil structure, there are two main methods. Firstly, increasing the particle size by cementation to produce an increment in shear strength, reduction in plasticity index, and reduction in expansion potential. Secondly, improve the compaction and physical properties of the soil by using absorption and chemical binding of moisture (Onyelowe and Chibuzor,2012).

2.5.3 Types of Additives used in Soil Stabilization

There are many additives that have been used to improve the engineering properties of expansive soil. These additives can be classified as waste materials such as dust, agricultural wastes, synthetic wastes, and organic wastes to enhance the economic cost.

Table 2.1: Additives Employed in Soil Stabilization Salem, (2018).

Industrial Solid Waste	Agricultural Solid Waste	Domestic Solid Waste	Mineral Solid Waste
Fly Ash	Rice Husk Ash	Incinerator Ash	Quarry Dust, Stone Dust or Chipping Dust
Cement Kiln Ash	Bagasse Ash	Waste Tire	Marble Dust
Silica Fume	Groundnut Shell Ash	Egg Shell Powder	Limestone Dust
Copper Slag	Plantain Peel Ash	Grain Storage Dust	Granite Dust
Granulated Blast Furnace Slag	Banana Leaf Ash	Glass Cullet	Mine Tailings
Phosphogypsum	Concob Ash		Baryte
Ceramic Dust	Guinea Corn Ash		
Brick Dust	Bamboo Leaf Ash		

2.6 Fine Aggregate (Sand)

Aggregates are generally divided into two groups: Fine and Coarse. Fine aggregate consists of natural or manufactured sand with particles sizes up to 5mm. It consists of inert natural sand conforming to BSI,(1992). It does not contain more than a total of 5% by weight of the followings: shale, silt and structurally weak particles Grow, (1938).

Aggregates make up or occupy 60% to 80% of concrete volume making its selection highly important (Neville,2000). Aggregate should consist of particles with adequate strength and resistance to exposure condition and should not contain materials that will cause deterioration of concrete. All natural aggregate particles originally formed a part of a larger parent mass. This may have been fragmented by natural processes of weathering and abrasion or artificially by crushing. Thus, many properties of the aggregate depend entirely on the properties of the parent rock, for example, chemical and mineral composition, petrologic character, specific gravity, hardness, strength, physical and chemical stability, pore structure and color (Neville,1981). Fine aggregates provide support function to the finer solids by producing voids of a size which do no contain or support the finer particles. Particle shape affects the behavior of the water, harsh angular aggregates not packing well and resulting in high void content (Neville,1979). Such aggregates may have a high surface area, but because of a lack of contact between the particles, it does not effectively control the finer particles. Smooth rounded aggregates have the disadvantage that, although theoretically it should pack together and produce low voids, this situation does not necessarily occur in a graded material of this type.

Aggregates for mortar must be clean, sharp and free from salt and organic contamination. Most natural aggregates contain a small quantity of silt or clay. A small quantity of silt improves workability. Marine or estuarine aggregate should not be used unless washed completely to remove the magnesium and sodium chloride salts which are deliquescent and attract moisture (Hendry, Sinha and Davies,1987). The most suitable aggregate would appear to be one that is well graded with a balance between rounded and angular particles and a surface texture that is not too smooth. In practice it has been found that a natural river aggregate with a grading complying with BSI, (1992) is the most suitable. Sea-dredged and crushed aggregates produce more extreme types, either all smooth and rounded or harsh and angular and generally requiring greater care in design. Sand is used in Nigeria as a fine aggregate in the building sector, as test samples in geotechnical and soil science laboratories, as an experimental porous media in hydrogeology investigations, and for other purposes (Okonkwo, et al,2022). The usage of sand for construction purposes has expanded rapidly as a result of the need for a more paved road of sand network and housing plans. The realization of the usefulness and effect of fine aggregate on the strength of lateritic soils used as a road construction material has put into the minds of Engineers and researchers to lay more emphasis into the study of its properties and usefulness. Emphasis is made on such properties like bulk density, specific gravity, silt content and particle size distribution.

2.6.1 Review of Past Works on Laterite Modified Sand Samples

An experimental study on evaluation of strength of laterite stabilized with sand was conducted by (Azu,2018). The laterite used for the study was stabilized with 2%, 4%, 6% and 8% sharp sand by weight of the soils. The results obtained shows an improvement in CBR values for sample A and the optimum sharp sand content should be 2% of weight of dry soil while that of sample B is 4% of dry weight of soil with CBR of 31.08% and 36.10% respectively.

Another experimental study on effect of sand on geotechnical properties of laterite was conducted by (Osinubi,2004). From the findings, it was deduced that the maximum dry density (MDD) and California Bearing Ratio (CBR) improved.

In another study (Okonkwo, et al.,2022) investigated the effect of stabilization of lateritic soils with Portland cement and sand for road pavement. The soil sample had a California Bearing Ratio (CBR) value of 24%. This demonstrated that the laterite was insufficient for both sub-base and base course materials for road pavement and so required stabilization. The soil was stabilized by adding different percentages of cement in the range of 3%, 6%, 9%, and 12% by weight, as well as various percentages of fine sand in the range of 15%, 30%, 45%, and 60% by weight. The soil was additionally stabilized using varying percentages of both cement and sand, for a total of 16 mix combinations. A soil-cement mixture with 6% cement gave the maximum CBR of 175%, while a CBR of 86% was obtained in a soil-sand mixture with 30% sand. For soil-cement-sand mixtures, mixtures containing 6% cement and 45% sand, as well as 9% cement and 45% sand, yielded a CBR value of 112%. Consequently, some soil-cement, soil-sand, and soil-cement-sand mixtures satisfied the criterion for road pavement sub-base and base course materials.

2.7 Portland Cement

Cement is a binder material, a substance made of burned lime and clay which after mixing with water, set and harden independently and can bind other materials together (Ezeokonkwo,2014). According to (Onwuka and Omerekpe,2003), cement as a hydraulic binders react exothermically with water to form hard strong masses with extremely low solubility. They consist of chemical compounds such as calcium silicate and calcium aluminates. Cement is a cementitious material which has adhesive and cohesive properties necessary to bound inert aggregates into a solid mass of adequate strength and durability. (Neville,2012) also adds that cement is the binding material constituent of concrete which reacts chemically with water and aggregate to form a hardened mass on hydrating. Iheama,(2010) further defines it as a finely pulverized product resulting from calcination of natural argillaceous limestone at a temperature below the fusion. In addition to this Ivor, (2014), defines cement as a mixture of compounds, consisting mainly of silicates and aluminates of calcium, formed out of calcium oxide, silica, aluminium oxide and iron oxide. Hydraulic cements are of four types: Portland cement, Blended Portland Cement, and Portland cement with addictives and High Alumina Cement. Cement varying chemical composition and physical characteristics exhibit different properties on hydration. The cement of desired properties can be produced by selecting suitable mixture of raw materials. The various types of Portland cement used in the construction industry are: Ordinary Portland Cement(OPC), Rapid Hardening Portland Cement(RHPC), Sulphate resisting Portland Cement(SRPC), Low Heat Portland Cement(LHPC), Blast Furnace Portland Cement(BFPC), Portland Pozzolana Cement(PPC), Modified Portland Slag Cement(MPC).

2.7.1 Soil stabilization using Portland cement.

Soil cement stabilization is soil particles bonding caused by hydration of the cement particles which grow into crystals that can interlock with one another giving a high compressive strength (Solihu,2020). In order to achieve a successful bond, the cement particles need to coat most of the material particles. To provide good contact between soil particles and cement, and thus efficient soil cement stabilization, mixing the cement and soil with certain particle size distribution is necessary (Habiba,2015). According to Solihu, (2020), advantages of soil stabilization using cement are: Cement is manufactured under strict ASTM standards, ensuring uniformity of quality and performance, it has a long-term performance record, Using cement can minimize volume increase compared with other reagents and cement is a non-proprietary manufactured product, readily available across the country in bag or bulk quantities.

2.7.2 Problem Associated With Cement Stabilization

Despite the many benefits, there are problems associated with cement stabilized materials that entail due considerations. The main problems that will have pronounced negative effects if not controlled are cracking and carbonation. This problem occurs mostly in compacted stabilized layer after construction (Eskedil,2014).

In cement-stabilized bases, cracking is attributed to materials characteristics, construction procedures, traffic loading, and restraint imposed on the base by the sub-grade (Eskedil,2014). The most common type of crack in cement-stabilized base is shrinkage crack. Shrinkage cracks are related to loss of water, cement content, density of compacted material, method of compaction, and pretreatment moisture content of the material to be stabilized. Cement treated materials begin to lose their moisture through evaporation immediately after they are placed if proper curing is not exercised. The loss of moisture then will lead to the drying and subsequent development of shrinkage cracks. Further, the final strength of the cement treated materials will be reduced as hydration of the cement is hampered due to lack of sufficient moisture in the mix (Eskedil,2014). The contribution of cement hydration in the development of shrinkage cracks is less as compared to water loss. Nevertheless, excessive amount of cement aggravates the development of cracks in two ways Solihu, (2020);

a) Higher amount of cement in the mix causes greater water consumption during hydration which in turn increases the drying shrinkage;
b) Increased amount of cement increases the rigidity and tensile strength of the treated materials. As a result, widely spaced wide cracks are developed. The wider spacing of the cracks is attributed to the higher tensile strength and the wider width of individual cracks is due to the distribution of total shrinkage of the material within smaller number of the widely spaced cracks Eskedil, (2014).

2.7.3 Effect of Cement on Properties of Lateritic Soils

An investigation on the effect of cement on compaction properties of lateritic soils was conducted by (Wahab, et al.2021). The soil samples were collected and tested for various basic soil properties tests such as Atterberg Limit, Specific Gravity (SG), Sieve Analysis, and Compaction test. Atterberg's consistency limit test shows that the liquid limit (LL) is 70.3%, the plastic limit (PL) is 42.0%, and the plasticity index (PI) is 28.3%. The specific gravity (SG) value for laterite is 2.74. For the compaction test, the optimum moisture content (OMC) and maximum dry density (MDD) obtained are 28% and 1.39 g/cm3. A laboratory study was performed to compare and evaluate the stabilization efficiency of different percentages (3%, 6%, 9%, 12%) of Ordinary Portland Cement

(OPC) when applied to the available laterite soil; a major soil group in the tropical areas. Analysis of laboratory data is assessed from a soil compaction test through the standard proctor method by using the automatic compactor. The soil mixtures were compacted at optimum moisture content in accordance with the British Standard (BS) of BS 1377-4:1990. From the preceding results, it was found that the OMC increase from 28% to 34% while the MDD increase from 1.39 g/cm³ to 1.47 g/cm³ with the rise in the percentage of cement.

In another study Okonkwo, et al., (2022) investigated the effect of stabilization of lateritic soils with Portland cement and sand for road pavement. The soil sample had a California Bearing Ratio (CBR) value of 24%. This demonstrated that the laterite was insufficient for both sub-base and base course materials for road pavement and so required stabilization. The soil was stabilized by adding different percentages of cement in the range of 3%, 6%, 9%, and 12% by weight, as well as various percentages of fine sand in the range of 15%, 30%, 45%, and 60% by weight. The soil was additionally stabilized using varying percentages of both cement and sand, for a total of 16 mix combinations. A soil-cement mixture with 6% cement gave the maximum CBR of 175%, while a CBR of 86% was obtained in a soil-sand mixture with 30% sand. For soil-cement-sand mixtures, mixtures containing 6% cement and 45% sand, as well as 9% cement and 45% sand, yielded a CBR value of 112%. Consequently, some soil-cement, soil-sand, and soil-cement-sand mixtures satisfied the criterion for road pavement sub-base and base course materials.

A review on cement soil stabilization as an improvement technique for rail track sub-grade, and highway sub-base and base courses was carried out by Solihu, (2020). Advantages and problems associated with soil stabilization using chemicals have also been briefly discussed in this report. It has been confirmed that ordinary Portland cement is an effective chemical stabilizer to improve both the index and strength properties of soils, however, the optima percentage of cement contents are varied from a soil type to another. In addition, further research has to be carried out as the percentage of cement content varies from region to region and from soil characteristics to another. This is necessary so as to determine the optimum percentage of cement content that would yield the desired sub-grade CBR values with some other index properties to meet the specified requirements in any selected design manual.

An experimental study on stabilization of engineering soil using Portland cement, coconut shell and husk ash as admixture was conducted by Onyelowe, (2016). The stabilization of laterite for improved engineering properties was investigated, and the geotechnical, chemical, and phase analytic method was used to characterize both the raw and treated laterite. Coconut Shell- Husk Ash (CSHA) was used as admixture for the stabilization in varying percentage at a constant percentage of Ordinary Portland Cement (OPC). The engineering soil used for this investigation was collected from Amizi, Olokoro in Umuahia South LGA, Abia State, Nigeria and preliminary tests carried out on the sample show that it is too brittle and thus not suitable as sub-base materials. The result of the sieve analysis and Atterberg limits tests graded the soil as Reddish Sandy Silt soil with a little high plasticity and it falls in the A-2-7 AASHTO classification system. It failed some of the standard requirement specified by the Ministry of Works and Housing in Nigeria. For instance, for the standard required 80% CBR, the sample had a value of 28% which is relatively low. The CBR test shows that the addition of cement at 5% by mass improves the soil, and further addition of varying percentages of CSHA in the order; 2%, 4%, 6%, 8% and 10% increased it relatively and it reached its peak of 82% at 8% CSHA and 5%OPC which is which is considerably satisfactory. The triaxial test result showed an improvement from Cu=23 KN/m2 and Ǿ=200 at its natural state to Cu=25 KN/m2 and Ǿ=290 thereby making the soil satisfactory for sub-base material in road pavement construction.

2.8 Summary of Research Findings

Researcher	Test Conducted	Percentages of Sand used.	Percentages of Portland cement used	Research Findings
Onyelowe, (2016)	Sieve analysis, Atterberg limit, Compaction, Triaxial and California bearing ratio test.	-	2, 4, 6, 8 and 10% by weight of laterite	Maximum dry unit weight and strength properties of the laterite samples was found to increase with consistent addition of Portland cement thereby making the soil satisfactory for sub-base material in road pavement construction.

Okonkwo, et al., (2022)	Sieve analysis, Atterberg limit, Compaction and California bearing ratio test.	15, 30, 45 and 60% by dry weight of the natural laterite samples.	3, 6, 9 and 12% by dry weight of laterite sample.	California bearing ratio of the laterite stabilized with cement and sand was found to increase thereby satisfying the requirement for use at the sub-base and base course level of road construction.
Wahab, et al., (2021)	Sieve analysis, Specific Gravity Atterberg limit, Compaction and California bearing ratio test.		3, 6, 9, 12 and 15% by dry weight of the natural laterite sample.	Maximum dry unit weight and corresponding optimum moisture content increased with consistent addition of cement to the natural laterite samples.
Azu, (2018)	Sieve analysis, Specific Gravity Atterberg limit, Compaction and California bearing ratio test.	2, 4, 6, 8 and 10% by weight of laterite sample.		California bearing ratio of laterite increased from it natural value up to 10% addition of sand.

However, the current study will bridge gaps identified from past work and investigate deeply on the strength properties of laterite stabilized with cement and sand. Sand sample will be used to partly replace laterite in a stepped increase of 8% to 40% by weight of laterite while laterite will be partially replaced with cement in a stepped increase of 4% to 20% by dry weight of the laterite samples.

CHAPTER THREE

MATERIALS AND METHODS

This section presents the materials and methods used to achieve the research work. Relevant standards were employed to ascertain how the properties of the materials collected from the respective study areas can be determined through various laboratory testing. All Tests such as Sieve analysis test, Specific gravity test, Atterberg limit test, Compaction and California bearing ratio test were carried out at Civil Engineering Laboratory located in Nnamdi Azikiwe University Awka Anambra State Nigeria. Below is a description of study materials and procedures for testing.

3.1 Collection and Preparation of Materials

3.1.1 Laterite

Natural reddish brown lateritic soil designated as LAT was obtained from borrow pit in Anambra state. Agu-Awka in Anambra State, Nigeria near Enugwu-Agidi along Amawbia to Igbariam Road. The choice of sites for collection of the lateritic soil samples was justified by the fact that it is a borrow pits where construction companies obtain their materials for building and road construction .The laterite sample was collected with the aid of a digger and a shovel at a depth of 300mm. The sample passed all physical tests that could classify them as lateritic soils in that, it is reddish-brown in colour, fine grained in texture and could become hard during the dry season. These samples were collected in four cement bags each and were conveyed via public transport to the school laboratory for various laboratory testing. The in-situ moisture content of the sample was determined using oven-dried method before air-drying for a period of two weeks in an open area using corrugated roofing sheets (commonly known as zinc) so as to ensure complete and even dissipation of moisture from the samples (i.e. zero moisture content).

3.1.2 Portland Cement

Ordinary Portland cement (Dangote cement) was used for the experimental study. This cement is designated as OPC. The cement was purchased at Onitsha Market in Anambra State. Upon purchase, the cement was conveyed to school laboratory where it was kept in a cool dry place preparatory for various laboratory testing. The cement sample satisfy the requirement for use as one of the major component of concrete in that, it was not caked or baked through visual inspection and quick setting time. Relevant laboratory test performed on the cement was soundness and fineness test. Portland cement used for the experimental study will be added to laterite in a stepped increase of 4% up to 20% by dry weight of laterite sample.

3.1.3 Sand

Sand sample used in stabiliting the natural laterite sample was collected at a construction site at Nnamdi Azikiwe University Campus. The sand was sieved through 5.0mm test sieve before it was added to the concrete mix to ensure uniformity of particle size and also to remove impurities. After sieving, the aggregate were air-dried to a saturated state of an aggregate. Sieve analysis of the aggregate was conducted according to the ASTM C136 (2006). The sample passed the necessary requirement for use as ingredient of concrete based on the fact that it is gritty with particle sizes visible to the naked eyes, physical properties of the sand samples were determined prior to its incorporation into the laterite. The laterite sample will be partly admixed with sand in a stepped increase of 8% to 40% by dry weight of laterite.

3.2 Sampling Locality

The lateritic soil samples, sand and cement used for the experimental study were collected at Onitsha and Nnamdi Azikiwe University campus Awka Anambra State. Enugwu Agidi where the laterite sample was collected is a small hilly community in Njikoka Local Government Area of Anambra State. It is situated in Awka South lies within longitude 6^o 13 North, to 7^o30lE and latitude 6^o00lN to 6^o30lN and is situated to the village Nawfia and Nwofia.

3.3 Laboratory Investigation

This section presents the experimental procedure and laboratory tests that were used to investigate the effect of sand and on geotechnical properties of laterite. The tests was conducted for the samples and specimen are: sieve analysis test, specific gravity test, atterberg limit test, compaction test and California bearing ratio test the above listed tests were carried out at Nnamdi Azikiwe University Civil Engineering Laboratory located inside the school campus. Below is a description of test procedures and apparatus:

3.3.1 Particle Size Distribution (Sieve Analysis)

Sieve analysis is a procedure used to assess the particle size distribution of a granular material (sand, gravel). The size distribution is often of critical importance to the behavior of the material during use. Sieve analysis can performed on any type of non-organic or organic granular material including sand, crushed rock, clay, granite, feldspar and a wide range of manufactured powders, grains and seed down to minimum size depending on the exact method. The standard grain size analysis test determines the relative proportion of different grain sizes as they are distributed among certain size ranges.

The grain size analysis is widely used in classification of soils. The data obtained from the grain distribution curve is used in the design of filters for earth dams and to determine the suitability of soil for road construction, air field etc. Information obtained from grain size analysis can be used to predict soil water movement although permeability test are more generally used. Soil gradation is very important to geotechnical engineering; it is an indication of other engineering properties such as shear strength, compressibility and hydraulic conductivity. In a design, the gradation of the in-situ- soil help in the selection of filler material for the construction of highway embankment and it also controls the design and ground water drainage of site. A poorly graded soil (one with predominantly one-sized particle) will have better drainage property than the well graded soil (soil with varieties of particle sizes) because of the relatively higher magnitude of void present. A well graded can be easily compacted more than a poorly graded soil. However most Engineering project may have gradation requirement that must be satisfied before the soil is to be used is accepted for construction work. When options for ground remediation technique are to be considered the soil gradation is a controlling factor.

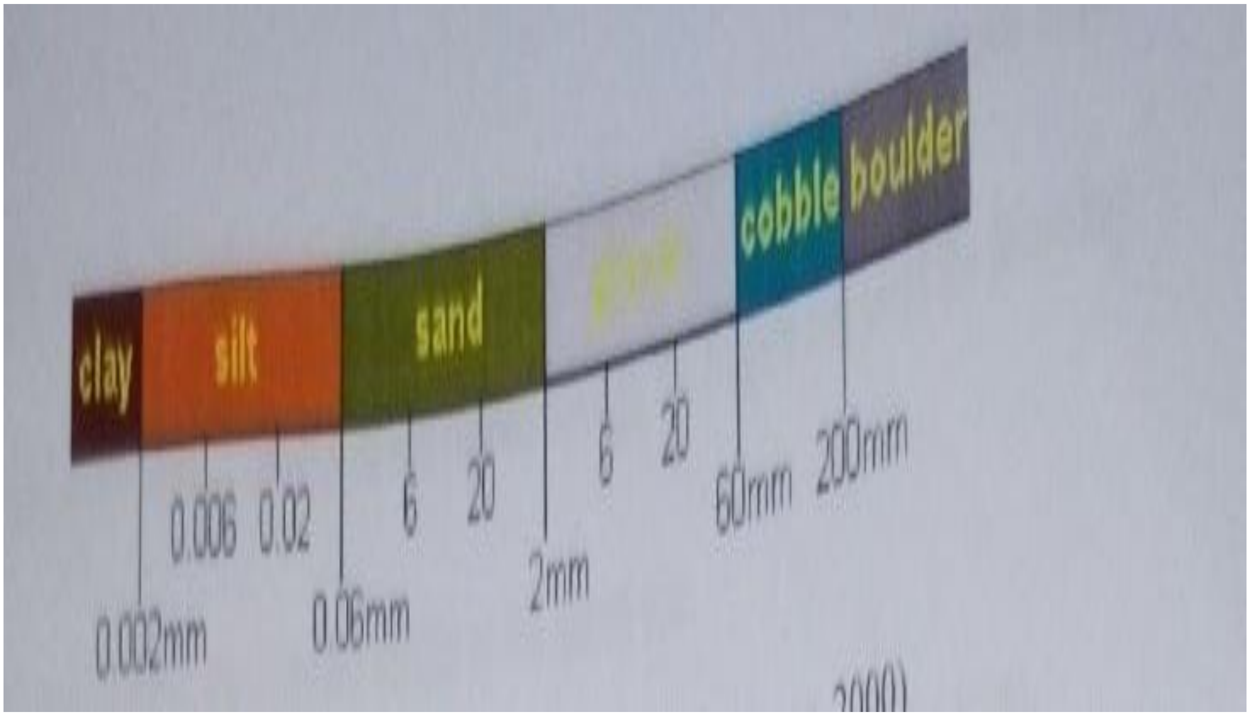


Plate 3.1 Ranges for grain Sizes of different Soil type Atkinson, (2000).

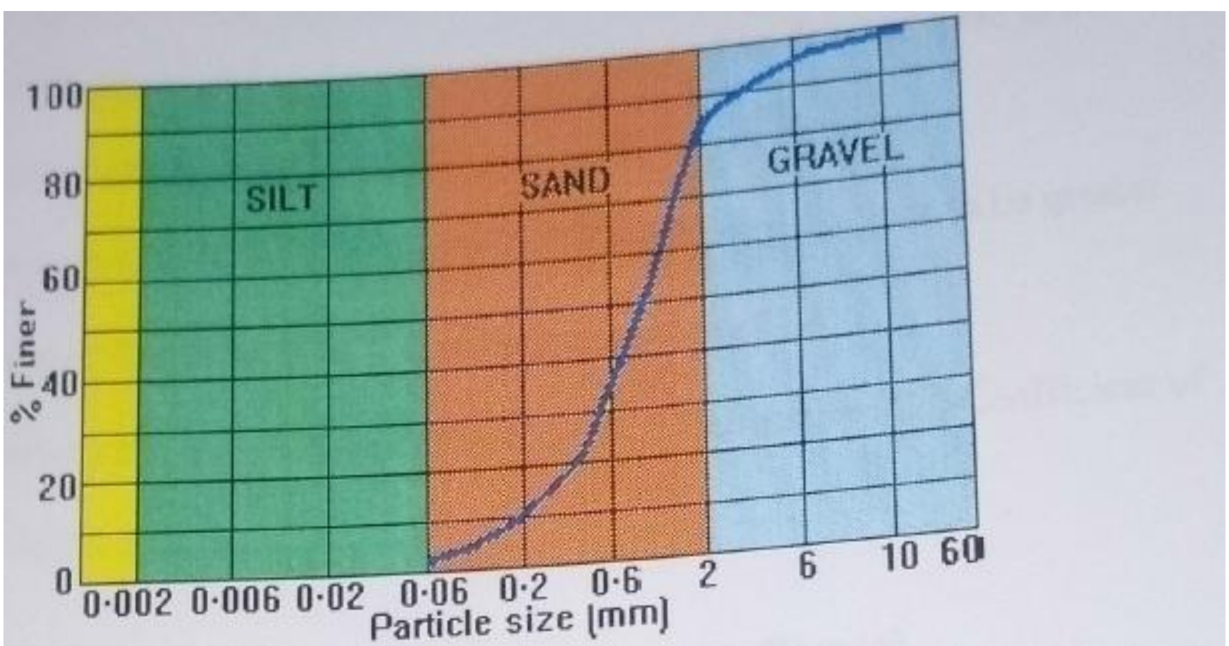


Plate 3.2 Grading Curve Ranges for Different Soil Types Atkinson, (2000)

Soil posses a number of physical characteristics which can be used as aid to identify it sizes in the field. A handful of soil rubbed through the finger can yield the following:

1. Sand and other coarser particle are visible to the naked eye.
2. Silt particle becomes dusty and are easily brushed off.
3. Clay particle are greasy and sticky when wet and hard when dry and have to be scrapped or washed off hand and boot

For a soil to be well graded the value of coefficient of uniformity (Cu) has to be greater than 4 and 6 for gravel and sand respectively, while the Coefficient of Curvature (C_v) should be in the range of 1 to 3.

The apparatus needed for this experiment is listed below:

1. Stack of sieves including pan and cover.
2. Mechanical sieve shaker.
3. Weighing balance of 0.01g sensitivity.
4. Hand brush
5. Mortar and pestle (Used for crushing if the sample is conglomerated or lumped)
6. Thermostatically controlled Oven (With temperature of about $80^{O}C$-$110^{O}C$).
7. Masking tape for identification of sample.
8. Exercise book and pen for recording of result.
9. The calculation for attaining Coefficient of uniformity and Coefficient of curvature are outlined below.

Percentage retained (%) $=\frac{\text{mass of soil retained in the sieve(g)}}{\text{total mass of soil sample(g)}} \times 100$

Cumulative percentage retained = $\sum \text{Percentage retained (\%)}$

Cumulative Percentage Finer (%) = 100-Cummulative percentage retained.

Coefficient of Curvature $=\frac{D60}{D10}$

Coefficient of Uniformity $=\frac{(D30)2}{D10\times D60}$

Where.

D10 = particle size such that 10% of the soil is finer than the size.

D30 = particle size such that 30% of the soil is finer than the size.

D60 = particle size such that 60% of the soil is finer than the size.

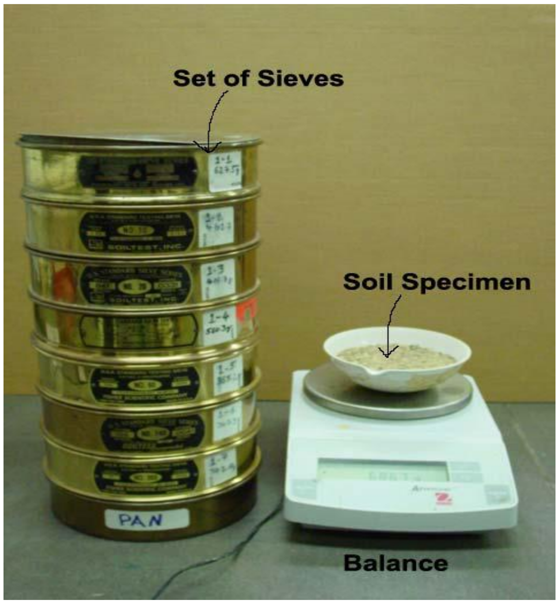


Plate 3.2: Apparatus for Particle Size Distribution Test (Sieve Analysis).

Plate 3.3: Apparatus for Particle Size Distribution Test (Sieve Analysis).

Test Procedure

The stack of sieves to be used for the experiment was properly cleaned using hand brush.

1. About 500g of air-dried soil sample was weighed with the aid of a weighing balance.

2. The weighed soil sample was poured into 75um sieve and wash under a steady supply of water until clear water start coming out from the sieve after passing through the soil sample.
3. After washing pour the washed soil sample into a pre-weighed plate and dry it inside the thermostatically controlled oven at a controlled temperature of 80-110^{O}C for 16-24hrs.
4. The sample was removed from the oven and the weight was determine (net weight) by deducting the weight of plate from the weight of plate and soil.
5. The stacks of sieve was arranged in the ascending order, placed in a mechanical sieve shaker, and thereafter the sample was poured and connected to the shaker for about 10-15 minute.
6. The sieve shaker was disconnected and the mass retained on each of the sieve sizes was determined.
7. The percentage retained, Cumulative percentage retained and Cumulative percentage finer was determined.
8. The graph of sieve Cumulative percentage finer against sieve sizes was plotted.
9. D10, D30 and D60 were determined from the plotted graph.
10. The Coefficient of Curvature and Coefficient of Uniformity was determined and used to classify the soil adopting the American Association of State Highway and Transportation Official (AASHTO) and Unified Soil Classification System (USCS) respectively.

3.3.2 Specific Gravity Test

Specific gravity is the ratio of mass of unit volume of soil at a stated temperature to mass of equal volume of gas-free distilled water at the same temperature Krishna, (2002). Also as defined by Braja, (2006), Specific gravity can be defined as the ratio of unit weight of a material to unit weight of water. The specific gravity of soil solids is often needed for various calculations in soil mechanics. It can be determined accurately in the soil laboratory.

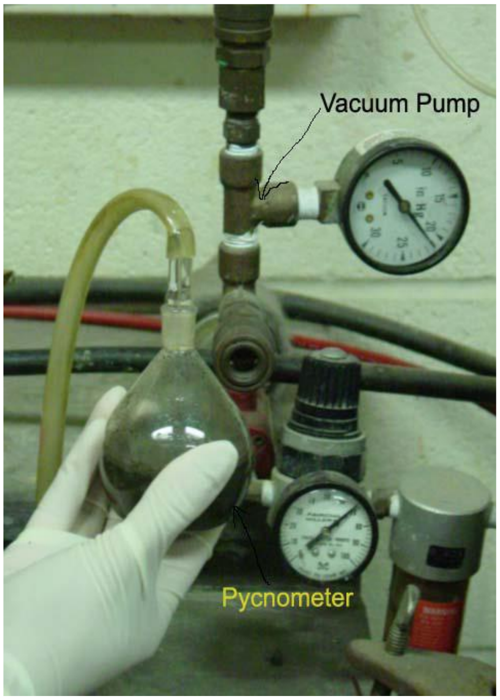
Apparatus employed for this experiment are:

1. Density bottle of 50ml capacity and a stopper.
2. Desiccator containing anhydrous silica gel.
3. Thermostatically controlled oven with temperature of about 80-110^{O}C.
4. Weighing balance of 0.01g sensitivity.

5. Mantle heater.
6. Plastic wash bottle.
7. Distilled water.
8. Funnel

Thin glass rod for stirring.

9. 425um Sieve.
10. Dry piece of cloth for cleaning.
11. Masking tape for identification of sample.
12. Exercise book and pen for recording of result.



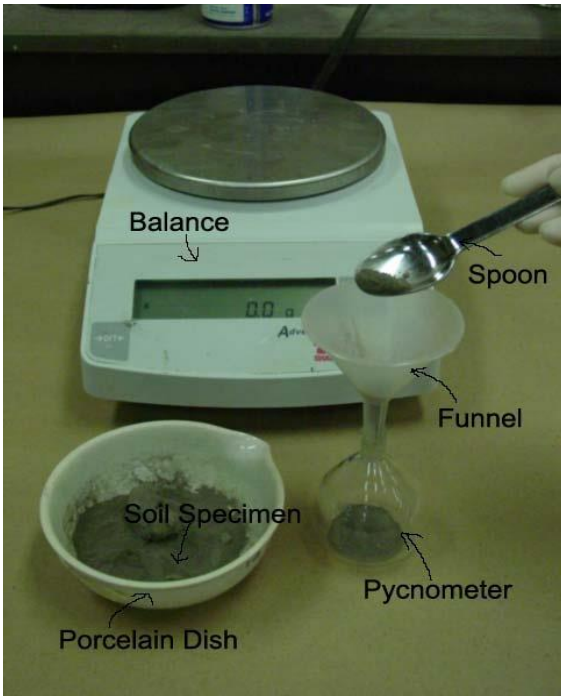


Plate 3.4: Apparatus used for Specific Gravity Test.

Test Procedure

1. The density bottle properly cleaned and rinsed with distilled water, thereafter oven- dried and then cooled it in a desiccator so as to remove any moisture present.
2. The empty clean and dry density bottle was weighed and recorded as (M_1).

3. About 10-15g of soil passing through 425um sieve was placed inside the density bottle, weigh and the weight of density bottle +dry soil + stopper was recorded as (M_2).
4. Distilled water was added to fill about half to three-fourth of the density bottle, and then the sample was soaked for 24hrs (The time stated is to enable complete settlement of the soil particle which is evident when clear water appears above the submerged soil).
5. The density bottle was gently stirred using thin glass rod and thereafter connected to a mantle heater to de-air the sample, the sample was not allowed to boil over.
6. After agitation, the sample was allowed to cool at room temperature and then filled with distilled water up to the specified mark (at lower meniscus level), the exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle + stopper +soil filled with water was determined and recorded as (M_3).
7. The density bottle was emptied, cleaned and rinsed with distilled water, then filled with distilled water up to the same mark. The exterior surface of the density bottle was cleaned with a clean dry cloth and the weight of the density bottle filled with distilled water + stopper was determined and recorded as (M_4).
8. The test procedure was repeated for two more trials and the average specific gravity value was obtained from the total no of trial, the variation in the specific gravity result obtained for each trial must not exceed 2%, otherwise repeat the experiment.

The Procedure for Computation of result obtained are as follows:

Specific gravity (G_S) = $\frac{(M2-M1)}{(M2-M1)-(M3-M4)}$

Where M_1= weight of density bottle + stopper

M_2= Weight of density bottle + air-dried soil + stopper.

3.3.3 Atterberg Limit Test

The behavior of soils especially fine grained soils differs considerably in the presence of water. Clay in the presence of water may almost take a liquid or can be quite hard. Consistency is the property of soil that offers resistance to deformation, it denote the degree of firmness of a soil and can be explained in terms of plasticity and stickiness of soil. Stickiness is the ability of soil especially fine grained soil to adhere to other materials while plasticity on the other hand is the ability of soils to undergo a change in shape under the action of an impressed force without a change in volume.

Stickiness of soils especially fine grained soils can be identified practically by mixing of an air-dried soil with a given quantity of water and then interposing the soil between the thumb and the fore finger (index finger), thereafter the following inference are made as it regards to the observation and this includes:

1. **Non-Sticky:** If the wet soil falls freely between the thumb and the forefinger without leaving any remain or without stretching.
2. **Slightly Sticky:** If the wet soil falls slowly with an infinitesimal traces of remains but without stretching.
3. **Sticky:** If the wet soil falls quite slowly with visible remains and apparent stretching.
4. **Very Sticky:** If the wet soil stretches between the thumb and the fore finger without falling.

The plasticity of soils can be identified practically by rolling a known weight of wet soil into a 3mm uniform diameter thread and the following inferences based on the observation are made and they are as follows:

1. **Non-Plastic:** If the wet soil cannot be rolled into thread.
2. **Slightly Plastic:** If the wet soil can be rolled into thread but crumbles easily under application of little pressure.
3. **Plastic:** If the wet soil can be rolled into 3mm thread but crumbles under intense application of pressure and cannot be reformed.
4. **Very Plastic:** If the wet soil can be rolled into 3mm diameter thread but crumbles under intense application of pressure and can be reformed.

The atterberg limit is a limit characterized by visible transition of soil (especially fine grained soils) from liquid-plastic-semi-solid-solid state consequent upon the variation of moisture content. This test was developed by Albert Atterberg a Swedish agricultural scientist in 1911. This test is divided into three limits namely:

1. Liquid Limit (LL)

2. Plastic Limit (PL)
3. Shrinkage Limit (SL)

3.3.3.1 Liquid Limit Test

It is the water content at which the soil has a small shear strength that it flows to close a groove of standard width when jarred in a specified manner. It is the minimum water content at which the soil tends to flow like a liquid. When a soil is mixed with an excessive amount of water, it will be in a liquid state and flow like a viscous liquid. When the viscous liquid dries gradually due to loss of moisture it will pass into a plastic state. With further loss of moisture, the soil will pass into a semi-solid state. With even further reduction of moisture, the soil will pass into a solid state. The moisture content (%) at which a cohesive soil will pass from liquid state to plastic state is referred to as the liquid limit of the soil.

In order to study the liquid limit of the soil Casagrande test was conducted. liquid limit is generally determined by the mechanical method using Casagrande apparatus or the standard liquid limit test apparatus. With respect to this method, the liquid limit is defined as the moisture content at which 25 blows or drop in standard liquid limit apparatus will just close a groove of standardized dimension cut into sample by a grooving tool at a specified amount (Aroja, et al 2017).

The apparatus used for liquid limit determination is outlined below:

1. Liquid limit device (Cassagrande type)
2. Grooving tool
3. Moisture content tins
4. Porcelain evaporating dish
5. Spatula or pellet knife
6. Thermostatically controlled oven
7. Weighing balance sensitive to 0.01g
8. Plastic wash bottle containing distilled water
9. Paper towels
10. Masking tape for identification of tin.
11. Exercise book and pen for recording of data
12. 425um Sieve

13. Airtight container

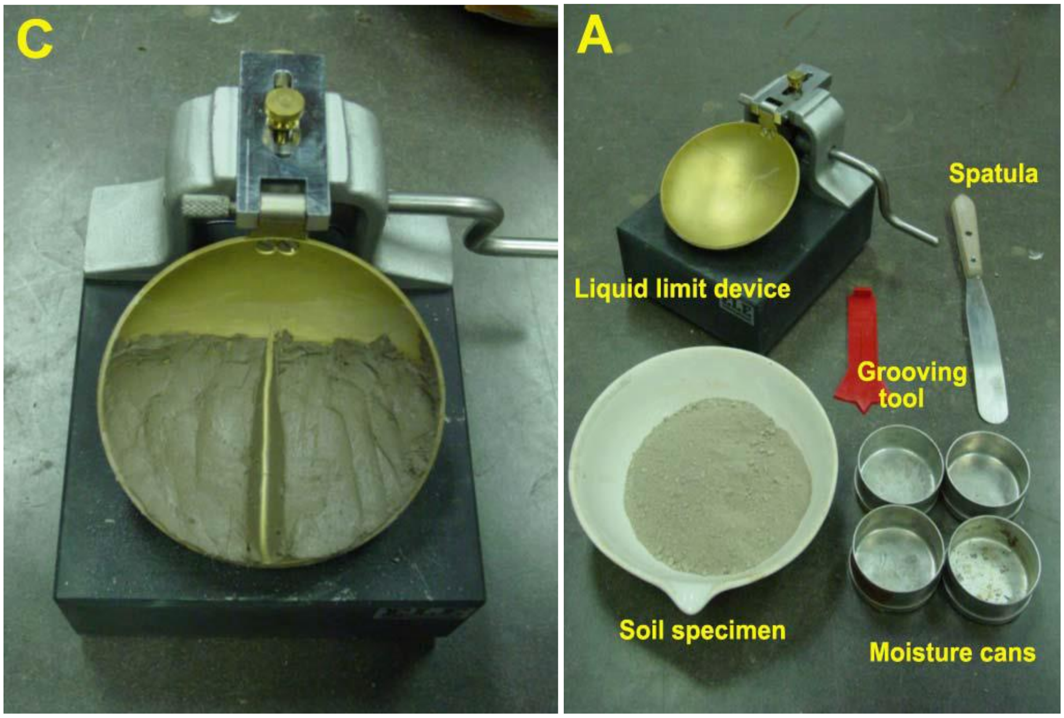


Plate 3.5: Apparatus for Atterberg Limit Test.

Test Procedure

1. The sample was prepared by weighing about 150g of soil and passing it through 425um sieve, the sample was mixed with distilled water in a glass plate with the aid of pellet knife, during the mixing operation, coarse particle was removed by hand and mixed the sample was mixed to form a thick homogenous paste, thereafter, the mixed soil was placed in an airtight container and leave to mature for 24hrs.
2. The mass of four moisture content tins was determined and recorded as (W_1)
3. The matured sample was placed on an evaporating dish with little water added to it using the plastic squeeze bottle; the soil was properly mixed to ensure uniform distribution of moisture.
4. A portion of the paste (mixed soil) was placed on the liquid limit device and then the mixture was leveled so as to obtain a maximum depth of 1cm.
5. The grooving tool was used to cut a groove along the symmetrical axis of the cup holding the tool perpendicular to the cup.

6. The handle of the crank of the liquid limit device was rotated at the rate of 2 revolution per second and the no of blows required to close the groove at a distance of 13mm was counted. Closing of the groove should be as a result of plastic flow of the soil and not by sliding, if sliding occurs repeat the test.
7. About 10g of soil in the closed groove was taken and placed in the moisture content tins for moisture content determination, the sample was weighed and recorded as (W_2)
8. The rest of the soil in the cup was removed and paper towel was used to clean the cassagrande cup properly.
9. The water content of the soil was altered and the process was repeated to obtain the required no of blows in the range of 15-40 blows.
10. The graph of moisture content against the log of no of blows was plotted and the moisture content corresponding to 25 blows on the abscissa gives the value of the liquid limit.

The Procedure employed for the Computation of the Result obtained is as Follows:

Moisture content = $\frac{Weight\ of\ water}{weight\ of\ dry\ soil} \times 100 = \frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tin.

W_2 = Weight of tin + wet soil.

W_3 = Weight of tin + oven-dried soil.

3.3.3.2 Plastic Limit Test

The plastic limit of a soil is the moisture content expressed as a percentage of the weight of oven-dried soil at the boundary between the plastic and the semi-solid state of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a uniform 3mm diameter thread using a glass plate or other recommended surface for rolling. Soil used for Atterberg limit test can be classified based on the plasticity index of the soil. The plasticity index is the amount of water required to change a soil from its plastic limit to liquid limit, in other word it is the numerical difference between the liquid limit and the plastic limit of soil. Table 3.2 is used to classify soil based on the ranges of it plasticity index.

Table: 3.1 Plasticity Ratings for Fine grained Soil Braja, (2002).

Plasticity Index	Plasticity
0	Non-Plasticity
<7	Low Plasticity
7-17	Medium Plasticity
17-35	High Plasticity
>35	Very High Plasticity

1. The apparatus used for this experiment includes:
2. A smooth glass plate about 300mm square and 10mm thick.
3. A palette knife or spatula
4. A short length of 3mm metal rod
5. Moisture content tins
6. Plastic squeeze bottle
7. Weighing balance with 0.01g sensitivity
8. Veneer caliper
9. Masking tape for tin identification
10. Exercise book and pen for recording of result.

Test Procedure

1. The sample was prepared by the method described in the liquid limit using the sample passing 425um sieve.
2. The empty moisture content tins was identified, weighed and recorded as (W1).
3. About 20g of the prepared soil paste was placed on a porcelain evaporating dish and water was added using the plastic squeeze bottle, the soil was mix thoroughly until the paste is plastic enough to be rolled into a ball.
4. A portion of the ball was taken and rolled on a glass plate with the palm of the hand into a thread of uniform diameter throughout its length by rolling forward and backward.
5. The rolling and remolding continued until the thread just start to crack at a distance of 3mm.

6. The small crumbed pieces was collected and placed in a moisture content tin a weighed and recorded as (W2).
7. The tin was placed in the oven at a constant temperature of 80-110^{O}C for a period of 16-24hrs.
8. After 24hrs, the tin was removed from the oven and the weight of the dry soil plus the tin was determined and recorded as (W_3).
9. The test procedure was repeated for at least two trials and takes the average plastic limit value for all the trials.

The Computation for Plastic Limit is as follows:

Plastic limit = $\frac{Weight\ of\ water}{Weight\ of\ oven-dried\ soil} \times 100 = \frac{W2-W3}{W3-W1} \times 100$

Where W_1 = Weight of empty tins.

W_2 = Weight of tin plus wet soil

W3 = Weight of tin plus oven-dried soil

3.3.4 Compaction Test

Compaction is the process of increasing the bulk density of the soil by driving out air. It involves the densification of soils by mechanical means thereby increasing the dry density of the soil. According to Shruthi, (2017) Compaction of soil is the process by which the soil solid are packed more closely together by mechanical means, thus increasing it dry density. It could also be stated as the process of packing the soil particle more closely together usually by tamping, rolling or other mechanical means, thus increasing the dry density of the soil. It is achieved through the reduction of the volume of air void in the soil with little or no reduction in water content. The process must not be confused with consolidation in which water is squeezed out under the action of steady static load. Consolidation is a natural process and result in dense packing of the soil.

In civil engineering practice soil compaction is essential for the following reasons:

1. Increasing the bearing strength of foundation
2. Provide stability to slope and foundation.
3. Prevention of undesirable settlement of structures

4. Reduction of water seepage from structure

The compaction methods to be adopted for this research are British Standard Light for the natural laterite sample and laterite stabilized with sand and cement.

Details of British Standard Compaction Process

Table 3.1: Details of Compaction Mould.

Type	Diameter (mm)	Height (mm)	Volume(cm^3)
British Standard	105	115.5	1000

Table 3.2: Details of Compaction Procedure.

Type of test	Mould (cm^3)	Rammer(kg)	Drop (mm)	No of layers	Blow per layer
BS light	1000	2.5	300	3	27
BS heavy	1000	4.5	450	5	27

The mechanical energy applied in each type of British Standard in term of work done is given as follows:

British Standard Light

Mechanical energy = $\frac{Weight\ of\ rammer \times \text{no of layers} \times \text{no of blows} \times \text{height of drop}}{\text{Volume of mould}}$

$=\frac{2.5\text{g}\times 3\text{layers}\times 27\text{blows}\times 300\text{mm}}{1000}$ = 60.75kgm =60.75× 9.81Nm=596j

Work done per unit volume of soil $=\frac{596}{1000}$ =596kj/m^3

British Standard Heavy

Mechanical energy $=\frac{4.5\times 5\times 27\times 450}{1000}$ =2652j

1. The apparatus used for the test are as follows:

2. Compaction mould with a detachable base plate and removable extension collar.
3. Metal rammer (either 2.5kg or 4.5kg)
4. Measuring Cylinder 200ml or 500ml
5. Large Metal tray (600mm×600mm ×600mm)
6. Balance up to 10kg readable to 1g
7. Small tools such as palette knife, steel straight edge about 300mm long.
8. Drying oven temperature of 105-110^{O}C
9. Apparatus for moisture content determination

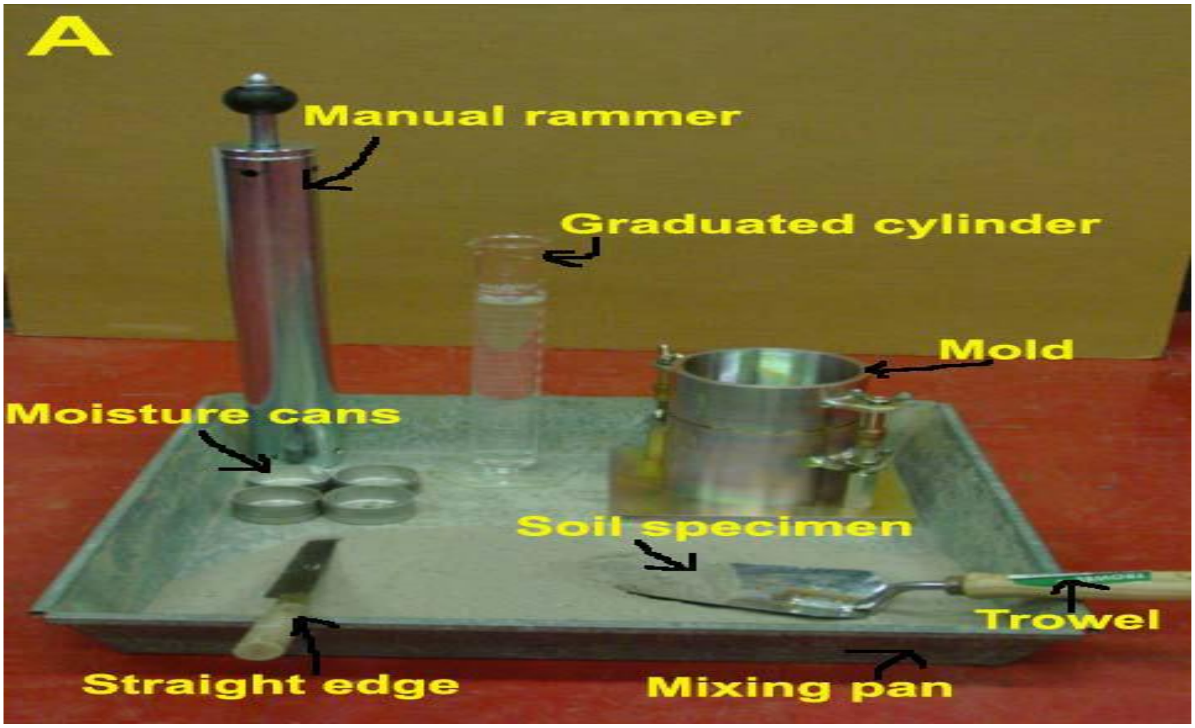


Plate 3.6: Apparatus employed for Compaction Test.

Test Procedure

1. The mould, extension collar and base plate was cleaned and dried. The dimension was measured and weigh to the nearest 1kg check if the rammer falls freely.
2. The internal surface of the mould was greased.
3. The extension collar was attached to the mould.
4. About 3kg of the soil sample was weighed on a weighing balance.

5. About 4% water was added to the soil sample, mixing it thoroughly and separating the soil into three layers for British Standard Light and five layers for British Standard Heavy.
6. The wet soil was poured into the mould and compacted thoroughly by applying the required no of blow using either a 2.5kg or 4.5kg rammer falling freely from a height of 300mm. The blow was distributed uniformly over the surface of the mould.
7. After completion of the compaction operation, the extension collar was removed and the top of the mould was carefully levelled by means of a straight edge.
8. The mould with the compacted soil to the nearest 1kg, was weighed and recorded as W_2.
9. The moisture content of the representative sample of the specimen was determined and recorded as M.
10. The procedure was repeated and 8%, 12%, 16% and 20% of water was added and the value obtained was recorded.
11. The graph of dry density against moisture content was plotted and the maximum dry density (MDD) of the soil at the corresponding optimum moisture content (OMC) was determined.

The Computation of the result obtained is as follows:

Determination of Dry Density (P_d).

Wt of mould (kg) = W_1

Wt of mould + wet soil (kg) = W_2

Wt of wet soil (kg) = W_2-W_1

Volume of mould (M^3) = W_4

Bulk Density (kg/m^3) = $\frac{\text{Wt of wet soil (kg)}}{\text{Vol of mould (m3)}} = \frac{W2-W1}{W4}$

Moisture Content (%) = $\frac{\text{mosture content (top)} + \text{moisture content (bottom)}}{2}$

Dry Density (kg/m^3) = $\frac{\text{Bulk density}}{1+\text{moisture content (\%)}} = \frac{Pb}{1+w/100}$

Determination of Moisture Content (w) for top and bottom respectively.

Wt of tin (kg) = W_1

Wt of tin + wet soil = W_2

Wet of wet soil (kg) = W_3 = W_2-W_1

Wt of tin + dry soil (kg) = W_4

Wt of dry soil (kg) = $W_5 = W_4 - W_1$

Wt of water (kg) = $W_6 = W_3 - W_5$

Moisture Content (%) = $\frac{\text{Wt of water}}{\text{Wt of dry soil}} \times 100 = \frac{W6}{W5} \times 100$

3.3.5 California Bearing Ratio Test

The California bearing ratio test was originally developed by the California division of highway in 1938, for the design of highway thickness. The test is used for evaluating the suitability of materials used in sub-grade, sub-base and base course respectively. The test result has been correlated with the thickness of various materials required for flexible pavement construction. The test may be conducted on a prepared specimen in a mould or on the soil in-situ condition.

In the test the load required to push a plunger into a soil specimen at a controlled rate is measured, then the load on the plunger at a certain depth is recorded as a percentage of a standardized load. The load necessary to push a plunger to a certain depth into the soil is expressed as a percentage of the load required to force the same plunger to the same depth into a standard sample of compacted crush stone. The construction of highway pavement requires a California Bearing Ratio value for 2.5mm and 5mm penetration respectively, with that of 2.5mm penetration being comparatively higher than that of 5mm penetration. The Federal Ministry of work Standard Specification for roads and bridges (1997) state that road construction material should have a CBR value of 10%, 20% and 80% for use as sub-grade, sub-base and base course respectively. The material to be used for the test will be subjected to 48 hours soaking in other to ascertain it behavior under worst condition (flooding as a result of intense rainfall).

Table 3.1 Standard load adopted for different penetration on a standard material with CBR value of 100%.

Penetration of plunger (mm)	Standard Load (kg)
2	1150
2.5	1320
4	1760
5	2000
6	2220
7.5	2630
8	2650
10	3180
12.5	3600

1. The apparatus used for the test are outlined below:
2. A cylindrical corrosion resistant mould 152mm×127mm having a diameter of 150-152mm with a detachable base plate and a removable extension collar.
3. A compressive device for static compaction of applying a force of at least 300KN
4. Metal plugs 150mm $\pm$ 0,5mm and 50mm thick.
5. Metal rammer 2.5kg or 4.5kg.
6. Dial gauge of 0.01g sensitivity.
7. Soaking tank.
8. A steel rod of about 16mm diameter and 600mm long and a straight edge of 300mm steel stripe and 3mm thick with one beveled edge.
9. Weighing balance of 25kg accuracy and a spatula.
10. Filter paper
11. Apparatus for moisture content determination.
12. Masking tape used for identification of moisture content tins.
13. Exercise book and pen for recording.

Plate 3.8 California Bearing Ratio (CBR) Test Machine.

Test Procedure

The methods used for California Bearing Ratio Test are:

1. Compression with tamping.
2. Recompaction with known maximum dry unit weight (MDUW) and optimum moisture content (OMC).
3. For this course of study the method for recompacted sample with known maximum dry unit weight (MDUW) and optimum moisture content (OMC) is to be adopted and the procedure is outlined below:
4. Carry out Compaction test using 6kg of soil sample, varying the moisture content at a particular percentage say 4%, determine the maximum dry density and optimum moisture content.

5. Clean properly and grease the internal surface of the CBR mould.
6. Weigh 6kg of soil mixing with the optimum moisture content determined from compaction test.
7. Divide the soil into 5 equal layer (CBR Heavy) and seal in an airtight container until requested for use.
8. Stand the mould assembly in a solid base, place the first soil portion and compact using 4.5kg rammer for 62 even blows.
9. Repeat using the remaining four portion of soil in turn so that the level of the soil is not more than 6mm above the top of the mould body.
10. Remove the collar and trim the soil flush with mould with the scrapper or knife edge.
11. Weigh the mould, soil and base plate to the nearest kg.

Preparation for Soaking

Soil may soften when load is placed on it due to flooding or increase in moisture content. Soaking of the sample is done primarily to determine the strength (load bearing strength) of the soil under worst condition (rainy season).Below are the list of apparatus used for CBR Soaking:

1. Perforated base plate fitted to CBR mould in place of normal base plate.
2. Perforated swell plate with an adjustable stem to provide a sealing for the stem of the dial gauge.
3. Tripod mounting to support dial gauge
4. Soaking tank
5. Annular Surcharge discs with internal diameter of 52-54mm and external diameter of 145mm to 150mm.
6. Petroleum jelly.
7. The Soaking procedures are enumerated as follows:
8. Remove the base plate and replace with perforated base plate.
9. Fit the collar to the other end of the mould, pack the screw thread with petroleum jelly to make it water tight.
10. Place the mould assembly in soaking, place the filter paper in the sample, the perforated swell plate, and then annular surcharge disc.

11. Mount dial gauge on top of the extension collar, secure the dial gauge in place and adjust the stem in the perforated base plate to give zero.
12. Fill the immersion tank with water just below the extension collar. Start the timer when water has just covered the base plate.
13. Record the time taken for water to appear at the top of the sample if it does occur within two days. Flood the top of the sample and leave to soak for a day.
14. Plot the swelling against elapsed time or square root of time. Flattening curve indicates that swelling is complete.
15. Take off the dial gauge and its support; remove the mould assembly and leave to drain for 15min.
16. Remove the Surcharge discs, perforated plate and collar, then fit the other base plate.
17. Weigh the sample + mould + base plate if density is required after soaking is completed.
18. If the sample has swollen, trim it to the level of the mould and reweigh
19. Test the sample by adjusting the dial gauge to start at zero and take the reading at interval of 0.5mm for every 30seconds till 7mm penetration.
20. Record the load at penetration of 0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5 and 7.0mm and express this force as percentage of the standard load.
21. Calculate the CBR for 2.5 and 5mm penetration; repeat the same procedure for top and bottom, the higher CBR value will be used as the CBR for the material.
22. Plot the graph of force (KN) against penetration (mm).
23. The normal curve is convex upward, but if the initial part is concave upward applies the necessary correction to the curve.

Mathematically it is expressed as $\frac{\textbf{test load}}{\textbf{standard load}} \times \mathbf{100}$

Where

Test load = dial gauge reading × proof ring constant

CHAPTER FOUR

RESULTS AND DISCUSSION

During the course of the experimentation phase of the study, certain results were obtained which was valuable in evaluation the effect of cement and sand on strength properties of laterite. These results are presented in Table 4.0 below:

4.1 Results

Table 4.0: Index Properties of Laterite Stabilized with Cement and Sand

Percentage Replacement/ Properties	LAT + 0%C + 0%SD	LAT + 4%C + 8%SD	LAT + 8%C + 16%SD	LAT + 12%C + 24%SD	LAT + 16%C + 32%SD	LAT + 20%C + 40%SD
Specific Gravity	2.65	2.70	2.71	2.74	2.76	2.72
Liquid Limit (%)	36.8	32.4	26.6	–	–	–
Plastic Limit (%)	20.46	17.57	15.83	–	–	–
Plasticity Index (%)	16.34	14.83	10.77	Non-plastic	Non-plastic	Non-plastic
Percentages Passing Through Sieve No 200 (0.075mm)	22.26	–	–	–	–	–
AASHTO Classification System	A-2-6	–	–	–	–	–
Unified Soil Classification System	SC	–	–	–	–	–

Table 4.1: Compaction Characteristics of Laterite Stabilized with Cement

Percentage Replacement/Properties	LAT + 0%C	LAT + 4%C	LAT + 8%C	LAT + 12%C	LAT + 16%C	LAT + 20%C
Maximum Dry Unit Weight (kN/m^3)	16.96	17.2	19.6	18.7	19.8	18.8
Optimum Moisture Content (%)	12.78	17.78	12.9	15.07	11.8	15.1
Soaked CBR Values (%)	10.61	13.6	15.2	16.7	19.7	17.4

Table 4.2: Compaction and CBR Characteristics of Laterite Stabilized with Sand

Percentage Replacement/Properties	LAT + 0%SD	LAT + 8%SD	LAT + 16%SD	LAT + 24%SD	LAT + 32%SD	LAT + 40%SD
Maximum Dry Unit Weight (kN/m^3)	16.96	18.53	18.9	19.5	19.6	20
Optimum Moisture Content (%)	12.78	12.78	14	13.3	12.7	12.6
Soaked CBR Values (%)	10.61	18.9	21.2	26.7	29.5	31.1

Table 4.3: Compaction and CBR Characteristics of Laterite Stabilized with Cement and Sand

Percentage Replacement/Properties	LAT + 0%C + 0%SD	LAT + 4%C + 8%SD	LAT + 8%C + 16%SD	LAT + 12%C + 24%SD	LAT + 16%C + 32%SD	LAT + 20%C + 40%SD
Maximum Dry Unit Weight (kN/m^3)	16.96	19.2	20.9	21.5	20.1	20.3

Optimum Moisture Content (%)	**12.78**	**12.78**	**14**	**13.3**	**12.7**	**12.6**
Soaked CBR Values (%)	**10.61**	**15.9**	**18.2**	**26.5**	**31.1**	**34.1**

4.2 Discussion on Findings

4.2.1 Sieve Analysis Test

Figure 4.0 is a semi-logarithmic plot of the particle size distribution of laterite and sand. The percentage passing through sieve No 200 (0.075mm) for sand is 22.26 and as a result, the laterite is classified as A-2-6 according to AASHTO Soil Classification System and SM (sand mixed with silt) according to Unified Soil Classification System. The percentage passing through sieve No 200 for sand is 8.03; the coefficient of uniformity and coefficient of curvature were 5.4 and 3.4 respectively. The sand sample was classified A-2-4 according to AASHTO Soil Classification System and SC (sand mixed with clay) according to Unified Soil Classification System. Gradation assessment of sand revealed that the sand samples were poorly graded.

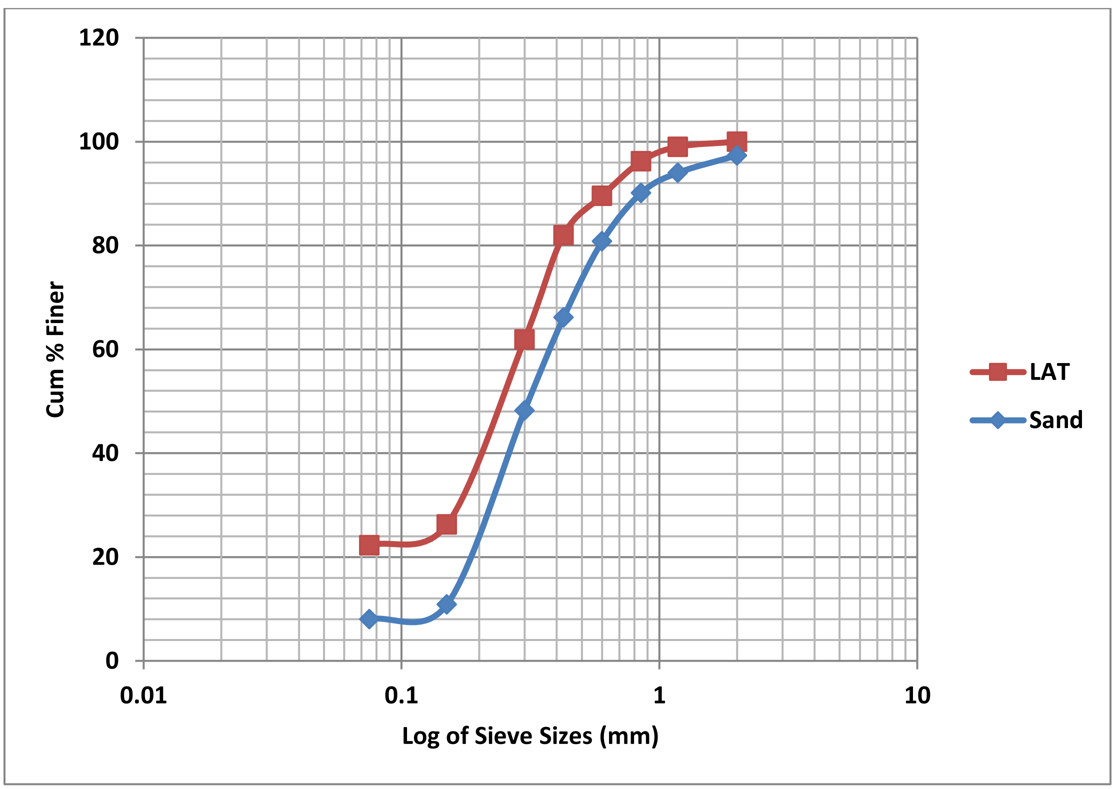


Figure 4.0: Particle Size Distribution Curve for Laterite and Sand Samples

4.2.2 Specific Gravity

The specific gravity results for laterite stabilized with cement and sand are shown in Table 4.0. Results obtained revealed that the specific gravity of laterite increased from 2.65 to 23.76 on addition of cement and sand up to 16 and 32% respectively. The increase in specific gravity of laterite could be attributed to the high specific gravity of sand. Sand has high unit weight than laterite and addition of sand to laterite raises the specific gravity of laterite.

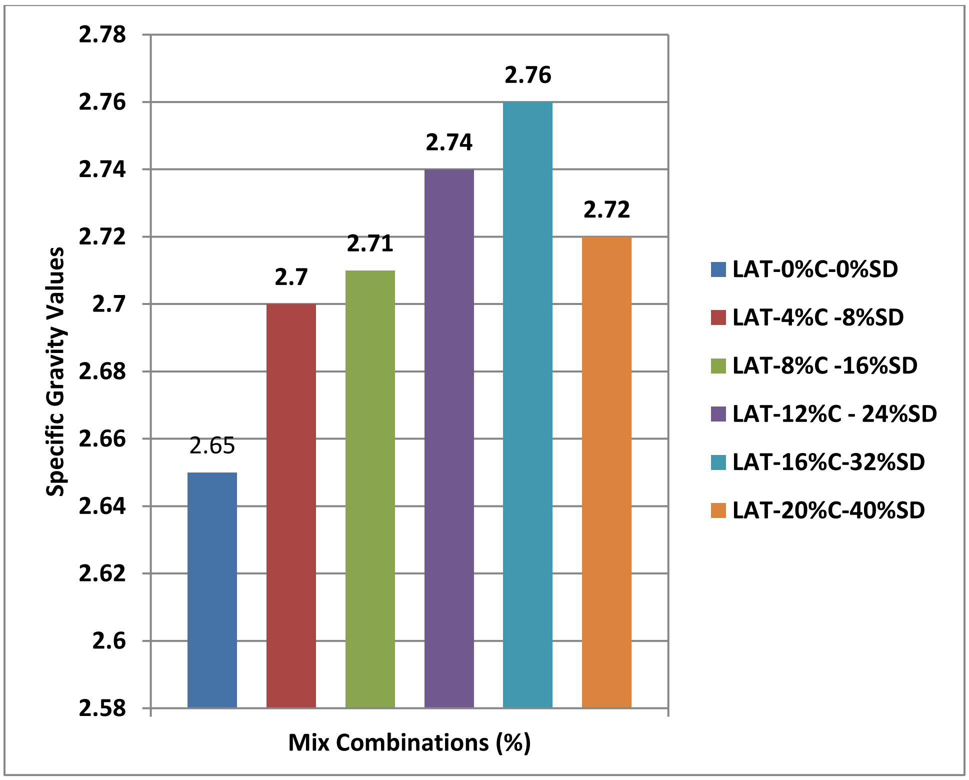


Figure 4.1: Chart Showing the Specific Gravity Value of Laterite Stabilized with Cement and Sand

4.2.3 Atterberg Limit

Figure 4.2-4.4 depicts the liquid, plastic and plasticity index of laterite stabilized with sand and cement at varying percentages. It was observed that the liquid limit, plastic limit and plasticity index of laterite decreased on addition of cement and sand to laterite. It was also observed that beyond 8% cement and 16% sand, the mixture became non-plastic. The decline in liquid limt, plastic limit and plasticity index of laterite could be attributed to the non-plastic nature of sand.

Sand is a coarse grained soil and addition of sand depresses the liquid, plastic and plasticity index of the mixture.

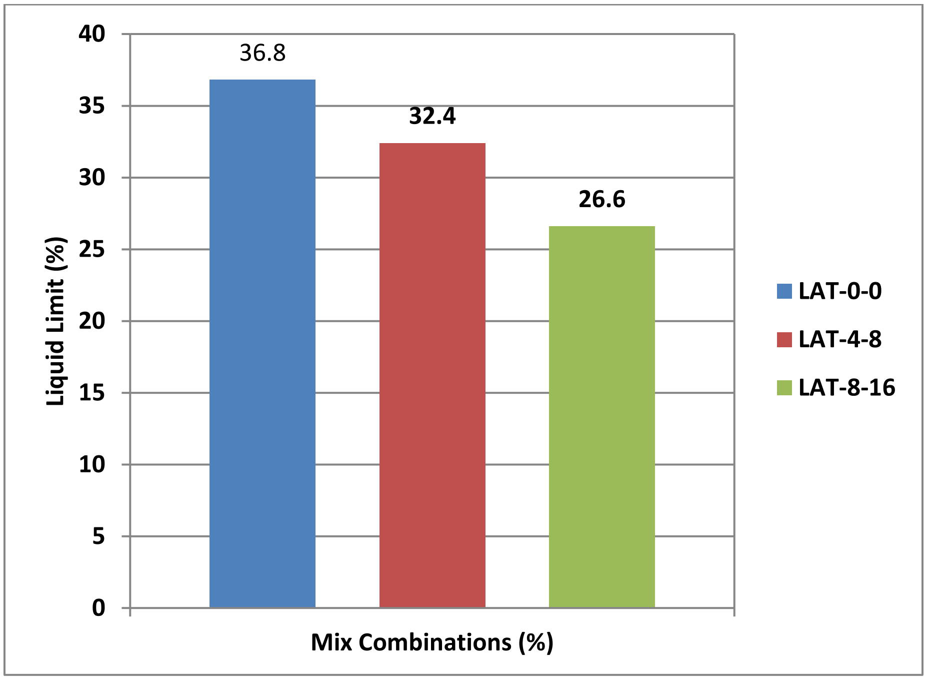


Figure 4.2: Chart Showing the Liquid Limit Value of Laterite Stabilized with Cement and Sand

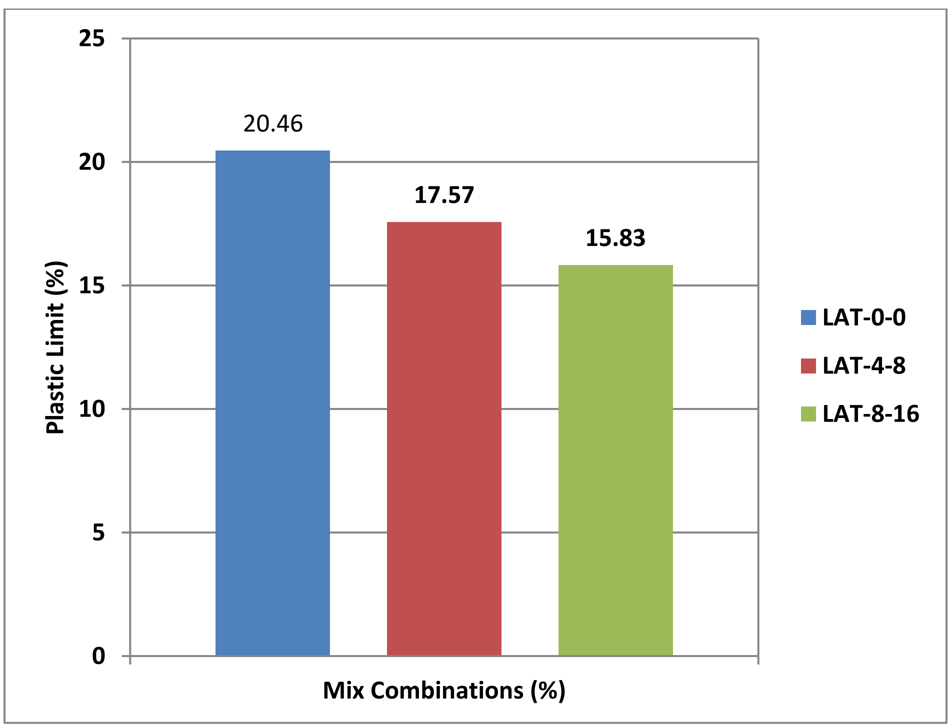


Figure 4.3: Chart Showing the Plastic Limit Value of Laterite Stabilized with Cement and Sand

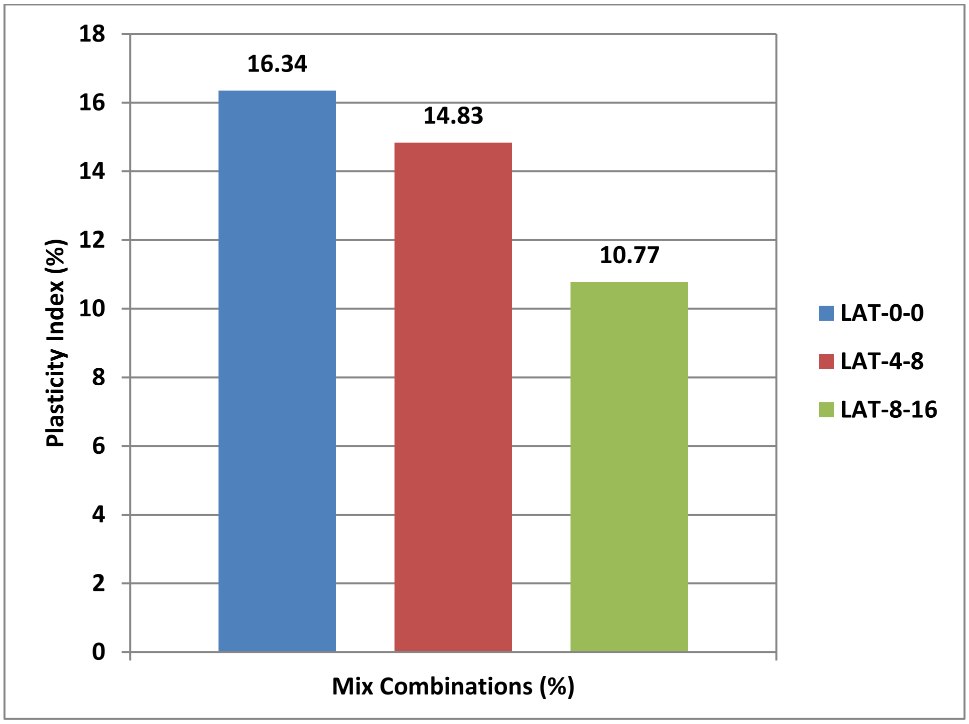


Figure 4.4: Chart Showing the Plasticity Index Value of Laterite Stabilized with Cement and Sand

4.2.4 Compaction Test

Table 4.1 to 4.3 shows the compaction characteristics of laterite stabilized with cement, sand and a blend of cement and sand at varying percentages. It was observed that on addition of cement to laterite from 4% to 20%, the maximum dry unit weight of laterite increased from its natural value of 16.96kN/m^3 to 19.8kN/m^3 from 4% cement content to 16% cement content, beyond 16% cement content, the maximum dry unit weight of the laterite decreased. Although a slight deviation was observed at 12% cement content, this could be attributed to lapses in the experimentation process. For a combination of laterite and sand, the maximum dry unit weight of laterite increased on consistent addition of sand to laterite. This result implies that sand is more effective in enhancing the compaction characteristics of laterite than cement. While for a blend of cement and sand added to the natural laterite soil, the maximum dry unit weight of laterite was observed to increase from its natural value of 16.96kN/m^3 to 21.5kN/m^3 at 12% cement and 24% sand content beyond this point, the maximum dry unit weight of laterite decreased. The latter decrease could be attributed to the high content of cement present in the mixture. It was also observed that the optimum moisture contents of the different mixtures generally decreased with increase in maximum dry unit weight. This agrees with Proctor (1933),Venkatramaiah (2006), Rowe (2000) and other concluded research works.

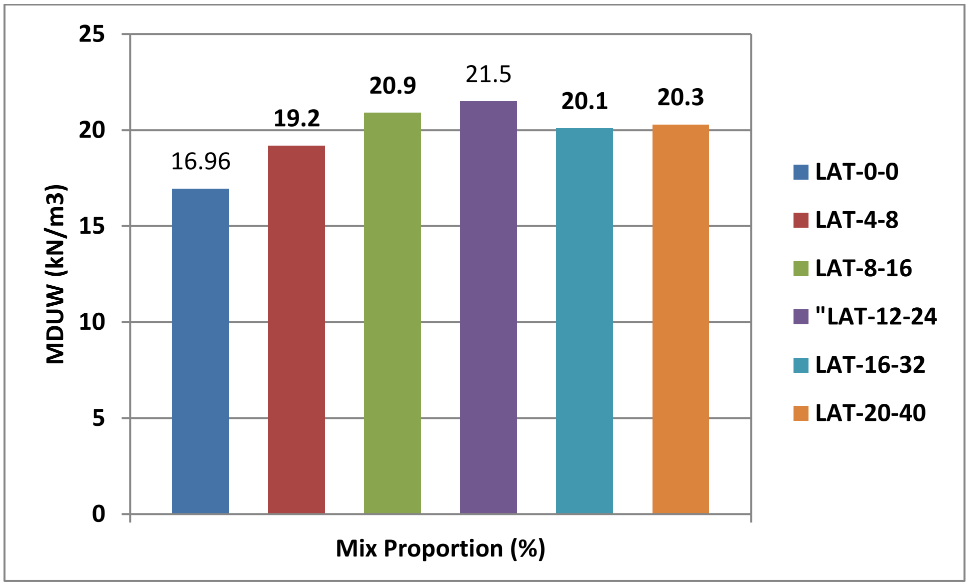


Figure 4.5: Maximum Dry Unit Weight of Laterite Stabilized with a Blend of Cement and Sand.

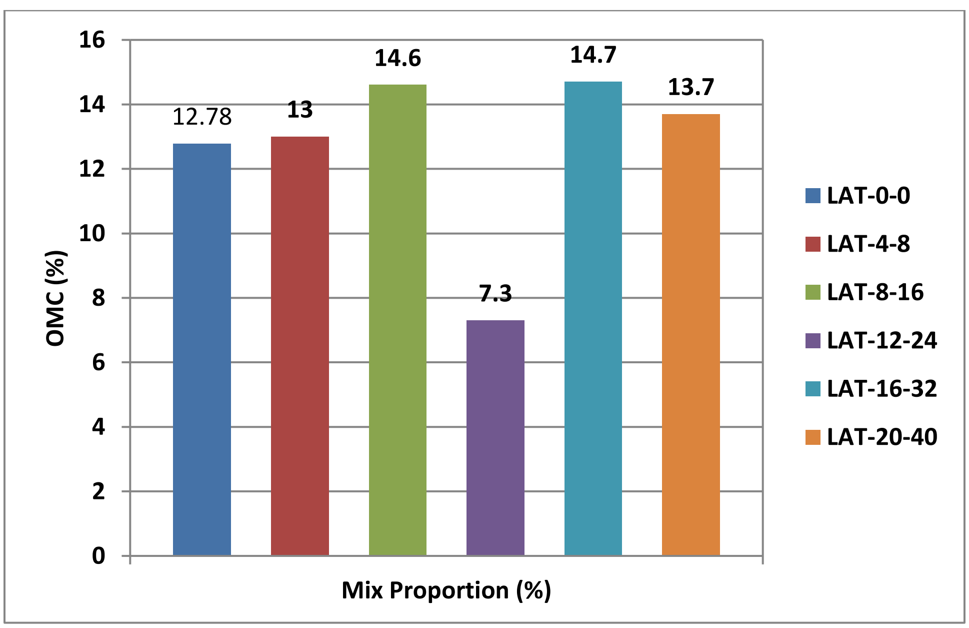


Figure 4.6: Optimum Moisture Content of Laterite Stabilized with a Blend of Cement and Sand.

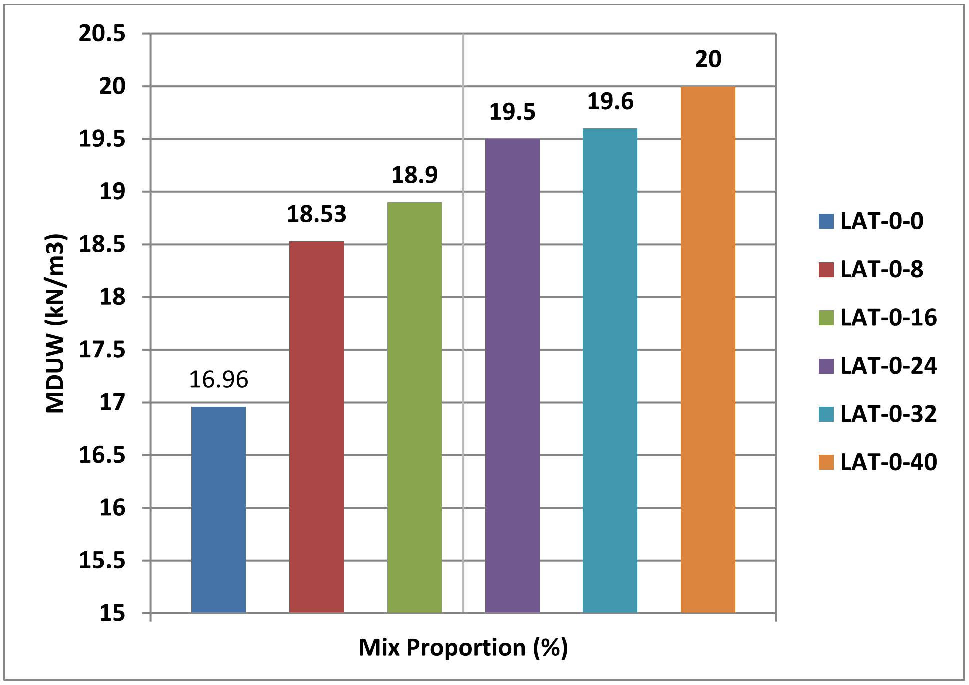


Figure 4.7: Maximum Dry Unit Weight of Laterite Stabilized with Cement

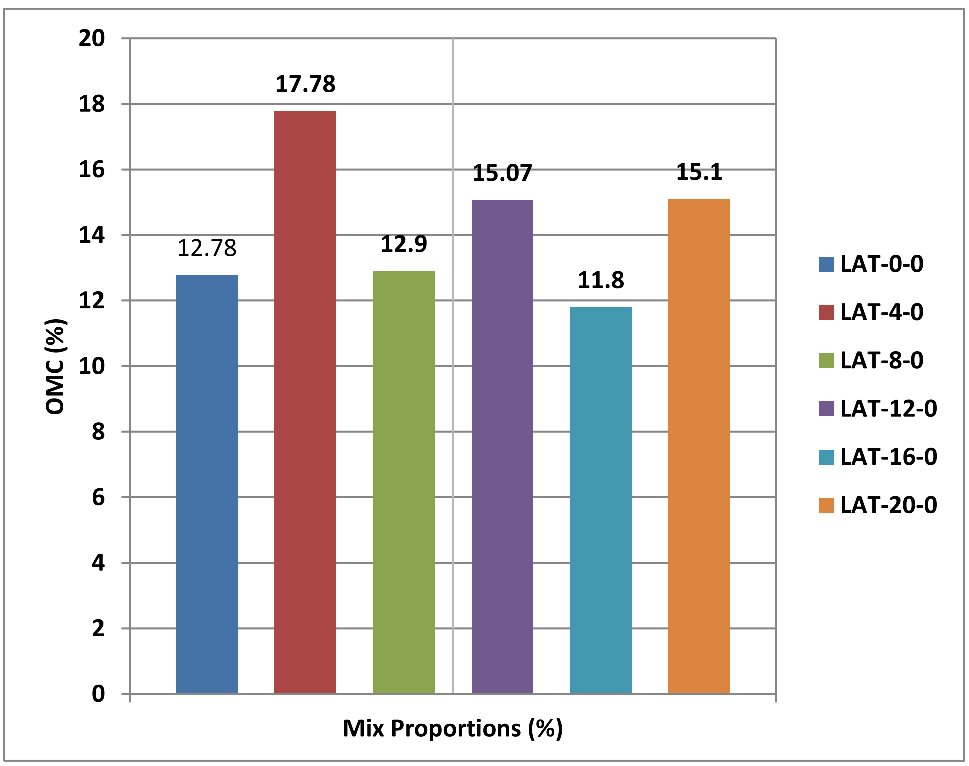


Figure 4.8: Optimum Moisture Content of Laterite Stabilized with Cement

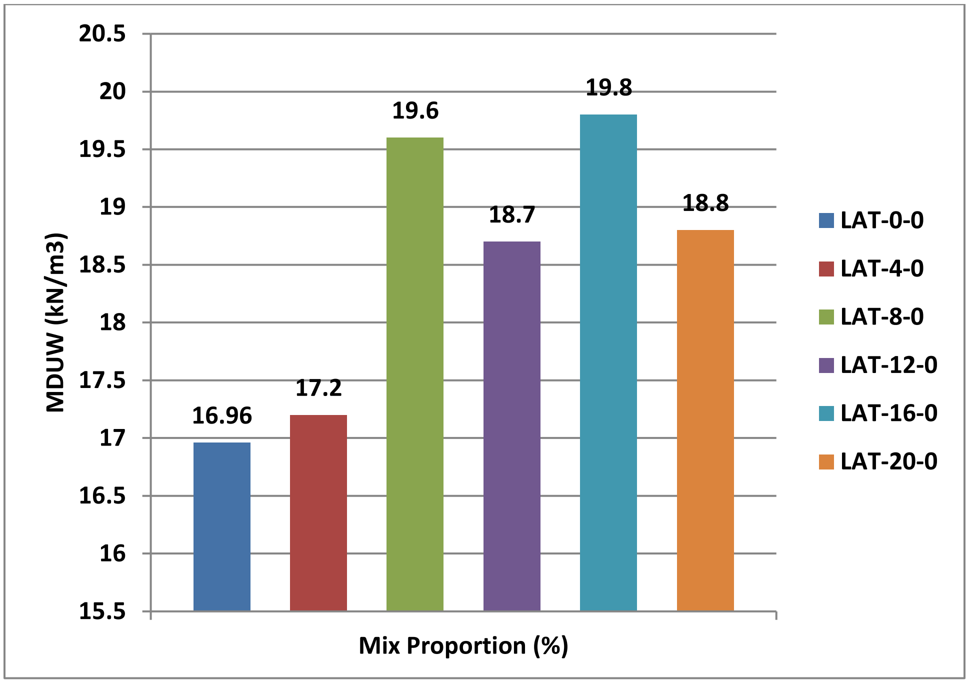


Figure 4.9: Maximum Dry Unit Weight of Laterite Stabilized with Sand

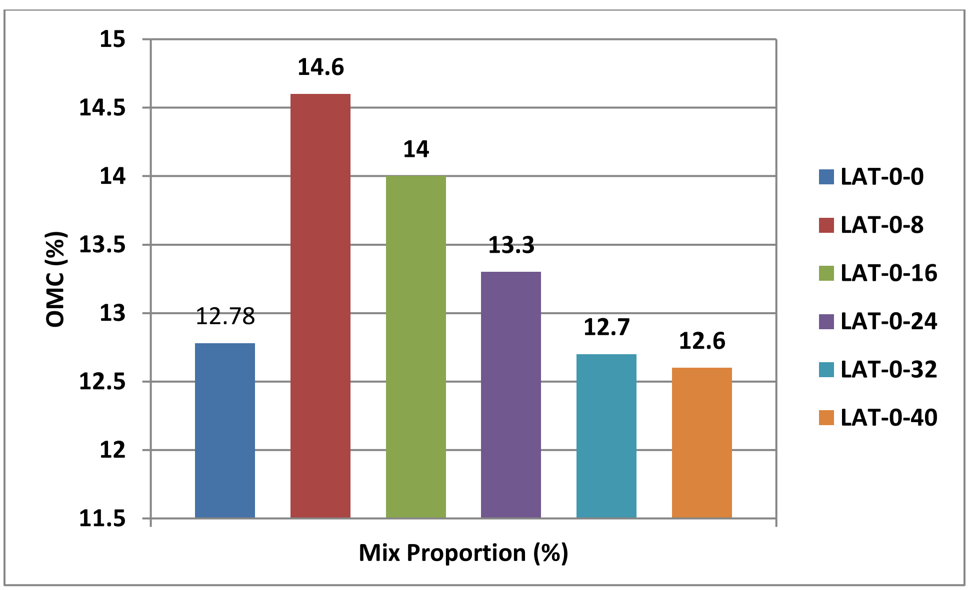


Figure 4.10: Optimum Moisture Content of Laterite Stabilized with Sand

4.2.5 CBR Test

Figure 4.11- 4.13 depicts the CBR characteristics of laterite stabilized with cement, sand and a blend of cement and sand. On addition of cement to laterite from 4% to 20%, it was observed that the CBR of laterite increased from 10.61% to 19.7% at 16% cement content, beyond 16% cement content, the CBR of the mixture decreased. While for sand to laterite mixture, the CBR of laterite increased from its natural value of 10.61% to 31.1%. Addition of blend of cement and sand to laterite increased the CBR of laterite from 15.9% to 34.1%. The improvement in Cbr of laterite on addition of cement, sand and a blend of cement and sand could be attributed to the bonding strength of cement and shear strength of sand. This finding is in agreement with the works of Okonkwo, et al. (2022).

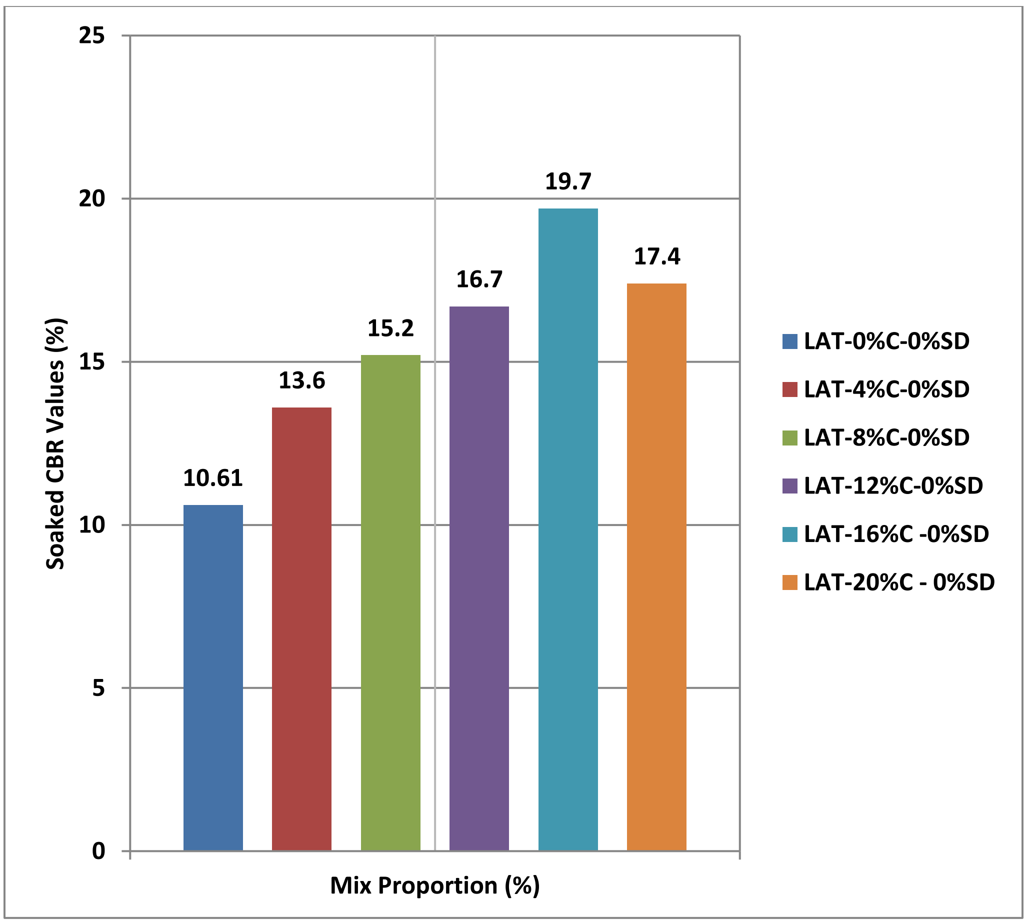


Figure 4.11: CBR Characteristics of Laterite Stabilized with Cement

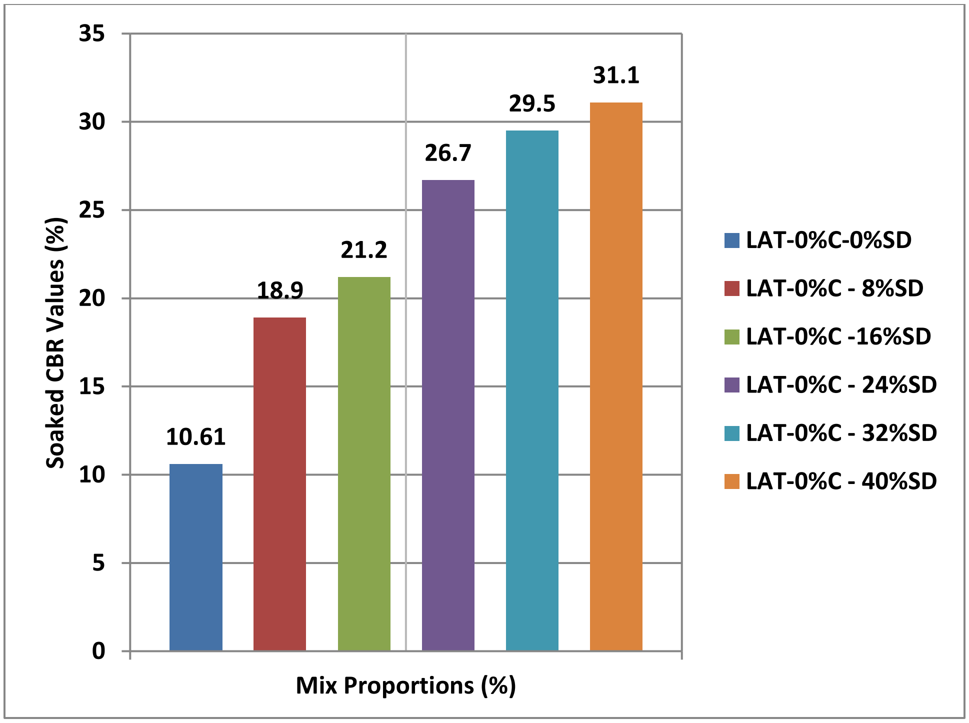


Figure 4.12: CBR Characteristics of Laterite Stabilized with Sand

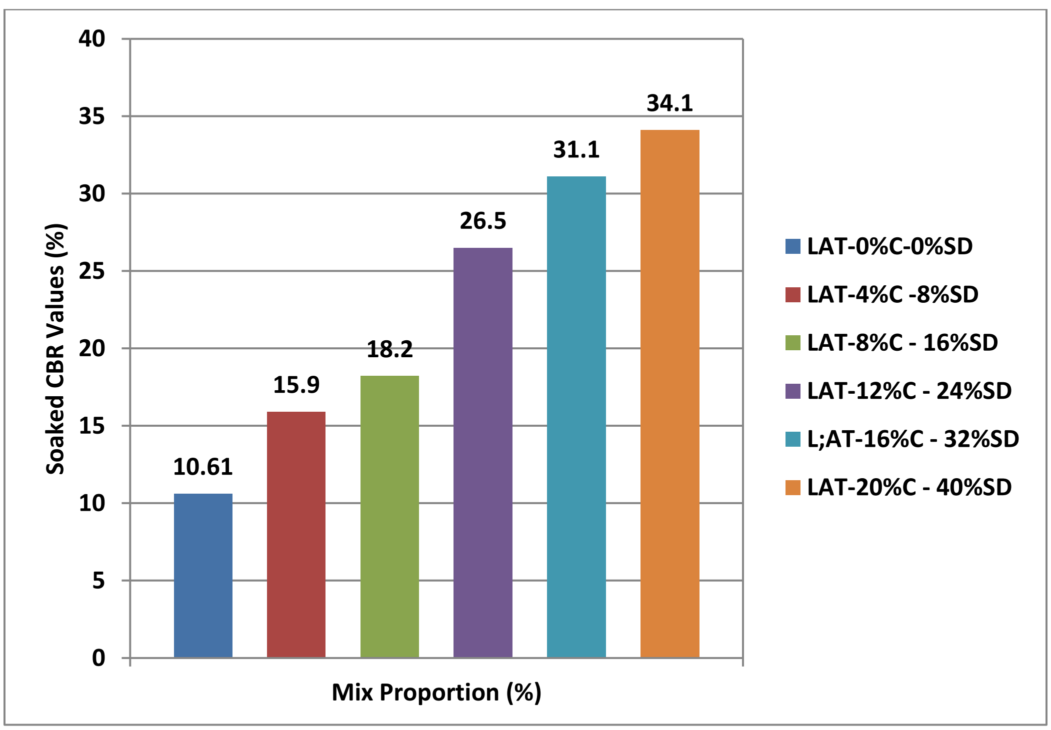


Figure 4.12: CBR Characteristics of Laterite Stabilized with Sand and Cement.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The following conclusion in the light of the findings obtained from investigation into the strength properties of laterite stabilized with cement and sand can be drawn:

1 Particle size distribution test for sand and laterite classified both samples as A-2-4 and A-2-6 according to AASHTO Classification System and SC (sand mixed with clay) and SM (sand mixed with silt) according to Unified Soil Classification System.

2 The specific gravity of laterite increased from 2.66 to 2.72 on addition of cement and sand at 16% and 32% beyond 16% cement and 32% sand, the specific gravity of the mixture decreased.

3 The liquid limit, plastic limit and plasticity index of laterite decreased up to 8% cement and 16% sand content. Beyond 8% cement and 16% sand content, addition of blend of sand and cement to laterite produced non-plastic results.

4 The maximum dry unit weight of laterite increased on addition of cement, sand and a blend of cement and sand to laterite.

5 The optimum moisture content of laterite decreased on addition of cement, sand and a blend of sand and cement to laterite. This implies that less amount of water is required to achieve maximum dry unit weight during field compaction.

6 The CBR of laterite increased on addition of cement, sand and a blend of cement and sand to laterite with the increase in CBR more substantial when a blend of cement and sand was added to laterite.

7 Cement and sand were adjudged as an effective modifier for enhancing the strength properties of poor lateritic soils.

5.2 Recommendations

From the findings obtained on strength properties of laterite stabilized with cement and sand, the following recommendation can be made:

1 The study encourages the use of sand and cement for treatment of poor lateritic soils as addition of cement and sand to laterite positively modifies the geotechnical properties

(particularly the strength properties) of laterite making them satisfy the criterion for use as building and road construction material.

2 In other to achieve cost effectiveness in stabilization of laterite using sand and cement, it is necessary to ascertain the minimum amount of cement required to yield optimum geotechnical properties (especially strength properties) in laterite as cement is a relatively expensive chemical stabilizing agents.

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APPENDICES

APPENDIX A

Specific Gravity Test

Table A1. Specific Gravity Result for LAT + 0%C + 0%SD

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.81	25.12	26.14
Wt of bottle + dry soil, W_2 (g).	34.79	35.11	36.12
Wt of bottle + soil + water, W_3 (g).	78.89	79.34	84.32
Wt of bottle + water, W_4 (g).	72.73	73.12	78.04

The Specific gravity of the sample is calculated as follows:

Specific Gravity for LAT + 0%C + 0%SD

Trial 1 (G_{S1}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.79-24.81)}{(34.79-24.81)-(78.89-72.73)} = 2.61$

Trial 2 (G_{S2}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.11-25.12)}{(35.11-25.12)-(79.34-73.12)} = 2.65$

Trial 3 (G_{S3}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(36.12-26.14)}{(36.12-26.14)-(84.32-78.04)} = 2.7$

Specific Gravity $= \frac{(GS1+GS2+GS3)}{3} = \frac{(2.61+2.65+2.70)}{3} = 2.65$

Table A2: Specific Gravity Result for LAT + 4%C + 8%SD

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.24	24.82	25.12
Wt of bottle + dry soil, W_2 (g).	35.26	34.85	35.16
Wt of bottle + soil + water, W_3 (g).	79.92	83.46	82.26
Wt of bottle + water, W_4 (g).	73.65	77.13	75.91

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The Specific gravity of the sample is calculated as follows:

Specific Gravity for LAT + 4%C + 8%SD

Trial 1 $(G_{S1}) = \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.26-24.24)}{(34.26-24.24)-(79.92-73.65)} = 2.67$

Trial 2 $(G_{S2}) = \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.85-24.82)}{(34.85-24.82)-(83.46-77.13)} = 2.72$

Trial 3 $(G_{S3}) = \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.16-25.12)}{(35.16-25.12)-(82.26-75.91)} = 271$

Specific Gravity $= \frac{(GS1+GS2+GS3)}{3} = \frac{(2.67+2.72+2.71)}{3} = 2.70$

Table A3: Specific Gravity Results for LAT + 8%C + 16%SD

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.82	25.24	25.62
Wt of bottle + dry soil, W_2 (g).	34.86	35.24	35.64
Wt of bottle + soil + water, W_3 (g).	80.14	78.64	82.24
Wt of bottle + water, W_4 (g).	73.80	72.36	75.89

The Specific gravity of the sample is calculated as follows:

Specific Gravity for LAT + 8%C + 16%SD

Trial 1 (G_{S1}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.86-24.82)}{(34.86-24.82)-(80.14-73.80)} = 2.71$

Trial 2 (G_{S2}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.24-25.24)}{(35.24-25.24)-(78.64-72.36)} = 2.69$

Trial 3 (G_{S3}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.64-25.64)}{(35.64-25.64)-(82.24-75.89)} = 2.73$

Specific Gravity $= \frac{(GS1+GS2+GS3)}{3} = \frac{(2.71+2.69+2.73)}{3} = 2.71$

Table A4. Specific Gravity Result for LAT + 12%C + 24%SD

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	**24.82**	**25.32**	**24.64**
Wt of bottle + dry soil, W_2 (g).	**34.86**	**35.35**	**34.66**
Wt of bottle + soil + water, W_3 (g).	**80.08**	**82.14**	**78.63**
Wt of bottle + water, W_4 (g).	**73.73**	**75.77**	**72.25**

The Specific gravity of the sample is calculated as follows:

Specific Gravity for LAT + 12%C + 24%SD

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.86-24.82)}{(34.86-24.82)-(80.08-73.73)} = 2.72$

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.35-25.32)}{(35.35-25.32)-(82.14-75.77)} = 2.74$

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(34.66-24.64)}{(34.66-24.64)-(78.63-72.25)} = 2.75$

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.72+2.74+2.75)}{3} = 2.74$

Table A5: Specific Gravity Results for LAT + 16%C + 32%SD

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	24.98	25.90	25.73
Wt of bottle + dry soil, W_2 (g).	34.98	35.90	25.73
Wt of bottle + soil + water, W_3 (g).	84.90	85.79	85.62
Wt of bottle + water, W_4 (g).	78.51	79.41	79.25

The Specific gravity of the sample is calculated as follows:

Specific Gravity for LAT + 16%C + 32%SD

Trial 1 (G_{S1}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.98-24.98)}{(34.98-24.98)-(84.90-78.51)}$ **= 2.77**

Trial 2 (G_{S2}) = $\frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.90-25.90)}{(35.90-25.90)-(85.79-79.41)}$ **= 2.76**

Trial 3 (G_{S3}) = $\frac{(W2-W1)}{(W2-W1)-\ (W3-W4)} = \frac{(35.73-25.73)}{(35.73-25.73)(85.62-79.25)}$ **= 2.75**

Specific Gravity = $\frac{(GS1+GS2+GS3)}{3} = \frac{(2.77+2.76+2.75)}{3}$ **= 2.76**

Table A6: Specific Gravity Result for LAT + 20%C + 40%SD

Determinants	Trial 1	Trial 2	Trial 3
Wt of density bottle, W_1 (g).	25.12	24.91	24.66
Wt of bottle + dry soil, W_2 (g).	35.12	34.90	34.68
Wt of bottle + soil + water, W_3 (g).	82.16	78.84	80.54
Wt of bottle + water, W_4 (g).	75.86	72.50	77.20

The Specific gravity of the sample is calculated as follows:

Specific Gravity for LAT + 20%C + 40%SD

Trial 1 (G_{S1}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(35.12-25.12)}{(35.12-25.12)-(82.16-75.86)} = 2.70$

Trial 2 (G_{S2}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.90-24.91)}{(34.90-24.91)-(78.84-72.50)} = 2.74$

Trial 3 (G_{S3}) $= \frac{(W2-W1)}{(W2-W1)-(W3-W4)} = \frac{(34.68-24.66)}{(34.68-24.66)-(80.54-77.20)} = 2.72$

Specific Gravity $= \frac{(GS1+GS2+GS3)}{3} = \frac{(2.70+2.74+2.72)}{3} = 2.72$

APPENDIX B

CBR Test

Table B1 CBR Result for LAT-0%C -0%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.3	0.5	0.8	1.1	1.4	1.7	2	2.2	2.5	2.8	3.1	3.4	3.7	4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.1	0.4	0.7	0.9	1.1	1.4	1.7	2.0	2.2	2.5	2.8	3.1	3.4	3.7

Table B2 CBR Result for LAT-4%C -0%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.6	0.9	1.1	1.5	1.8	2.1	2.4	2.6	2.9	3.2	3.5	3.8	4.1	4.4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.4	0.6	0.9	1.1	1.3	1.7	1.9	2.2	2.5	2.8	3.1	3.4	3.6	3.9

Table B3 CBR Result for LAT-8%C -0%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.8	1.1	1.4	1.7	2	2.3	2.6	2.9	3.1	3.4	3.7	4	4.3	4.6
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.3	0.7	0.9	1.2	1.4	1.7	2	2.1	2.4	2.7	2.9	3.1	3.4	3.7

Table B4 CBR Result for LAT-12%C -0%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.1	1.4	1.7	2	2.2	2.5	2.7	2.9	3.2	3.4	3.6	3.8	4.1	4.3
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.7	1	1.2	1.5	1.7	1.9	2.1	2.3	2.5	2.7	2.9	3.1	3.3	3.5

Table B5 CBR Result for LAT-16%C -0%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.3	1.6	1.8	2.3	2.6	2.8	3.0	3.2	3.5	3.7	3.8	4.1	4.3	4.5
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.9	1.2	1.5	1.8	2.1	2.4	2.6	2.9	3.1	3.3	3.5	3.7	3.8	4.0

Table B6 CBR Result for LAT-20%C -0%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.2	1.4	1.7	2	2.3	2.7	2.9	3	3.2	3.4	3.7	4	4.2	4.3
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.6	0.9	1.1	1.4	1.7	2.0	2.2	2.4	2.7	2.9	3.1	3.3	3.5	3.7

Table B7 CBR Result for LAT-0%C -8%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1	1.4	1.8	2.2	2.5	2.8	3.2	3.5	3.7	4	4.3	4.6	4.9	5.2
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.7	1	1.4	1.7	2	2.2	2.5	2.7	3	3.2	3.5	3.7	4	4.2

Table B8 CBR Result for LAT-0%C -16%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.4	1.8	2.3	2.5	2.8	3.1	3.4	3.7	3.9	4.2	4.4	4.6	4.9	5.2
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	1.1	1.3	1.7	2.1	2.4	2.7	3	3.3	3.6	3.9	4.2	4.5	4.8	5.1

Table B9 CBR Result for LAT-0%C -24%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.8	2.4	2.9	3.3	3.5	3.8	4.2	4.5	4.7	5	5.3	5.6	5.8	6.2
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	1.4	1.7	2.1	2.4	2.7	3	3.4	3.7	4	4.3	4.6	4.9	5.2	5.5

Table B10 CBR Result for LAT-0%C -32%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	2.2	2.7	3.1	3.5	3.9	4.2	4.5	4.8	5.1	5.4	5.7	6.1	6.4	6.7
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	1.7	2	2.4	2.7	3	3.3	3.7	4	4.3	4.6	4.9	5.2	5.5	5.8

Table B11 CBR Result for LAT-0%C -40%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	2.5	3	3.4	3.8	4.1	4.4	4.7	5	5.2	5.6	5.9	6.2	6.4	6.7
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4	4.3	4.6	4.9	5.2	5.5	5.8

Table B12 CBR Result for LAT-4%C -8%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	0.7	1.2	1.5	1.8	2.1	2.4	2.7	3	3.2	3.5	3.8	4.1	4.4	4.7
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.5	0.7	0.9	1.1	1.4	1.7	2	2.2	2.5	2.7	3	3.2	3.4	3.6

Table B13 CBR Result for LAT-8%C -16%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.2	1.6	1.9	2.3	2.4	2.7	3	3.3	3.5	3.8	4	4.2	4.4	4.7
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	0.8	1.1	1.4	1.7	2	2.3	2.6	2.9	3.2	3.5	3.8	4	4.1	4.4

Table B14 CBR Result for LAT-12%C -24%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	1.8	2.2	2.7	3.1	3.5	3.8	4.2	4.6	4.9	5.2	5.5	5.8	6.1	6.4
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	1.2	15	1.9	2.3	2.5	2.8	3.2	3.5	3.8	4.1	4.4	4.6	4.9	5.2

Table B15 CBR Result for LAT-16%C -32%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	2.2	2.6	3.3	3.7	4.1	4.4	4.7	5.1	5.4	5.7	6	6.3	6.6	6.8
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	1.7	2	2.3	2.6	2.9	3.2	3.5	3.8	4.1	4.4	4.7	5	5.2	5.5

Table B16 CBR Result for LAT-20%C -40%SD

Penetration (mm)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
Dial reading (Top)	36	54	71	84	96	108	115	122	126	130	136	140	154	158
Force (KN)	2.7	3.1	3.6	4	4.5	4.8	5.2	5.5	5.7	6.1	6.5	6.8	7.1	7.3
Dial reading (Bottom)	10	25	38	52	67	82	99	110	126	137	148	157	166	174
Force (KN)	2.1	2.4	2.7	3.1	3.4	3.8	4.2	4.5	4.9	5.2	5.5	5.8	6.1	6.4

APPENDIX C

Sieve Analysis Test

Table C1: Sieve Analysis Test Result for LAT

Sieve Sizes (mm)	Mass Retained	% Mass Retained	Cum % Retained	Cum % Finer
2	0.02	0.004	0.004	99.996
1.18	4.95	0.99	0.994	99.006
0.85	14.04	2.808	3.802	96.198
0.6	33.1	6.62	10.422	89.578
0.425	38	7.6	18.022	81.978
0.3	100.4	20.08	38.102	61.898
0.15	178.18	35.636	73.738	26.262
0.075	20.02	4.004	77.742	22.258
Tray	11.26			
Total	500			

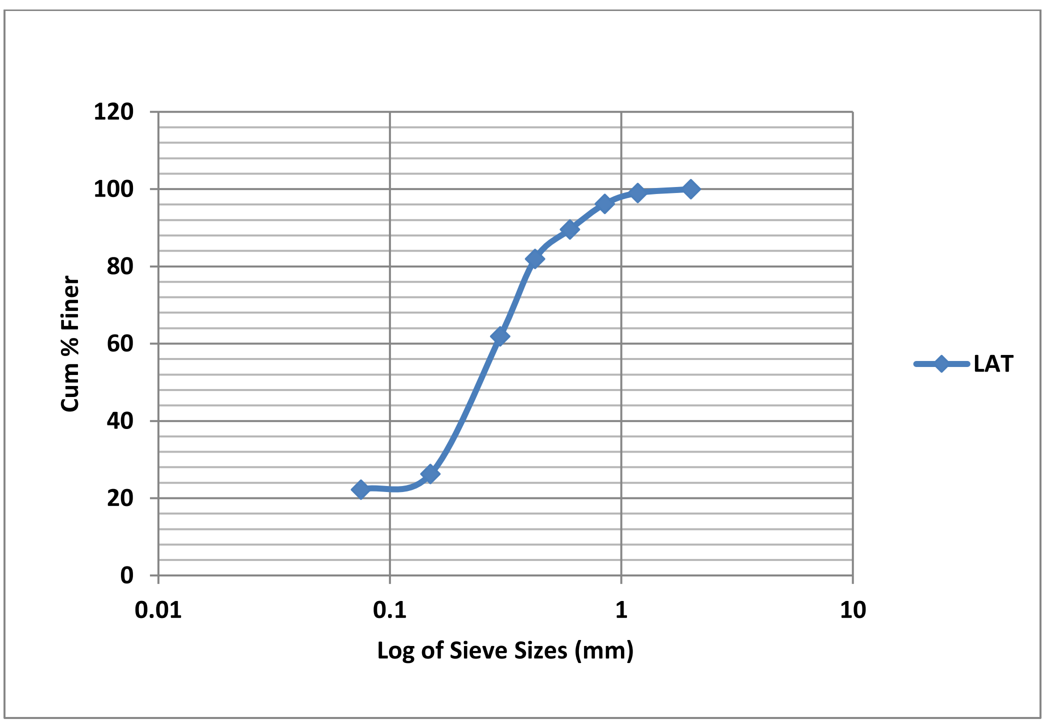

Table C1: Particle Size Distribution Curve for Laterite

Table C1: Sieve Analysis Test Result for Sand

Sieve Sizes (mm)	Mass Retained	% Mass Retained	Cum % Retained	Cum % Finer
2	7.97	2.66	2.66	97.34
1.18	9.98	3.33	5.99	94.01
0.85	11.73	3.91	9.90	90.10
0.6	27.95	9.32	19.21	80.79
0.425	44.02	14.67	33.89	66.11
0.3	53.69	17.90	51.78	48.22
0.15	112.1	37.37	89.15	10.85
0.075	8.46	2.82	91.97	8.03
Tray	2.3			
Total	300			

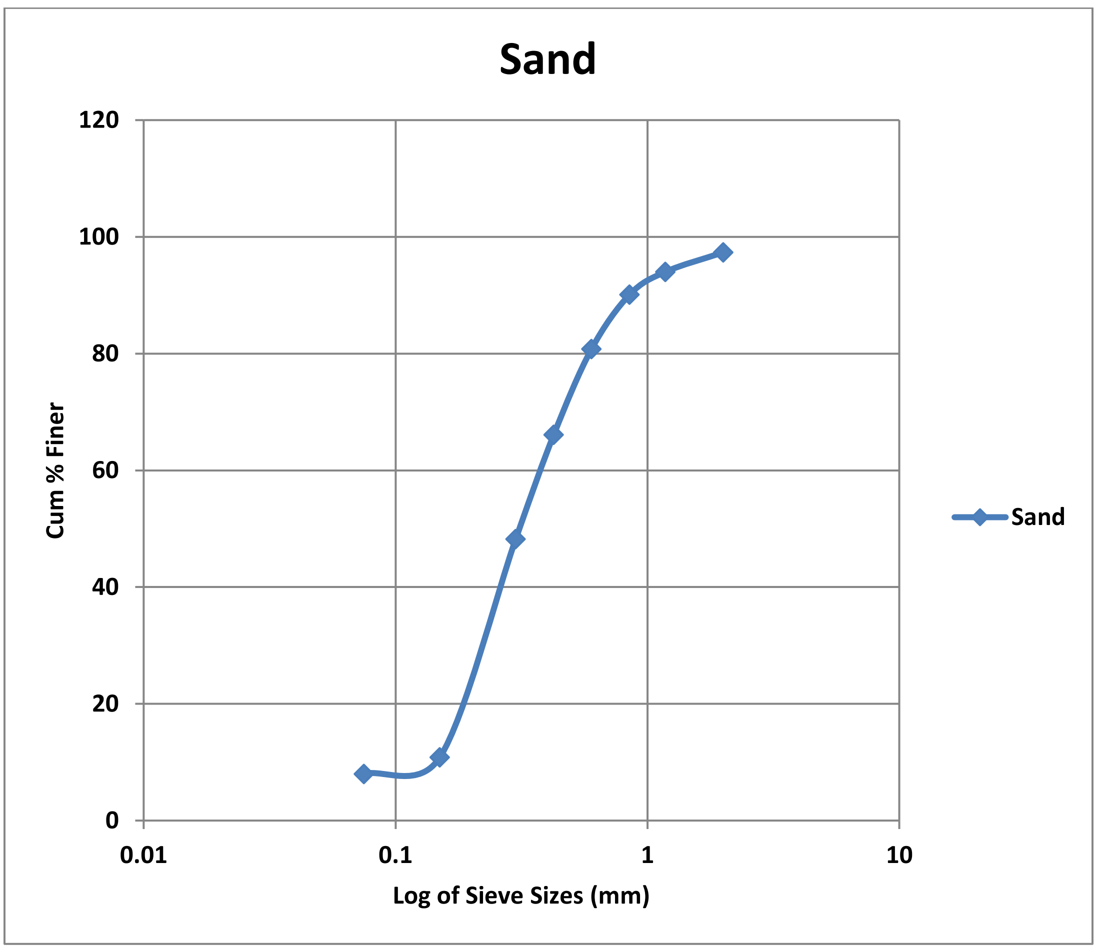

Table C2: Particle Size Distribution Curve for Sand

APPENDIX D

Atterberg Limit Test

Table D1: Liquid Limit Result for LAT +0%SD + 0%C

BLOWS	33	27	22	18	14
Wt of empty tin (g)	14.82	15.64	16.48	14.88	15.21
Wt of tin + wet soil (g)	46.24	34.62	32.28	26.94	30.66
Wt of wet soil (g)	31.42	18.98	15.8	12.06	15.45
Wt of tin +dry soil (g)	39.14	29.78	27.78	23.25	25.67
Wt of dry soil (g)	24.32	14.14	11.3	8.37	10.46
Wt of water (g)	7.1	4.84	4.5	3.69	4.99
Moisture Content (g)	29.19	34.23	39.82	44.09	47.71

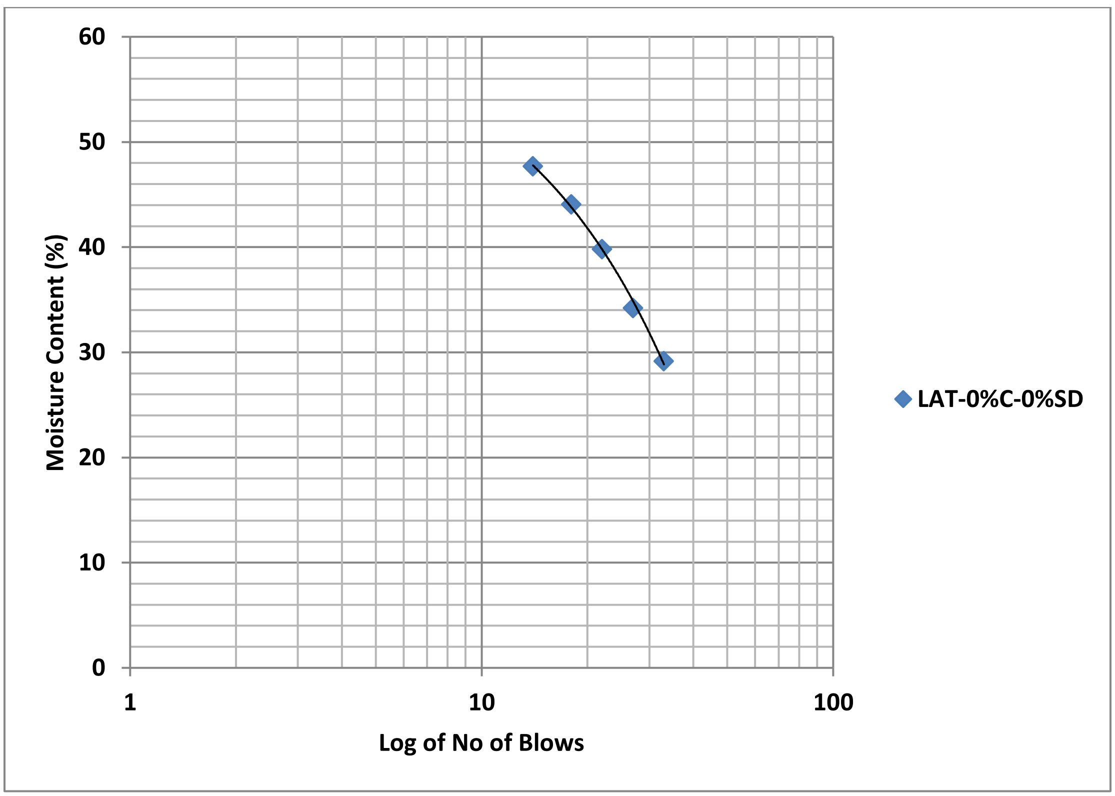

Figure D1: Liquid Limit Graph for LAT + 0%SD + 0%C

Table D2: Liquid Limit Result for LAT +8%SD + 4%C

BLOWS	33	28	23	17	13
Wt of empty tin (g)	14.18	16.42	17.16	15.49	16.04
Wt of tin + wet soil (g)	50.16	44.28	46.28	48.56	40.82
Wt of wet soil (g)	35.98	27.86	29.12	33.07	24.78
Wt of tin +dry soil (g)	42.68	37.69	38.59	39.45	33.67
Wt of dry soil (g)	28.5	21.27	21.43	23.96	17.63
Wt of water (g)	7.48	6.59	7.69	9.11	7.15
Moisture Content (g)	26.25	30.98	35.88	38.02	40.56

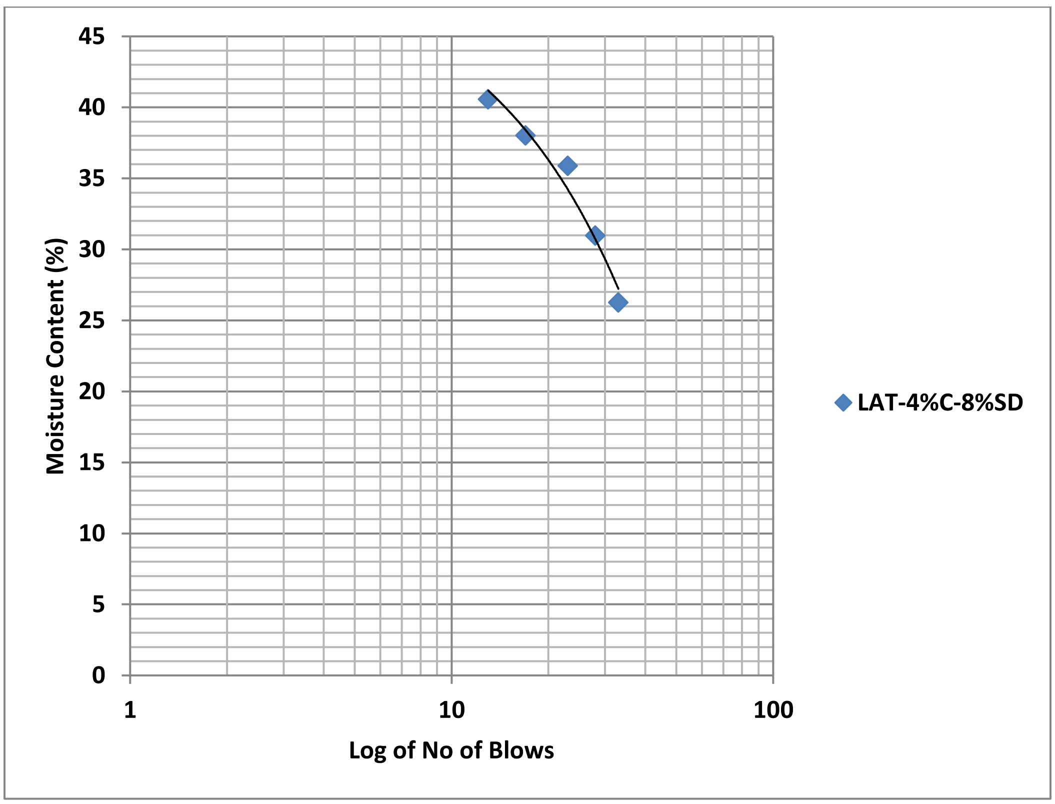

Figure D2: Liquid Limit Graph for LAT + 8%SD + 4%C

Table D3: Liquid Limit Result for LAT +16%SD + 8%C

BLOWS	33	26	22	18	14
Wt of empty tin (g)	14.89	18.46	17.56	15.06	16.57
Wt of tin + wet soil (g)	45.05	50.57	47.44	40.93	49.21
Wt of wet soil (g)	30.16	32.11	29.88	25.87	32.64
Wt of tin +dry soil (g)	39.98	43.96	41.05	34.71	40.61
Wt of dry soil (g)	25.09	25.5	23.49	19.65	24.04
Wt of water (g)	5.07	6.61	6.39	6.22	8.6
Moisture Content (g)	20.21	25.92	27.20	31.65	35.77

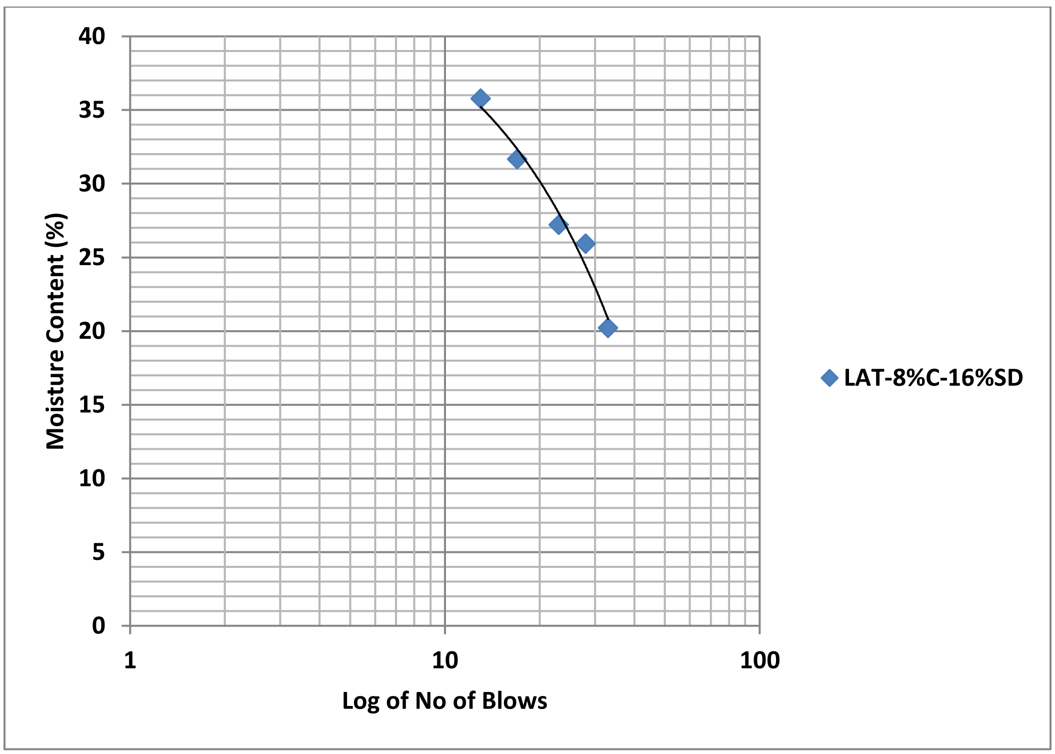

Figure D3: Liquid Limit Graph for LAT + 16%SD + 8%C

Table D4: Plastic Limit Results for LAT + 0%SD + 0%C

LAT-0%C-0%SD	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.32	15.84	14.82
Wt of tin + wet soil (g)	34.77	29.24	36.74
Wt of wet soil (g)	20.45	13.4	21.92
Wt of tin + dry soil (g)	31.31	26.91	33.09
Wt of dry soil (g)	16.99	11.07	18.27
Wt of water (g)	3.46	2.33	3.65
Plastic Limit (%)	20.36	21.05	19.98

Table D5: Plastic Limit Results for LAT + 8%SD + 4%C

LAT-4%C-8%SD	Test 1	Test 2	Test 3
Wt of empty tin (g)	15.62	14.7	13.64
Wt of tin + wet soil (g)	26.84	30.52	34.88
Wt of wet soil (g)	11.22	15.82	21.24
Wt of tin + dry soil (g)	25.02	28.44	31.61
Wt of dry soil (g)	9.4	13.74	17.97
Wt of water (g)	1.82	2.08	3.27
Plastic Limit (%)	19.36	15.14	18.20

Table D6: Plastic Limit Results for LAT + 16%SD + 8%C

LAT-8%C-16%SD	Test 1	Test 2	Test 3
Wt of empty tin (g)	14.63	15.42	16.88
Wt of tin + wet soil (g)	24.86	28.69	32.88
Wt of wet soil (g)	10.23	13.27	16
Wt of tin + dry soil (g)	23.49	27.03	30.47
Wt of dry soil (g)	8.86	11.61	13.59
Wt of water (g)	1.37	1.66	2.41
Plastic Limit (%)	15.46	14.30	17.73

APPENDIX E

Compaction Test

Table E1: Dry Unit Weight Results for LAT + 0%C + 0%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.5	1.5	14.72	3.35	14.24
8	0.001	4	5.65	1.65	16.19	5.47	15.35
12	0.001	4	5.8	1.8	17.66	7.76	16.39
16	0.001	4	5.95	1.95	19.13	12.78	16.96
20	0.001	4	5.9	1.9	18.64	16.59	15.99

Table E1.1: Moisture Content Determination for LAT + 0%C + 0%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.71	28.34	13.63	27.95	13.24	0.39	2.95
8	15.03	40.04	25.01	39.04	24.01	1	4.16
12	14.31	46.22	31.91	43.06	28.75	3.16	10.99
16	17.82	65.65	47.83	59.54	41.72	6.11	14.65
20	13.56	63.34	49.78	52.78	39.22	10.56	26.93

Table E1.2: Moisture Content Determination for LAT + 0%C + 0%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.83	32.73	16.9	32.12	16.29	0.61	3.74
8	14.44	46.43	31.99	44.06	29.62	2.37	8.00
12	14.06	45.44	31.38	42.24	28.18	3.2	11.36
16	17.45	55.99	38.54	51.09	33.64	4.9	14.57
20	13.79	55.75	41.96	49.19	35.4	6.56	18.53

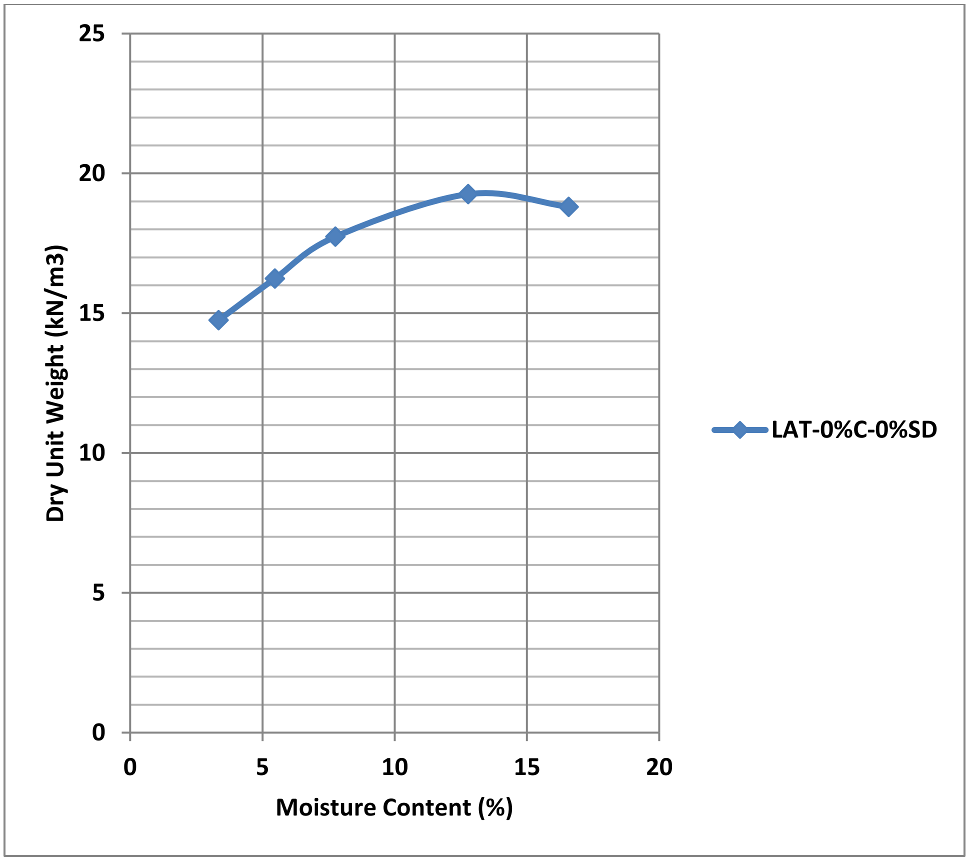

Figure E1: Compaction Curve for LAT + 0%C + 0%SD

Table E2: Dry Unit Weight Results for LAT + 4%C + 0%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.5	1.5	14.72	5.01	14.01
8	0.001	4	5.9	1.9	18.64	10.46	16.87
12	0.001	4	6	2	19.62	14.21	17.18
16	0.001	4	6.05	2.05	20.11	17.78	17.07
20	0.001	4	6	2	19.62	21.58	16.14

Table E2.1: Moisture Content Determination for LAT + 4%C + 0%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	16.65	34.58	17.93	33.68	17.03	0.9	5.28
8	15.88	47.69	31.81	44.95	29.07	2.74	9.43
12	15.3	51	35.7	46.74	31.44	4.26	13.55
16	14.18	70.95	56.77	62.5	48.32	8.45	17.49
20	15.08	65.43	50.35	56.86	41.78	8.57	20.51

Table E2.2: Moisture Content Determination for LAT + 4%C + 0%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.85	29.22	14.37	28.57	13.72	0.65	4.74
8	17.82	58.27	40.45	54.1	36.28	4.17	11.49
12	17.82	46.41	28.59	42.71	24.89	3.7	14.87
16	15.52	64.84	49.32	57.29	41.77	7.55	18.08
20	16.64	67.76	51.12	58.32	41.68	9.44	22.65

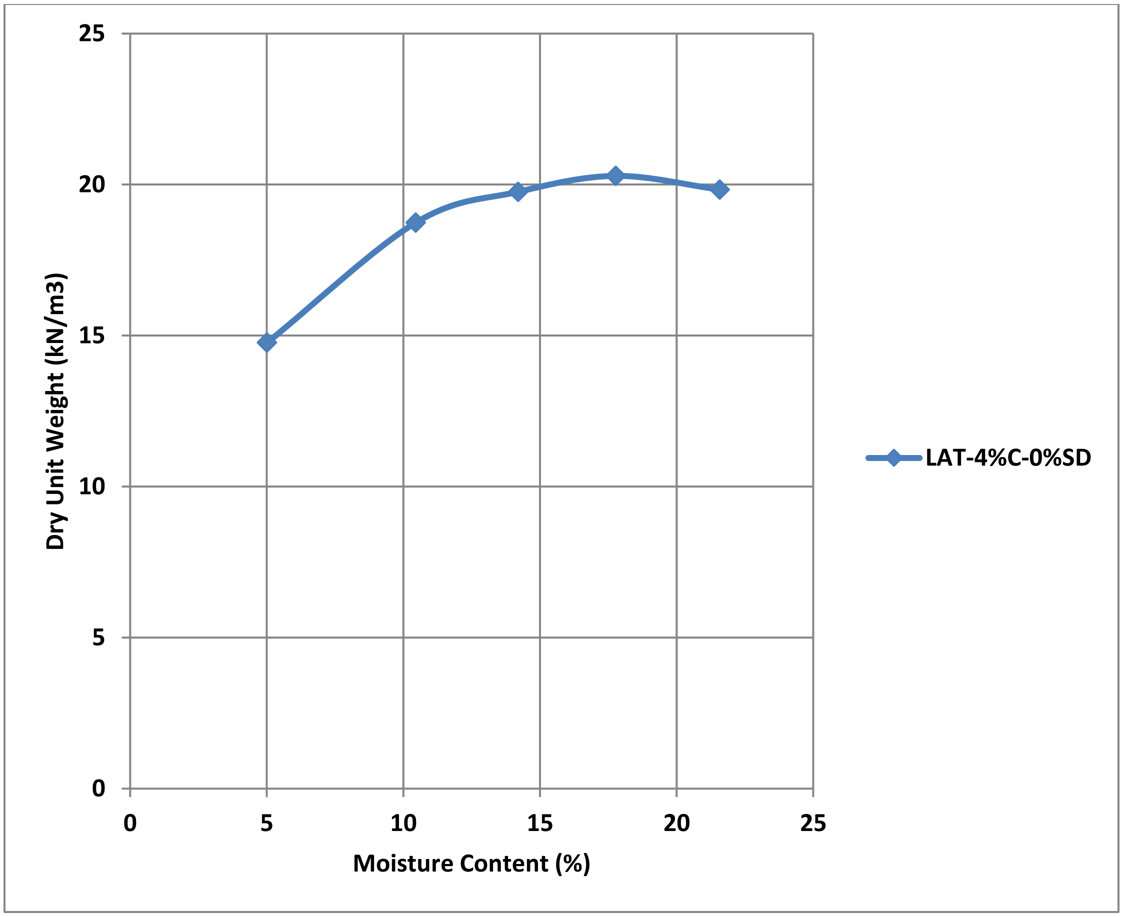

Figure E2: Compaction Curve for LAT + 4%C + 0%SD

Table E3: Dry Unit Weight Results for LAT + 8%C + 0%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.65	1.65	16.19	4.13	15.54
8	0.001	4	6	2	19.62	6.43	18.43
12	0.001	4	6.15	2.15	21.09	7.86	19.55
16	0.001	4	6.2	2.2	21.58	12.94	19.11
20	0.001	4	6.15	2.15	21.09	15.53	18.26

Table E3.1: Moisture Content Determination for LAT + 8%C + 0%SD (Top)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	15.7	31.13	15.43	30.53	14.83	0.6	4.05
8	14.07	38.5	24.43	37.04	22.97	1.46	6.36
12	17.79	47.27	29.48	45.85	28.06	1.42	5.06
16	14.36	56.93	42.57	52.08	37.72	4.85	12.86
20	15.07	62.54	47.47	55.95	40.88	6.59	16.12

Table E3.2: Moisture Content Determination for LAT + 8%C + 0%SD (Bottom)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	17.45	33.78	16.33	33.12	15.67	0.66	4.21
8	14.71	38.94	24.23	37.46	22.75	1.48	6.51
12	14.98	55.77	40.79	51.84	36.86	3.93	10.66
16	16.07	54.59	38.52	50.15	34.08	4.44	13.03
20	15.75	53.62	37.87	48.7	32.95	4.92	14.93

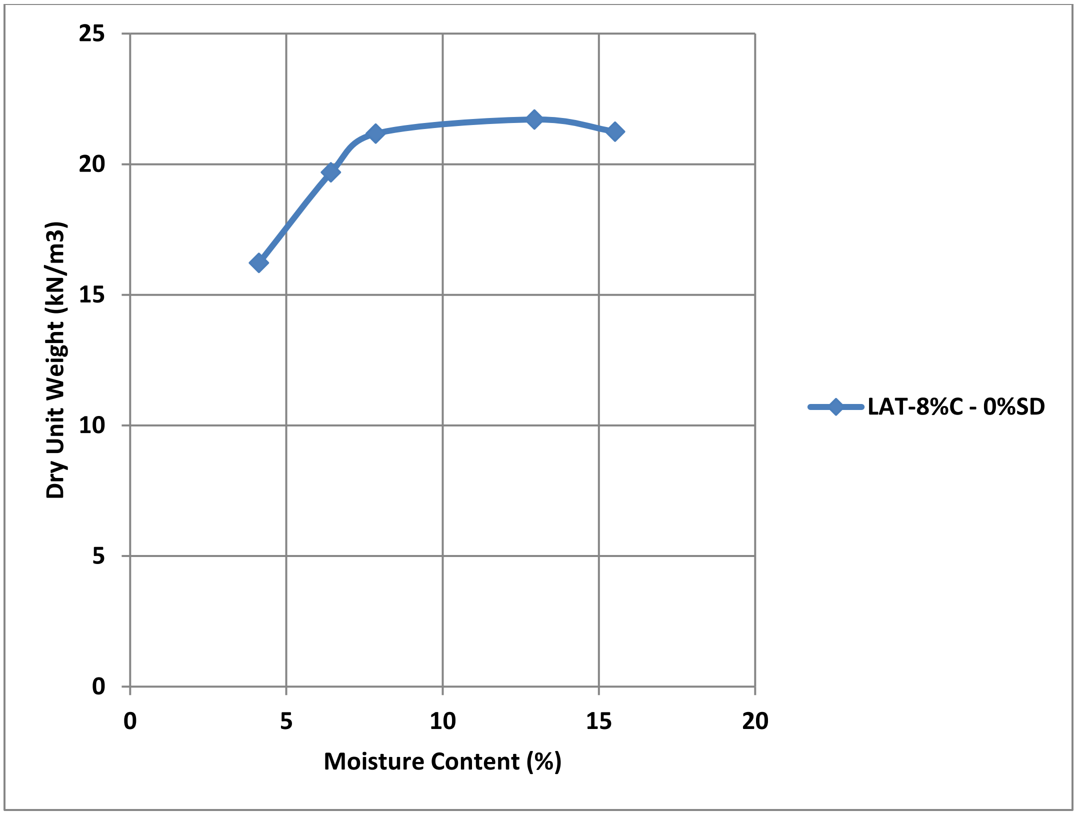

Figure E3: Compaction Curve for LAT + 8%C + 0%SD

Table E4: Dry Unit Weight Results for LAT + 12%C + 0%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.65	1.65	16.19	3.75	15.60
8	0.001	4	5.85	1.85	18.15	10.68	16.40
12	0.001	4	6.1	2.1	20.60	13.74	18.11
16	0.001	4	6.2	2.2	21.58	15.07	18.76
20	0.001	4	6.1	2.1	20.60	18.74	17.35

Table E4.1: Moisture Content Determination for LAT + 12%C + 0%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.57	37.31	21.74	36.63	21.06	0.68	3.23
8	15.36	38.45	23.09	36.27	20.91	2.18	10.43
12	14.53	45.31	30.78	41.65	27.12	3.66	13.50
16	13.59	50.38	36.79	45.93	32.34	4.45	13.76
20	16.62	60.99	44.37	53.59	36.97	7.4	20.02

Table E4.2: Moisture Content Determination for LAT + 12%C + 0%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.04	35.34	20.3	34.51	19.47	0.83	4.26
8	15.22	46.59	31.37	43.5	28.28	3.09	10.93
12	13.85	44.41	30.56	40.66	26.81	3.75	13.99
16	15.86	58.79	42.93	52.75	36.89	6.04	16.37
20	14.37	67.17	52.8	59.32	44.95	7.85	17.46

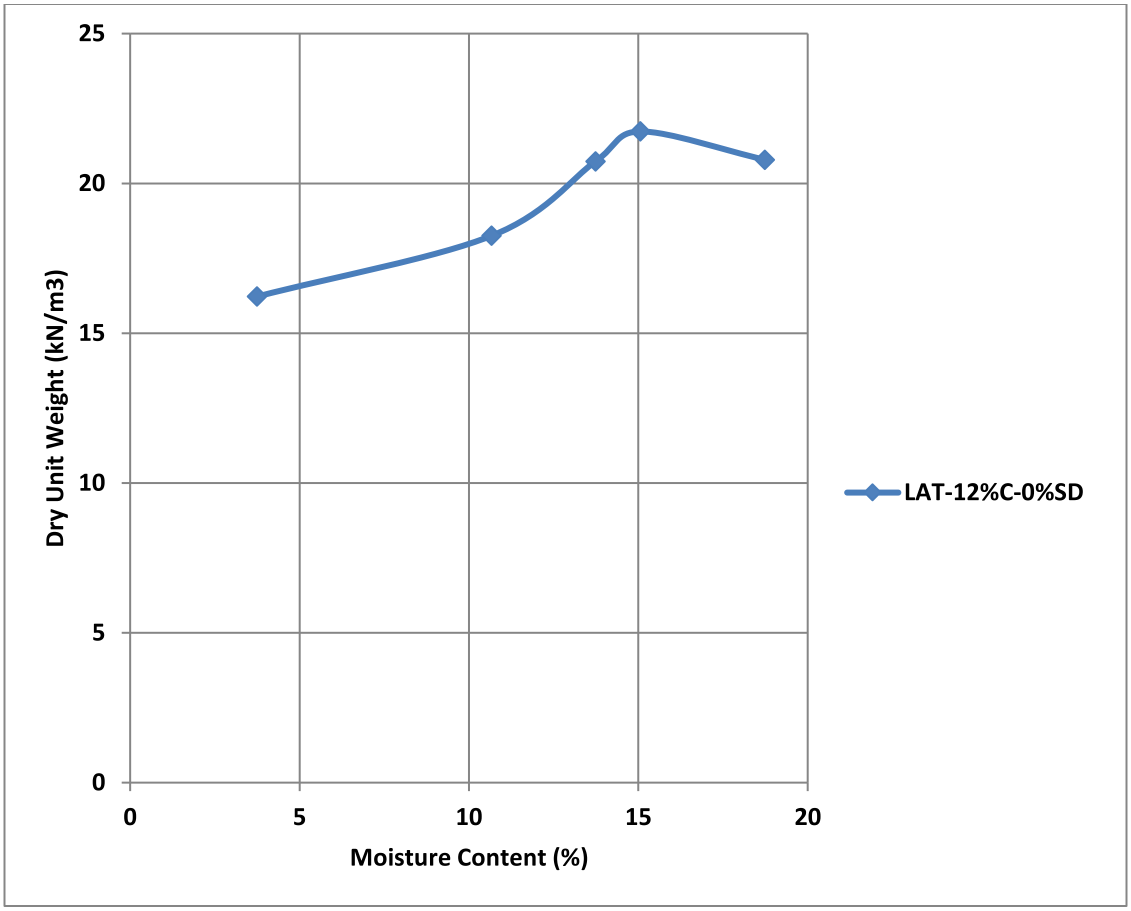

Figure E4: Compaction Curve for LAT + 12%C + 0%SD

Table E5: Dry Unit Weight Results for LAT + 16%C + 0%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.7	1.7	16.68	3.49	16.12
8	0.001	4	6.05	2.05	20.11	6.23	18.93
12	0.001	4	6.2	2.2	21.58	9.13	19.78
16	0.001	4	6.25	2.25	22.07	11.80	19.74
20	0.001	4	6.2	2.2	21.58	14.01	18.93

Table E5.1: Moisture Content Determination for LAT + 16%C + 0%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.23	31.78	16.55	31.21	15.98	0.57	3.57
8	14.85	47.69	32.84	45.76	30.91	1.93	6.24
12	14.54	56.54	42	53.04	38.5	3.5	9.09
16	14.12	59.37	45.25	54.61	40.49	4.76	11.76
20	14.07	59.08	45.01	53.64	39.57	5.44	13.75

Table E5.2: Moisture Content Determination for LAT + 16%C + 0%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	17.61	35.53	17.92	34.94	17.33	0.59	3.40
8	14.58	48.44	33.86	46.46	31.88	1.98	6.21
12	14.8	46.62	31.82	43.95	29.15	2.67	9.16
16	14.64	60.66	46.02	55.79	41.15	4.87	11.83
20	14.43	58.76	44.33	53.22	38.79	5.54	14.28

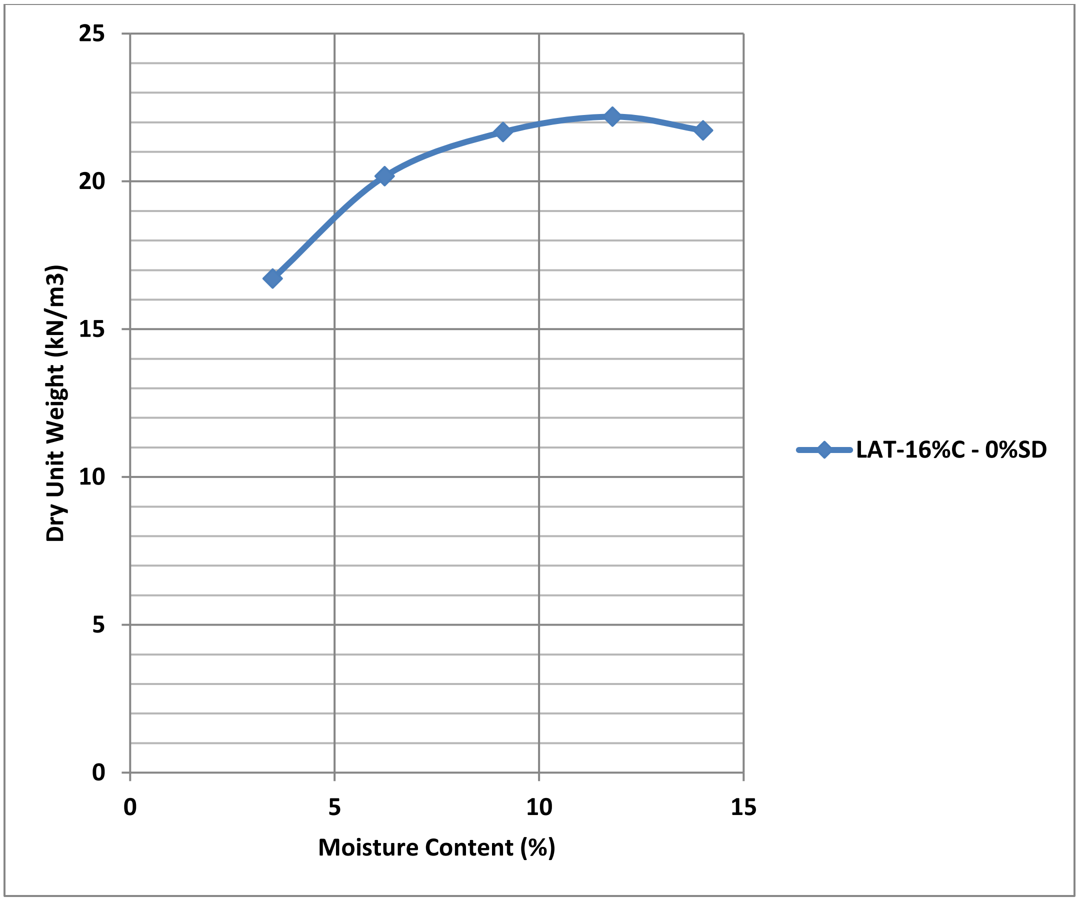

Figure E5: Compaction Curve for LAT + 16%C + 0%SD

Table E6: Dry Unit Weight Results for LAT + 20%C + 0%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.55	1.55	15.21	4.16	14.60
8	0.001	4	5.65	1.65	16.19	9.70	14.76
12	0.001	4	6.05	2.05	20.11	11.99	17.96
16	0.001	4	6.2	2.2	21.58	15.09	18.75
20	0.001	4	6	2	19.62	19.30	16.45

Table E6.1: Moisture Content Determination for LAT + 20%C + 0%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.96	31.69	16.73	30.96	16	0.73	4.56
8	15.21	37.1	21.89	35.08	19.87	2.02	10.17
12	13.82	40.93	27.11	38.05	24.23	2.88	11.89
16	15.83	56.01	40.18	50.72	34.89	5.29	15.16
20	14.41	70.05	55.64	61.45	47.04	8.6	18.28

Table E6.2: Moisture Content Determination for LAT + 20%C + 0%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.96	31.69	16.73	30.96	16	0.73	4.56
8	15.21	37.1	21.89	35.08	19.87	2.02	10.17
12	13.82	40.93	27.11	38.05	24.23	2.88	11.89
16	15.83	56.01	40.18	50.72	34.89	5.29	15.16
20	14.41	70.05	55.64	61.45	47.04	8.6	18.28

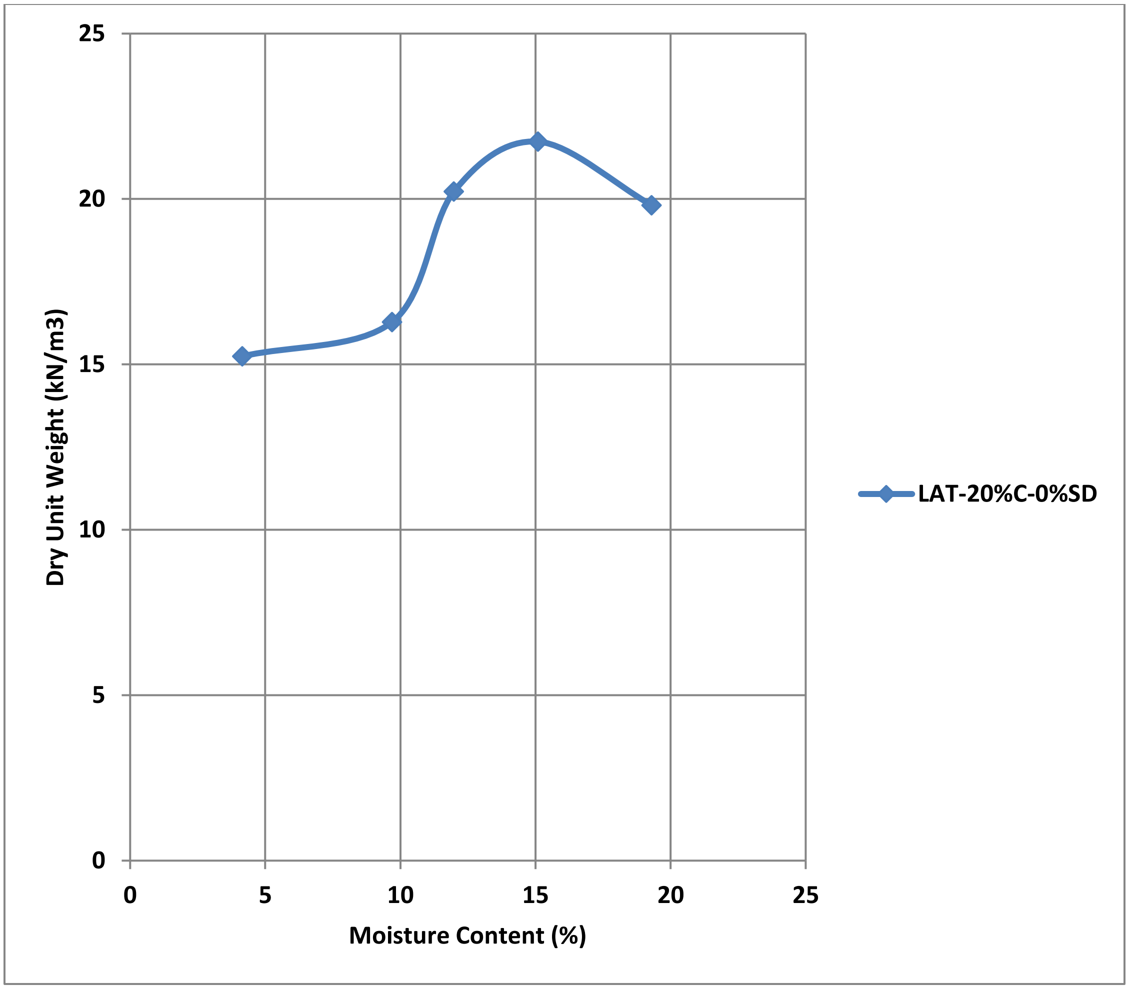

Figure E6: Compaction Curve for LAT + 20%C + 0%SD

Table E7: Dry Unit Weight Results for LAT + 0%C + 8%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.6	1.6	15.70	3.35	15.19
8	0.001	4	5.95	1.95	19.13	6.08	18.03
12	0.001	4	6.1	2.1	20.60	11.17	18.53
16	0.001	4	6.15	2.15	21.09	14.61	18.40
20	0.001	4	6.05	2.05	20.11	22.73	16.39

Table E7.1: Moisture Content Determination for LAT + 0%C + 8%SD (Top)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	14.71	28.34	13.63	27.95	13.24	0.39	2.95
8	15.03	40.04	25.01	39.04	24.01	1	4.16
12	14.31	46.22	31.91	43.06	28.75	3.16	10.99
16	17.82	65.65	47.83	59.54	41.72	6.11	14.65
20	13.56	63.34	49.78	52.78	39.22	10.56	26.93

Table E7.2: Moisture Content Determination for LAT + 0%C + 8%SD (Bottom)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	15.83	32.73	16.9	32.12	16.29	0.61	3.74
8	14.44	46.43	31.99	44.06	29.62	2.37	8.00
12	14.06	45.44	31.38	42.24	28.18	3.2	11.36
16	17.45	55.99	38.54	51.09	33.64	4.9	14.57
20	13.79	55.75	41.96	49.19	35.4	6.56	18.53

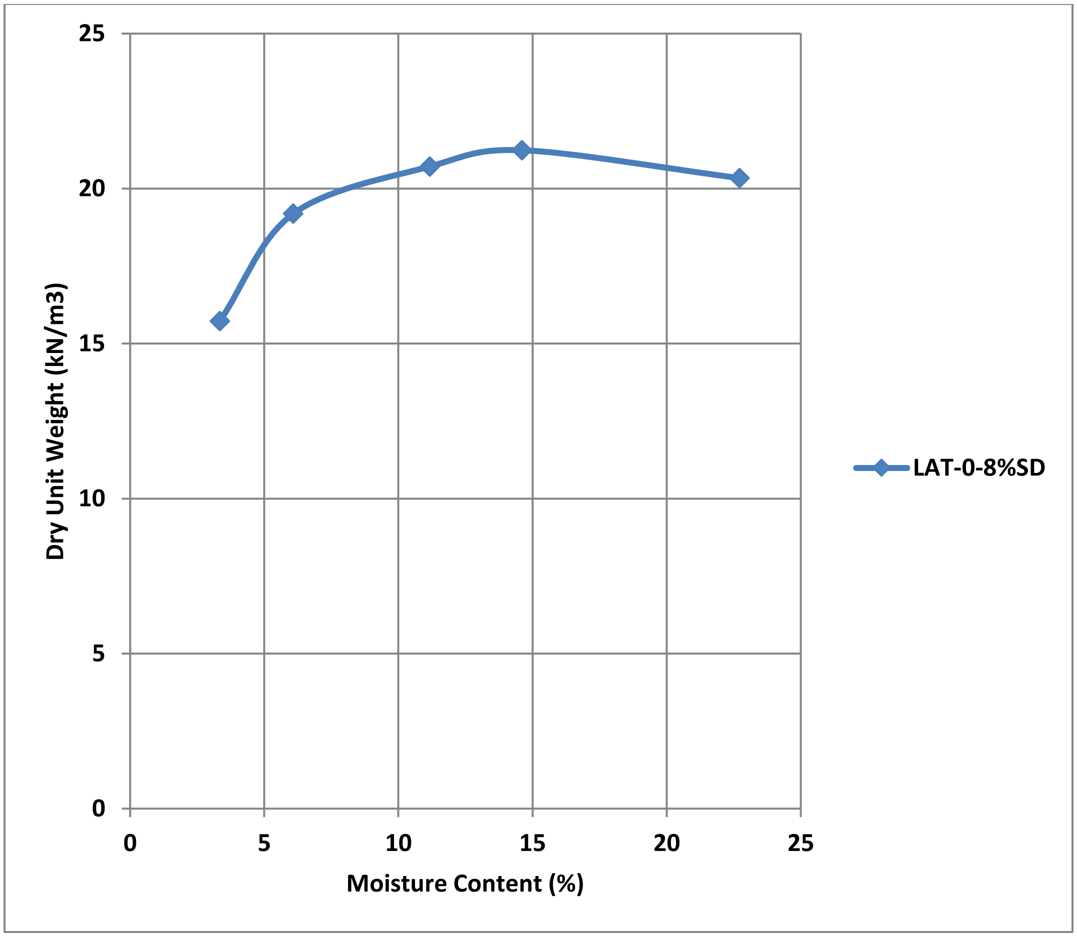

Figure E7: Compaction Curve for LAT + 0%C + 8%SD

Table E8: Dry Unit Weight Results for LAT + 0%C + 16%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.7	1.7	16.68	4.55	15.95
8	0.001	4	5.9	1.9	18.64	8.24	17.22
12	0.001	4	6.1	2.1	20.60	11.80	18.43
16	0.001	4	6.2	2.2	21.58	14.03	18.93
20	0.001	4	6.15	2.15	21.09	19.06	17.72

Table E8.1: Moisture Content Determination for LAT + 0%C + 16%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.53	30.36	15.83	29.67	15.14	0.69	4.56
8	14.04	39.51	25.47	37.68	23.64	1.83	7.74
12	14.54	43.27	28.73	40.24	25.7	3.03	11.79
16	14.28	48	33.72	43.97	29.69	4.03	13.57
20	14.82	52.36	37.54	46.37	31.55	5.99	18.99

Table E8.2: Moisture Content Determination for LAT + 0%C + 16%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.56	36.92	22.36	35.95	21.39	0.97	4.53
8	14.56	40.2	25.64	38.14	23.58	2.06	8.74
12	17.13	55.11	37.98	51.1	33.97	4.01	11.80
16	15.68	59.39	43.71	53.86	38.18	5.53	14.48
20	15.11	59.32	44.21	52.22	37.11	7.1	19.13

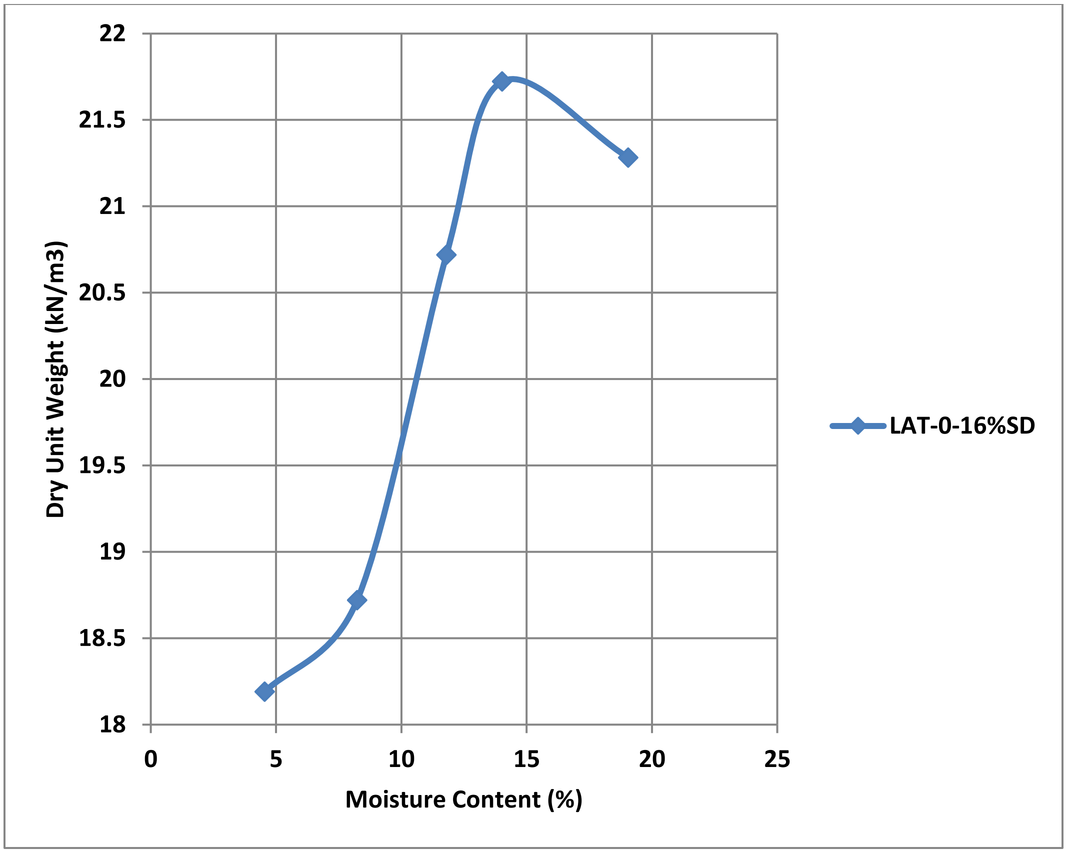

Figure E8: Compaction Curve for LAT + 0%C + 16%SD

Table E9: Dry Unit Weight Results for LAT + 0%C + 24%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.85	1.85	18.15	3.65	17.51
8	0.001	4	5.95	1.95	19.13	7.55	17.79
12	0.001	4	6.1	2.1	20.60	11.89	18.41
16	0.001	4	6.25	2.25	22.07	13.29	19.48
20	0.001	4	6.2	2.2	21.58	24.61	17.32

Table E9.1: Moisture Content Determination for LAT + 0%C + 24%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	16.65	34.58	17.93	33.88	17.23	0.7	4.06
8	15.88	47.69	31.81	44.95	29.07	2.74	9.43
12	15.3	51	35.7	47.94	32.64	3.06	9.38
16	14.18	70.95	56.77	66.5	52.32	4.45	8.51
20	15.08	65.43	50.35	54.86	39.78	10.57	26.57

Table E9.2: Moisture Content Determination for LAT + 0%C + 24%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.85	29.22	14.37	28.77	13.92	0.45	3.23
8	17.82	58.27	40.45	56.1	38.28	2.17	5.67
12	17.82	46.41	28.59	42.81	24.99	3.6	14.41
16	15.52	64.84	49.32	57.29	41.77	7.55	18.08
20	16.64	67.76	51.12	58.32	41.68	9.44	22.65

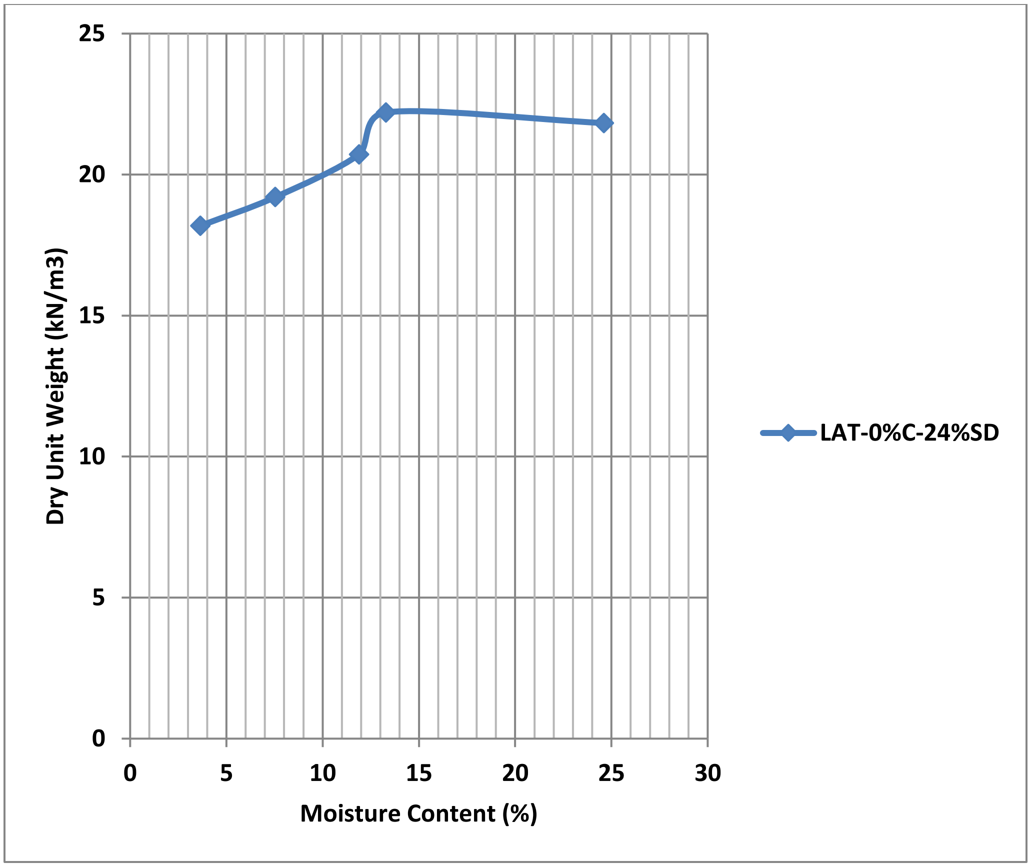

Figure E9: Compaction Curve for LAT + 0%C + 24%SD

Table E10: Dry Unit Weight Results for LAT + 0%C + 32%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.7	1.7	16.68	2.90	16.21
8	0.001	4	6.05	2.05	20.11	9.52	18.36
12	0.001	4	6.1	2.1	20.60	11.23	18.52
16	0.001	4	6.25	2.25	22.07	12.70	19.58
20	0.001	4	6.15	2.15	21.09	20.66	17.48

Table E10.1: Moisture Content Determination for LAT + 0%C + 32%SD (Top)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	15.57	37.16	21.59	36.71	21.14	0.45	2.13
8	15.36	35.15	19.79	33.41	18.05	1.74	9.64
12	14.48	41.96	27.48	38.99	24.51	2.97	12.12
16	13.6	53.16	39.56	50.45	36.85	2.71	7.35
20	16.7	58.92	42.22	51.67	34.97	7.25	20.73

Table E10.2: Moisture Content Determination for LAT + 0%C + 32%SD (Bottom)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	14.98	43.71	28.73	42.69	27.71	1.02	3.68
8	15.22	38.73	23.51	36.71	21.49	2.02	9.40
12	13.84	57.38	43.54	53.3	39.46	4.08	10.34
16	15.86	63.41	47.55	56.14	40.28	7.27	18.05
20	14.49	60.63	46.14	52.75	38.26	7.88	20.60

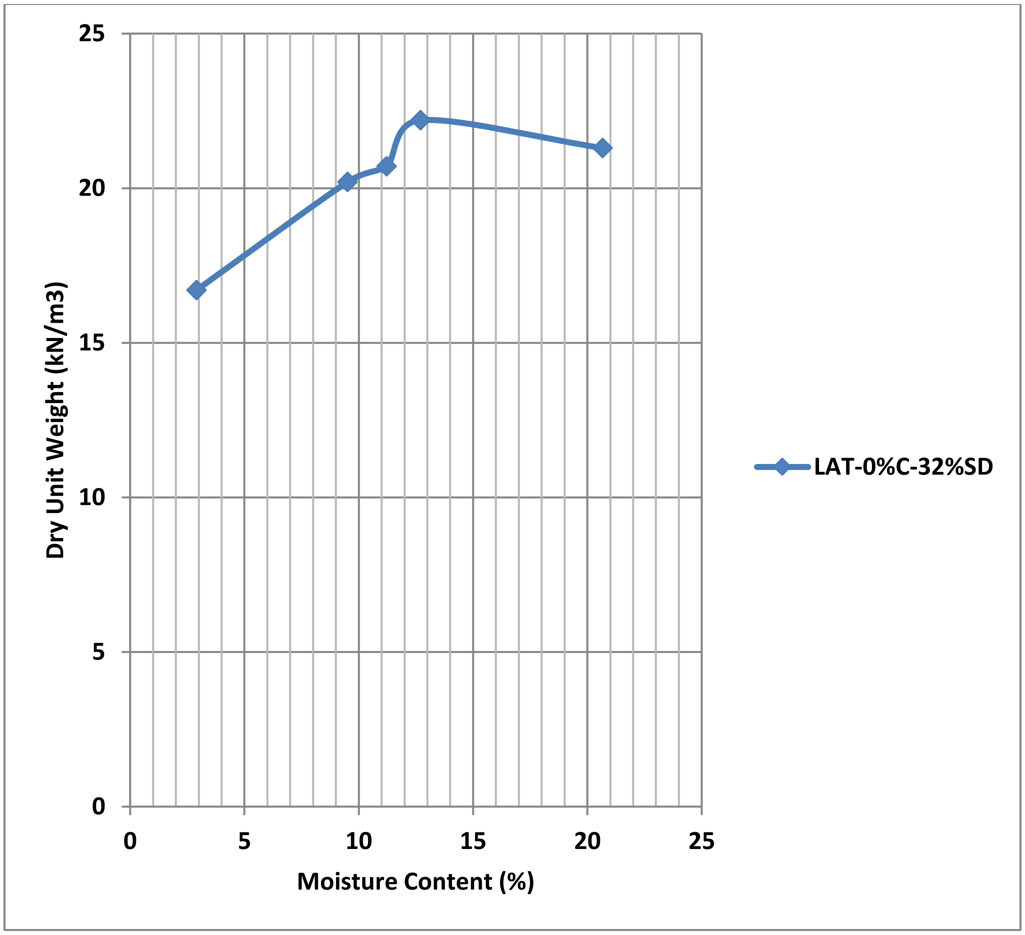

Figure E10: Compaction Curve for LAT + 0%C + 32%SD

Table E11: Dry Unit Weight Results for LAT + 0%C + 40%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.85	1.85	18.15	3.64	17.51
8	0.001	4	6	2	19.62	6.43	18.43
12	0.001	4	6.15	2.15	21.09	8.24	19.49
16	0.001	4	6.3	2.3	22.56	12.63	20.03
20	0.001	4	6.2	2.2	21.58	15.53	18.68

Table E11.1: Moisture Content Determination for LAT + 0%C + 40%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.7	31.13	15.43	30.67	14.97	0.46	3.07
8	14.07	38.5	24.43	37.04	22.97	1.46	6.36
12	17.79	47.27	29.48	45.65	27.86	1.62	5.81
16	14.36	56.93	42.57	52.29	37.93	4.64	12.23
20	15.07	62.54	47.47	55.95	40.88	6.59	16.12

Table E11.2: Moisture Content Determination for LAT + 0%C + 40%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	17.45	33.78	16.33	33.12	15.67	0.66	4.21
8	14.71	38.94	24.23	37.46	22.75	1.48	6.51
12	14.98	55.77	40.79	51.84	36.86	3.93	10.66
16	16.07	54.59	38.52	50.15	34.08	4.44	13.03
20	15.75	53.62	37.87	48.7	32.95	4.92	14.93

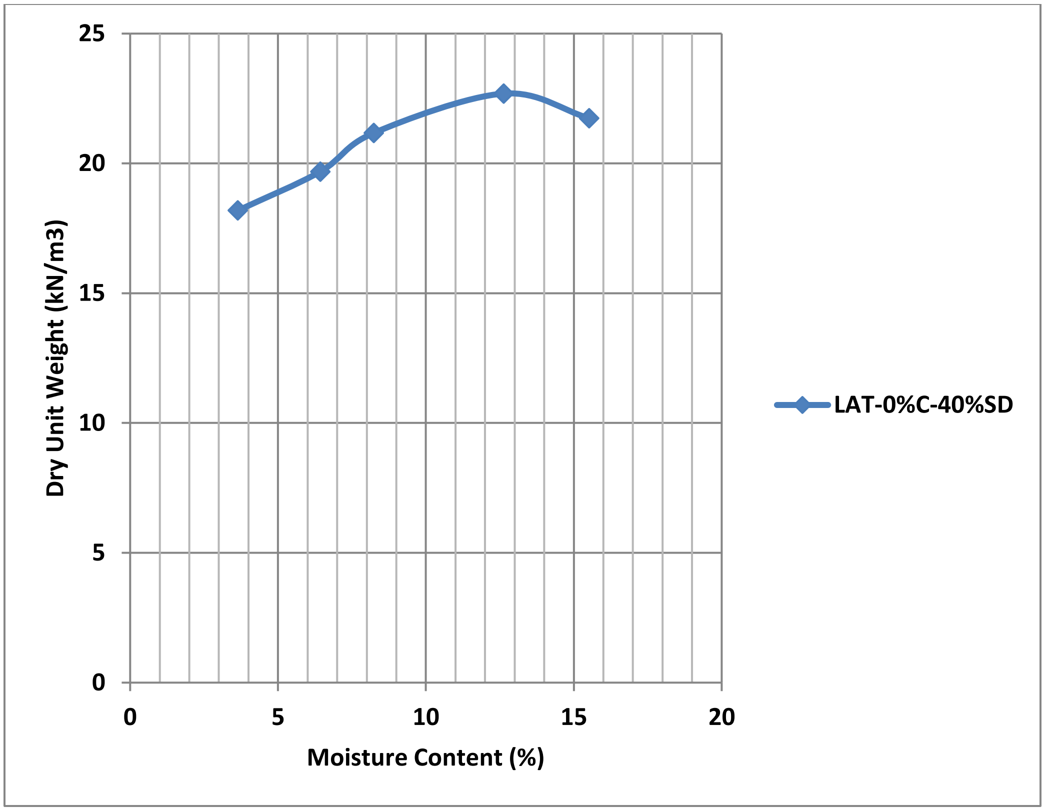

Figure E11: Compaction Curve for LAT + 0%C + 40%SD

Table E12: Dry Unit Weight Results for LAT + 4%C + 8%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.9	1.9	18.64	4.09	17.91
8	0.001	4	6.1	2.1	20.60	8.04	19.07
12	0.001	4	6.15	2.15	21.09	10.12	19.15
16	0.001	4	6.2	2.2	21.58	12.99	19.10
20	0.001	4	6.15	2.15	21.09	17.39	17.97

Table E12.1: Moisture Content Determination for LAT + 4%C + 8%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	13.36	32.56	19.2	31.85	18.49	0.71	3.84
8	14.21	43.22	29.01	41.27	27.06	1.95	7.21
12	15.65	46.71	31.06	44.24	28.59	2.47	8.64
16	16.23	38.9	22.67	36.59	20.36	2.31	11.35
20	14.42	56.68	42.26	50.84	36.42	5.84	16.04

Table E12.2: Moisture Content Determination for LAT + 4%C + 8%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.31	38.64	23.33	37.67	22.36	0.97	4.34
8	14.95	40.09	25.14	38.04	23.09	2.05	8.88
12	16.47	46.76	30.29	43.61	27.14	3.15	11.61
16	15.28	52.24	36.96	47.52	32.24	4.72	14.64
20	15.72	60.84	45.12	53.72	38	7.12	18.74

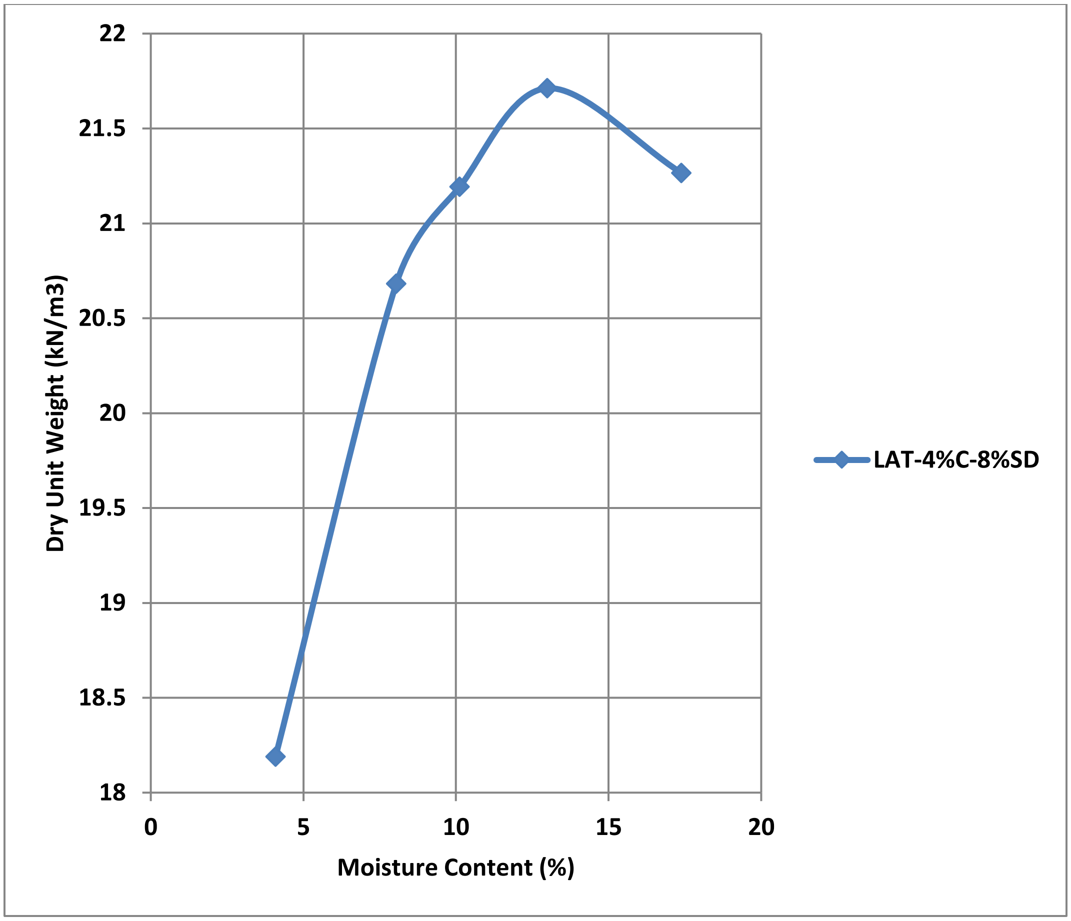

Figure E12: Compaction Curve for LAT + 4%C + 8%SD

Table E13: Dry Unit Weight Results for LAT + 8%C + 16%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	6	2	19.62	4.67	18.74
8	0.001	4	6.3	2.3	22.56	7.85	20.92
12	0.001	4	6.25	2.25	22.07	10.62	19.95
16	0.001	4	6.25	2.25	22.07	14.63	19.26
20	0.001	4	6.15	2.15	21.09	18.73	17.76

Table E13.1: Moisture Content Determination for LAT + 8%C + 16%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (%)
4	15.24	40.68	25.44	39.58	24.34	1.1	4.52
8	14.38	48.64	34.26	45.86	31.48	2.78	8.83
12	15.16	50.48	35.32	46.95	31.79	3.53	11.10
16	16.2	52.11	35.91	47.44	31.24	4.67	14.95
20	17.12	56.85	39.73	50.81	33.69	6.04	17.93

Table E13.2: Moisture Content Determination for LAT + 8%C + 16%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.88	41.36	26.48	40.14	25.26	1.22	4.83
8	15.64	47.88	32.24	45.81	30.17	2.07	6.86
12	16.39	45.28	28.89	42.62	26.23	2.66	10.14
16	12.86	50.42	37.56	45.72	32.86	4.7	14.30
20	15.12	61.28	46.16	53.74	38.62	7.54	19.52

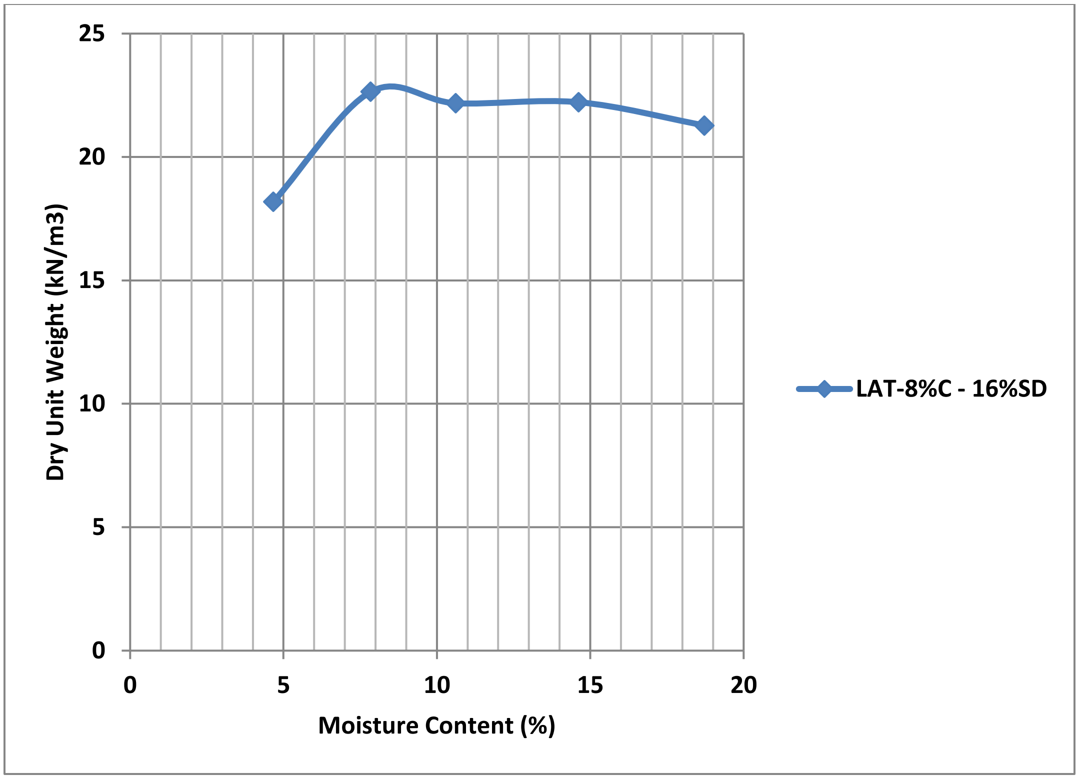

Figure E13: Compaction Curve for LAT + 8%C + 16%SD

Table E14: Dry Unit Weight Results for LAT + 12%C + 24%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	6.1	2.1	20.60	4.92	19.63
8	0.001	4	6.35	2.35	23.05	7.31	21.48
12	0.001	4	6.3	2.3	22.56	11.89	20.17
16	0.001	4	6.25	2.25	22.07	15.27	19.15
20	0.001	4	6.2	2.2	21.58	19.96	17.99

Table E14.1: Moisture Content Determination for LAT + 12%C + 24%SD (Top)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	15.24	43.76	28.52	42.65	27.41	1.11	4.05
8	14.92	46.84	31.92	44.76	29.84	2.08	6.97
12	13.36	49.28	35.92	45.75	32.39	3.53	10.90
16	15.21	53.74	38.53	48.96	33.75	4.78	14.16
20	17.44	55.94	38.5	49.44	32	6.5	20.31

Table E14.2: Moisture Content Determination for LAT + 12%C + 24%SD (Bottom)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	16.42	42.15	25.73	40.74	24.32	1.41	5.80
8	14.54	48.74	34.2	46.31	31.77	2.43	7.65
12	16.28	50.12	33.84	46.26	29.98	3.86	12.88
16	13.58	58.42	44.84	52.11	38.53	6.31	16.38
20	15.69	63.26	47.57	55.46	39.77	7.8	19.61

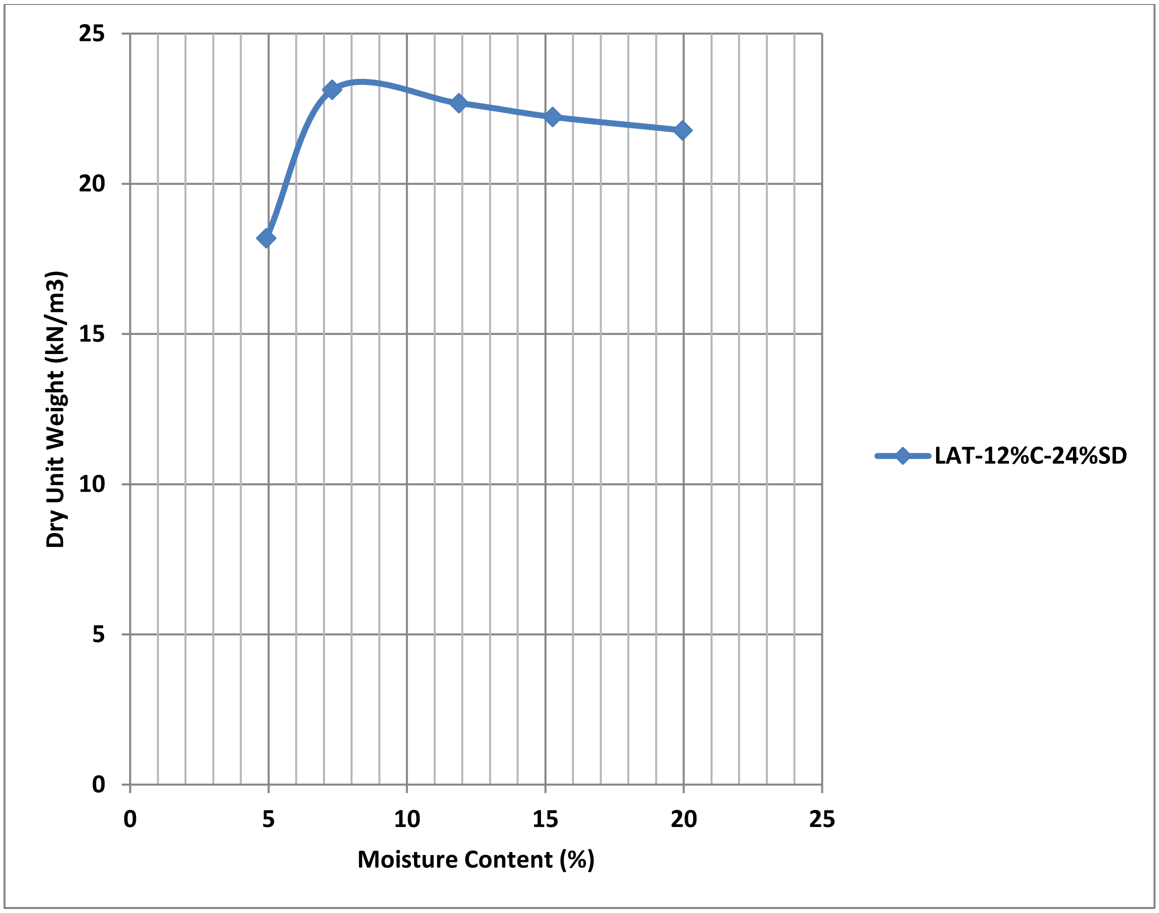

Figure E14: Compaction Curve for LAT + 12%C + 24%SD

Table E15: Dry Unit Weight Results for LAT + 16%C + 32%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.95	1.95	19.13	4.64	18.28
8	0.001	4	6.2	2.2	21.58	8.33	19.92
12	0.001	4	6.3	2.3	22.56	11.54	20.23
16	0.001	4	6.35	2.35	23.05	14.69	20.10
20	0.001	4	6.2	2.2	21.58	20.61	17.89

Table E15.1: Moisture Content Determination for LAT + 16%C + 32%SD (Top)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	14.48	44.58	30.1	43.37	28.89	1.21	4.19
8	17.12	51.93	34.81	49.25	32.13	2.68	8.34
12	15.38	46.88	31.5	43.66	28.28	3.22	11.39
16	14.45	48.36	33.91	44.76	30.31	3.6	11.88
20	15.12	54.28	39.16	47.45	32.33	6.83	21.13

Table E15.2: Moisture Content Determination for LAT + 16%C + 32%SD (Bottom)

Percentages of Water (%)	Wt of tin (g)	Wt of tin + wet Soil (g)	Wt of wet Soil (g)	Wt of tin + dry Soil (g)	Wt of dry Soil (g)	Wt of Water (g)	Moisture Content (g)
4	13.48	45.09	31.61	43.56	30.08	1.53	5.09
8	14.52	52.63	38.11	49.7	35.18	2.93	8.33
12	15.16	55.76	40.6	51.51	36.35	4.25	11.69
16	16.68	48.29	31.61	43.58	26.9	4.71	17.51
20	15.12	64.92	49.8	56.59	41.47	8.33	20.09

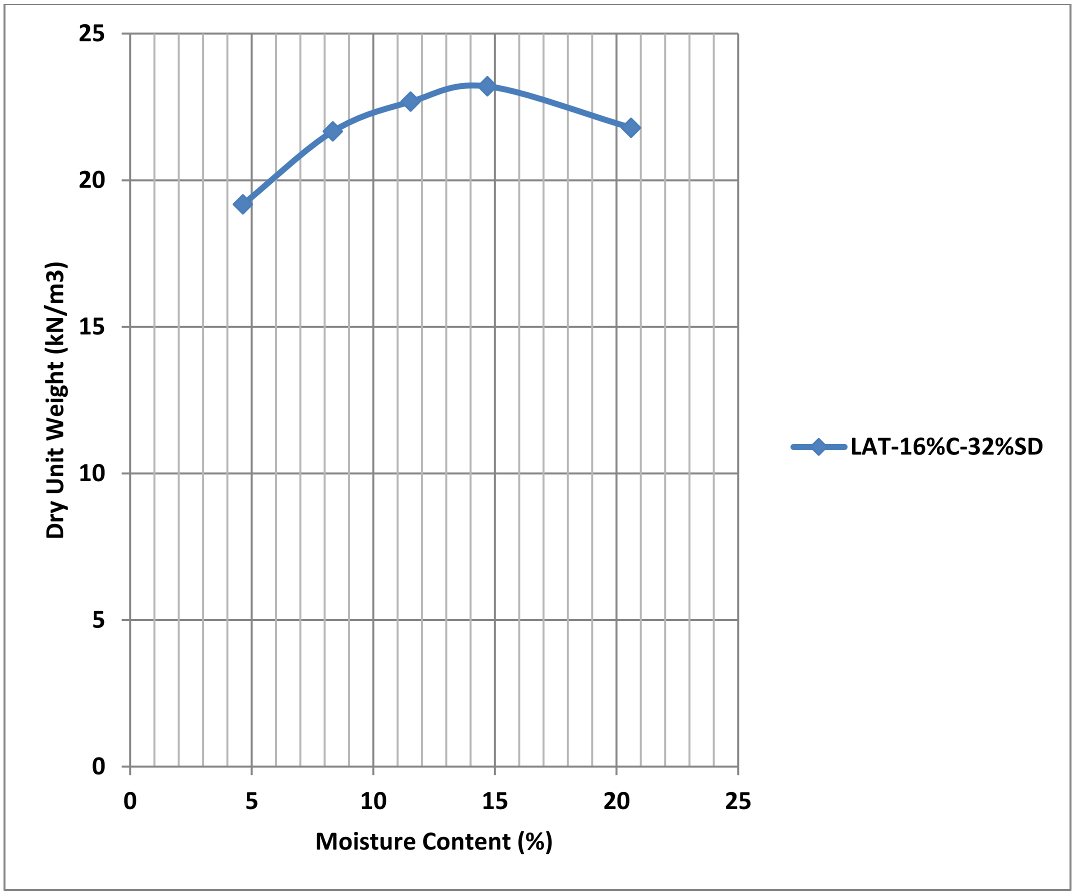

Table E15: Dry Unit Weight Results for LAT + 16%C + 32%SD

Table E16: Dry Unit Weight Results for LAT + 20%C + 40%SD

Percentages of Water	Vol of Mould	Wt of Mould	Wt of Mould + Wet Soil	Wt of Wet Soil	Bulk Density	Moisture Content	Dry Unit Weight
(%)	(m^3)	(kg)	(kg)	(kg)	(kN/m^3)	(%)	(kN/m^3)
4	0.001	4	5.9	1.9	18.64	3.56	18.00
8	0.001	4	6.1	2.1	20.60	9.95	18.74
12	0.001	4	6.25	2.25	22.07	10.57	19.96
16	0.001	4	6.35	2.35	23.05	13.73	20.27
20	0.001	4	6.3	2.3	22.56	18.99	18.96

Table E16.1: Moisture Content Determination for LAT + 20%C + 40%SD (Top)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	15.21	46.38	31.17	45.93	30.72	0.45	1.46
8	16.84	52.28	35.44	49.15	32.31	3.13	9.69
12	14.58	48.64	34.06	45.26	30.68	3.38	11.02
16	14.38	50.52	36.14	45.69	31.31	4.83	15.43
20	15.12	52.18	37.06	46.05	30.93	6.13	19.82

Table E16.2: Moisture Content Determination for LAT + 20%C + 40%SD (Bottom)

Percentages of Water	Wt of tin	Wt of tin + wet	Wt of wet Soil (g)	Wt of tin + dry Soil	Wt of dry	Wt of Water (g)	Moisture
(%)	(g)	Soil (g)		(g)	Soil (g)		Content (g)
4	14.68	50.56	35.88	48.64	33.96	1.92	5.65
8	14.52	56.42	41.9	52.54	38.02	3.88	10.21
12	15.43	55.38	39.95	51.71	36.28	3.67	10.12
16	16.62	44.92	28.3	41.88	25.26	3.04	12.03
20	14.88	67.12	52.24	59.09	44.21	8.03	18.16

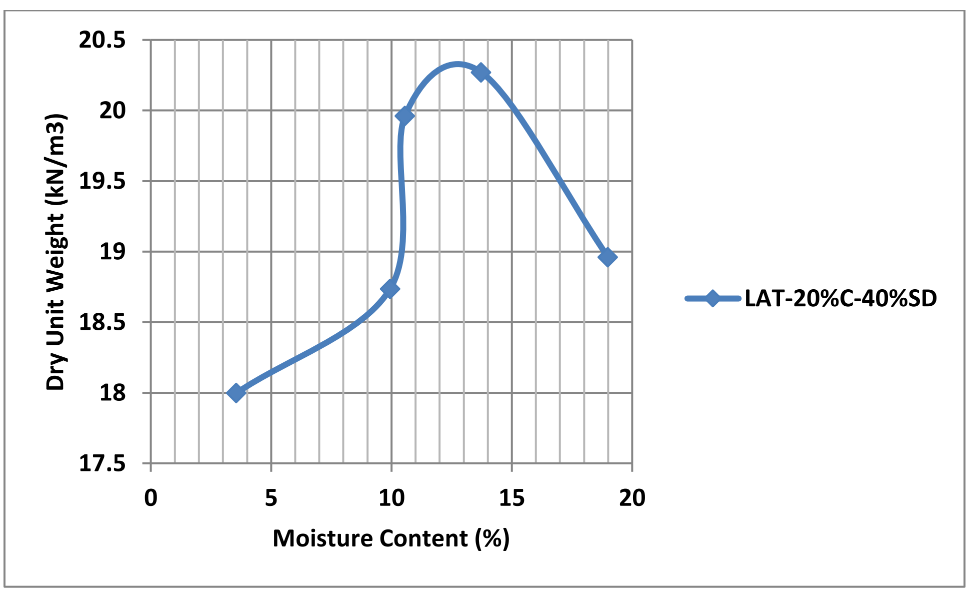

Figure E16: Dry Unit Weight Results for LAT + 20%C + 40%SD