STABILIZATION OF LATERITE SOIL USING ORANGE PEELS AT FACULTY OF ENGINEERING UNIVERSITY OF BENIN CITY, EDO STATE

UGBOWO CAMPUS

By

BASSEY RAJUNOR SUNDAY,

ENG1603942

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CERTIFICATION

This is to certify that this work was carried out by BASSEY RAJUNOR SUNDAY with Matriculation Number, ENG1603942, of the Department of Civil Engineering, Faculty of Engineering, University of Benin, Benin City, Edo State, Nigeria.

__________________________ __________________________

ENGR. E.E. ORIA-USIFO **Date**

(Project Coordinator)

__________________________ __________________________

ENGR. J.E. EKHODIAEHI **Date**

(Project Supervisor)

__________________________ __________________________

ENGR. DR. S.D. IYEKE **Date**

(Head of Department)

DEDICATION

To Jehovah God Almighty.

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ABSTRACT

Pollution is a very predominant problem in Nigeria. As a developing nation, we face challenges of constant economic recession and this can have effect on so many facets of life including construction. Road construction is now so expensive. In other to combat pollution and bad roads, waste can be used with other construction materials to construct better roads.

In this study, the geotechnical properties of natural soil were determined, the effect of mixture of orange peels in proportions of 2%, 4%, 6% and 8% were observed and the appropriate mix ratio was ascertained. The soil samples were obtained from Faculty of Engineering, University of Benin, Benin City, Edo State, from two locations. The following tests were carried out; Moisture content Test, Specific Gravity Test, Sieve analysis, Atterberg Limit tests, Compaction tests, California Bearing Ratio tests and on the treated and untreated laterite soil. This laboratory tests were carried out to determine the suitability of these waste materials when mixed with the soil samples.

For the control sample, the specific gravity was obtained to be 2.69 and 2.73 which is above 2.50 but not greater than 3.0 for lateritic soils, this shows a high plasticity. The plasticity index was obtained as 39.49% which indicates that the soil is not medium in plasticity. Also, from the sieve analysis, the soil is classified as an A-2-6 soil according to AASTHO. This value indicates that it is good for subgrade material. Therefore, these soil reinforcement techniques with orange peel can only be used for Trunk D roads or as subgrade materials which require a minimum soaked CBR of 5%. For Trunk A, B and C roads, the mixture would require a binder like cement before being considered as subbase and base course materials

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LIST OF SYMBOLS

AASHTO - American Association of State Highway and Transportation Officials

ASTM - American Society for Testing and Materials

a.u. - Arbitrary unit

CBR - California Bearing Ratio

OMC - Optimum Moisture Content

MDD - Maximum dry density

DD - Dry Density

GS - Specific Gravity

LL - Liquid Limit

PL - Plastic Limit

LS1 - Laterite Sample 1

LS2 - Laterite Sample 2

CHAPTER ONE

INTRODUCTION

1.1 Background of study

Laterite soils are considered sustainable when used as a construction material because they allow current needs to be met without compromising the potential of future generations. These materials are eco-friendly and beneficial. The demand for laterite inclusion in earlier and more contemporary projects was prompted by the high cost of construction projects. Since earth elements are freely available practically everywhere on the planet, buildings made of them are the most popular economical housing. A category of extremely weathered soils known as laterite are created when hydrated oxides of iron and aluminum are concentrated in one place (Thagesen,1996). Other definitions have made use of the ratio of sesquioxides (Fe_2O_3 + Al_2O_3) and silica (SiO2) in lateritic soil when the ratios are less than 1.33. Having laterite. The most well-known and often used building and road construction material is laterite. Lateritic soils are utilized to build roads in tropical regions of the world, and they serve as the subgrade for the majority of tropical roadways. They serve as sub-bases and bases for inexpensive roadways that handle light to moderate traffic. Additionally, they are employed in Nigeria's rural areas as a construction material for block molding and plastering (Onyelowe, 2016).

Soil stabilization is the process of modifying soil by changing one or more of its physical or chemical properties to produce soil with improved and desired engineering properties. The American Society for Testing and Materials (ASTM) states that improving a soil's permeability, strengthening an existing soil to increase its load-bearing capacity, and strengthening a soil's resilience to weathering and traffic utilization are among the key goals of soil stabilization (ASTM, 2014).; Soil stabilization is essential for building long-lasting buildings out of easily available earth components because it improves mechanical characteristics, particle cohesiveness that minimizes porosity and variations in volume owing to moisture fluctuations, and resistance to rain, wind, and erosion. Soil may be stabilized in a number of ways, including by mechanical, physical, and chemical methods. Societies prioritize certain construction materials based on factors like cost and availability (Sergio,2008). Owning a home is generally challenging due to the high cost of construction materials in most developing nations. To address this issue, innovative low-cost methods of creating construction materials from locally accessible materials must be investigated. Laterite soil is composed of highly malleable clay, which may lead to mice cracks and other problems during the construction of high flyovers, pavement, and other surfaces. As a result, it is crucial to learn about laterite soil's characteristics and identify its best way of stabilization. Researchers in this study employed orange peels to act as laterite stabilizers. Accordingly, the purpose of this investigation is to contrast the performance of laterite soil that has been treated with orange peels to that of laterite soil that has not been treated.

1.2 Statement of Problem

The rising cost of cement and other traditional building materials has increased demand for environmentally friendly and economically feasible construction projects in Nigeria and other developing nations. Using readily available, inexpensive, and environmentally friendly resources from the surrounding area is crucial to green building practices. With a few caveats, laterite, a locally accessible material, seems to be a superior option to traditional construction supplies. Due to its high plastic clay content, laterite soil has to be stabilized to prevent cracking and damage in building foundations, paved roadways, and other construction projects. Pollution poses a major risk to human survival in the long term. Orange peels are a common sight in the marketplaces and streets of Nigeria, and they may be a major annoyance for shoppers and passers-by both. Therefore, in this experiment, orange peels were used to help stabilize laterite materials.

1.3 Aims and Objectives

1.3.1 Aim of Study

This research aims to study the behavior peels treated with laterite soil in

Comparison with untreated laterite soil.

1.3.2 Objectives

The objectives are as follows:

1. Identification of laterite
2. To determine an appropriate mix proportion of stabilized laterite soil using orange peels as a stabilizing agent.
3. To study the effect of orange peels when mixed with the laterite specimen.
4. Setting up an experiment to use different levels of the selected stabilization materials (orange peels) to develop stabilized laterite.
5. Comparison of the stabilized soils to the raw sample collected.

1.4 Scope of Work

This study examines the laterite soils used by the University of Benin's Department of Engineering in Benin City, Edo State. On both the treated and untreated laterite soil, a number of tests, including moisture content, specific gravity, sieve analysis, Atterberg Limit, compaction, and California Bearing Ratio, must be conducted.

1.5 Justification of Study

Laterite stabilization eliminates future issues like swelling and damping that might cause the structure created with untreated laterite to fail. It also helps roads and buildings constructed with laterite live longer, which reduces the need for costly repairs. Stabilizing laterite helps make it stronger and last longer by decreasing its permeability. As a cement alternative for the readily accessible laterite components, orange peels were used to address environmental concerns. Earth materials (laterite) stabilized with orange peels might be used to construct low-cost houses and long-lasting roadways, as shown by this research. This study will only apply its findings to laterite specimens from lateritic soils in the Faculty of Engineering at the University of Benin in Benin City, Edo State, or to other laterite or soil samples with similar characteristics due to the wide variation in lateritic formations and mineral constituents.

CHAPTER 2

LITERATURE REVIEW

2.1 DEFINITION AND FORMATION OF LATERITE SOIL

From the Latin word "later," which means "brick," comes laterite. Conditions like high temperatures and a lot of rain contribute to its formation. The weathering of tropical rocks, which is extensive and long-lasting, and is accelerated by heavy rains and high temperatures, results in lateral soil (Schellmann 1983). In the process of leaching the underlying sedimentary, metamorphic, igneous, or mineralized proto-ore, laterites are created when the more insoluble ions, mainly iron and aluminum, are retained. In a humid subtropical monsoon environment, the hydrolysis and precipitation of insoluble oxides and sulfates of iron, aluminum, and silica occurs at high temperatures after the host mineral lattice has been dissolved by acid.

The repeating of wet and dry seasons is a crucial component in the creation of laterite. During the rainy season, percolating rainwater leaches rocks; during the dry season, capillary action brings the solution containing the leached ions to the surface. These ions cause the formation of water-soluble salt compounds near the surface, which are then washed away in the subsequent rainy season. Gentle crests and plateaus with little topographic relief are ideal for laterite formation because they prevent the eroding of the surface cover. Slowly but surely, as the water table rises, the sodium, potassium, calcium, and magnesium ions that are so readily leached out of the reaction zone are gone. These are crystalline in structure and contain very little clay compared to other sandstones. Less vegetation thrives there; grass is the sole major crop. Very worn residual soils found in the tropics or subtropics are what we call "lateritic soil."

It is frequently stated that the kind of soil relies on the type of rock, its mineral components, and the local climate conditions (Surendra, Sanjeev, 2017). Laterite therefore contains the elements iron, aluminum, lime, silica, nitrogen, and phosphorous. However, it is deficient in lime, silica, nitrogen, and phosphorus and abundant in iron and aluminum. It is acidic in nature and has a reddish to yellowish hue. Lateritic soil is one of the most crucial soils in construction because of factors including its distinctive color, low fertility, and high clay content (T-Zu-Hsing 2014). Studies have revealed that a laterite's strength and stability cannot always be assured when it contains a significant proportion of clay minerals. Similar laterites may be found in many tropical locations, such as Nigeria, where it may be more cost-effective to make improvements to the soil that already exists in order to get the desired result (Mustapha 2005).

All around the tropics, you may find lateritic soils. Determining whether an individual is accountable for his or her behavior is what is meant by the phrase "responsibility." The soil qualities in the vicinity of building projects may not always be up to code. Therefore, there is a need to enhance the characteristics of the current materials. (2008) Osinubi et al. One of the most prevalent building materials for engineers is lateritic soil. Kobe (1975) characterizes laterites as having three distinct layers—a layer of lateritic clay at the bottom, a layer of lateritic gravel in the middle, and a crust on top. Therefore, the following are examples of laterites:

I Laterite crust: the cellular nature of laterite crust makes it very challenging for a geologist to crack it with a hammer. This laterite may need the use of low explosives during excavation. It may be found on the tops of relatively level hills or as boulders on sloping slopes, both of which are popular places to find it when constructing construction foundations.

(ii) Laterite gravel: A layer of laterite crust covers a layer of laterite gravel. There are spots where the gravel deposit is only partly buried. It is usual to find pisolitic laterite gravel.

(iii) Laterite Clay: The clay layer of laterite is often found below the gravel or crust and above the worn foundation. It's mostly a deep reddish brown, with hints of pinkish white here and there (probably Kaolinite). Mica flakes are easily seen in hand specimens. As such, earthen dams are a common use.

2.2 Occurrence of Laterite

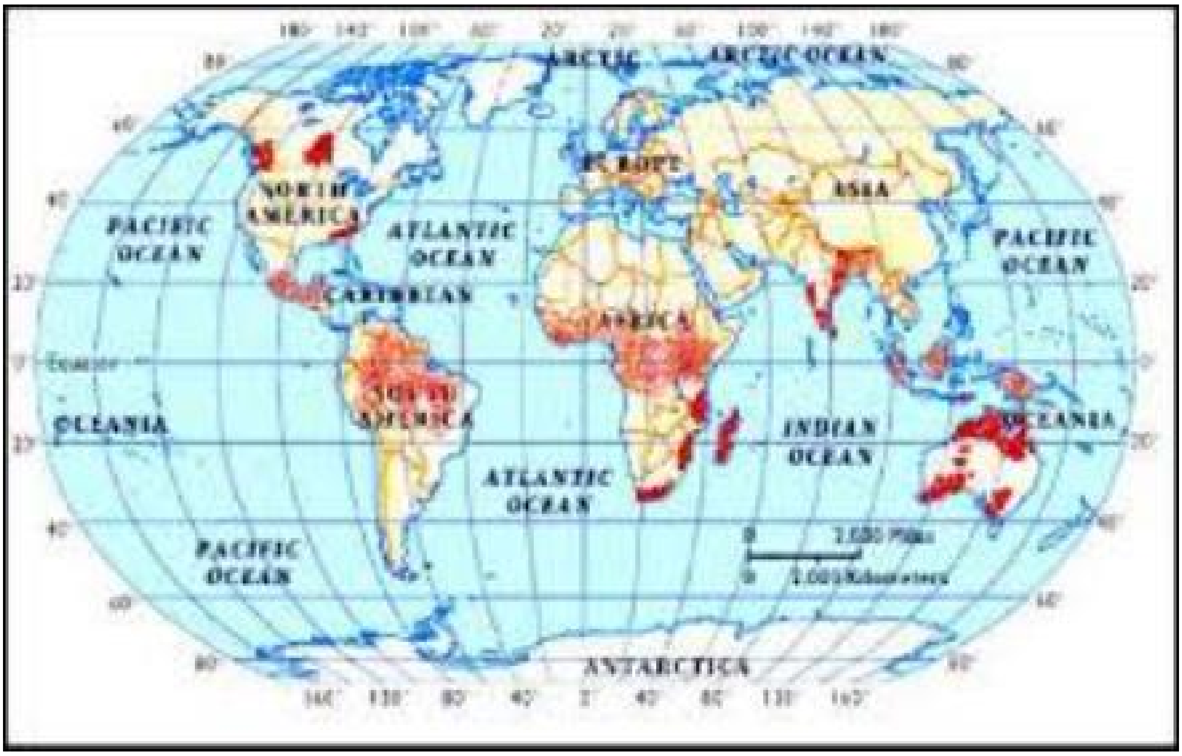

Figure 1 depicts the global distribution of laterite (Pearson, 1970)

Inconsistencies in nomenclature and information transmission provide a significant barrier to creating a unified database of laterite research from across the globe. There are several distinct names for laterite, both internationally and domestically. Therefore, it is crucial to be familiar with the numerous terms used to refer to it throughout the world. The tropics and subtropics of Africa, Australia, India, Southeast Asia, and South America all have laterite and related soils. The first worldwide synthesis of laterite distribution, shown in Figure 1 below, was compiled by Prescott and Pendleton in 1952. "(Pearson, 1970)"

2.3 SOIL STABILIZATION

Soil that requires many operations to bring it up to par is called "problematic" soil since it does not currently match the requisite standard. Soil stabilization is what happens when this happens. The term "soil stabilization" refers to the practice of enhancing soil's engineering qualities to make it more suited for engineering applications. This is essential when the readily accessible dirt is unsuitable for the project at hand. Physical or physiochemical methods are used in soil stabilization, which increases the soil's bearing capacity. This is essential when the readily accessible soil is unsuitable for the planned use. In order to increase the soil's carrying capacity, it must be stabilized, which may be done either physically or physiochemically. Sometimes, existing or naturally occurring soils are found to be exceedingly weak and need reinforcement in order to fulfill engineering requirements. Understanding the soil's qualities is the first step in stabilization since that's the only way to know how much can be accomplished with the soil. Soil qualities may be determined in a few different ways: by in-situ testing during field research, through laboratory testing, and through back analysis using data from the site's performance. Regardless of the strategies you choose to stabilize the soil, knowing its starting condition is essential. Engineering features of soil include: specific gravity; consistency limitations; density index; particle size analysis; compaction; consolidation; permeability; and compaction; (Sureand ndra, Sanjeev, 2017). Stabilization efforts add to the soil's long-term, permanent strength. Soil stabilization techniques improve the soil's resistance to shear and lateral stresses, making it less likely to liquefy or shift underweight. Examples of stabilizing agents include lime, fly ash, afly ash cement, and other similar substances.

The term "soil stabilization" refers to the process of enhancing the bearing capacity of soil by raising its shear strength characteristics. The earth cannot support building due to its poor bearing capacity. The term "soil stabilization" refers to the practice of altering soils in order to enhance their physical qualities.

By boosting its shear strength and/or regulating its shrink-sw characteristics, stabilization may increase a sub-load-bearing grade's capacity, making it better able to sustain pavements and foundations. The purpose of soil stabilization is to raise the shear strength and decrease the permeability of the soil mass in earthen construction (Habiba, 2017). It is necessary when the available soil cannot support the weight of the building. In order to improve their engineering performance, soils are often modified via a process known as stabilization. The load-bearing capacity of a sub-grade for supporting pavements and foundations may be enhanced by stabilization by increasing the soil's shear strength and/or controlling the soil's shrink-swell qualities.

By incorporating and blending in additional components, soil stabilization enhances the soil's inherent qualities. The purpose of soil stabilization is to enhance geotechnical qualities including compressibility, strength, permeability, and durability by incorporating stabilizing agents (binder materials) into weak soils. It's a strategy for boosting soil resilience in the face of severe moisture and stress circumstances (Gidigasu 1976). This term is used to describe the process of incorporating other materials into the parent soil, such as more soil, cement, lime, bituminous products, silicates, and other chemicals and natural or manufactured, organic, and inorganic elements (Osinubi et.al, 2014). Soil may be made more stable by adding lime, which raises the pH and allows the clay's native silica and alumina to be dissolved. Calcium-silicate-hydrates (CSH) and calcium-aluminate-hydrates (CAH) are formed when silica and aluminum oxide react with lime and water in the soil (CAH). The interactions between CSH and CAH in cement are identical. They create linkages that last forever and greatly increase the soil's bearing capacity. As the clay concentration of the soil declines, the byproducts of the lime producing process may be useful. From the lime-making process to the soil mix, these minerals contribute pozzolanic qualities, which may facilitate the formation of strong connections via chemical reactions. When natural pozzolans, found in clay particles, are unavailable, Portland cement may also be used to stabilize soils. In order to strengthen soil, cement interacts with the water present and hydrates, much as it does in concrete.

2.4 REASONS FOR STABILIZATION

1. It is used to increase the shear strength of the soil
2. It is used to enhance the stability of slopes in soil
3. It also increases the strength of soil against deformation and displacement under loads
4. Soil stabilization helps to improve the workability of the soil
5. Soil stabilization helps to enhance the bearing capacity of soil
6. It helps to reduce permeability and compressibility of soil

2.5 SOIL STABILIZATION METHODS

Road building projects rely heavily on soil, since it is the primary component of the pavement layers. Soil used in the construction of pavement must fulfill the requisite standard in order to serve its intended function. Soil stabilization involves combining natural resources with man-made substances like lime, cement, fly ash, or bitumen. Strength, permeability, and compressibility are all improved upon with stabilized soil materials. There are two approaches to implement the strategy:

1. In situ stabilization
2. Ex - situ stabilization.

Engineers need soils to have certain characteristics in order to build on them successfully. These characteristics include volume stability, strength, compressibility, permeability, and durability (Sherwood, 1993). Examples of stabilizing methods include;

1. Mechanical Stabilization

2. Stabilization by using different types admixers;

1. Lime Stabilization
2. Bituminous Stabilization
3. Chemical Stabilization
4. Fly ash Stabilization
5. bone ash as a soil stabilizer
6. Orange peel as a soil stabilizer
7. Rice Husk ash Stabilization
8. Cement Stabilization
9. Thermal Stabilization
10. Electrical Stabilization
11. Stabilization by Geo-textile and Fabrics
12. Recycled and Waste Products etc.

2.5.1 CEMENT STABILIZATION

For cement stabilization, crushed earth is combined with Portland cement, water, and compacted to create a more stable substance. Normal soil-cement, Plastic soil-cement, and Cement-modified soil are all examples of cement stabilization. Soil type, cement amount, water quantity, mixing, compaction, curing, and admixtures are only few of the variables that might impact the success of cement stabilization. It may be accomplished with the help of both conventional and unconventional means (Habiba, 2017). The distinction between the two groups is due to the fact that the older, more established additives have been used for longer than the newer, more cutting-edge compounds. Common calcium-based stabilizing agents include lime, cement, bitumen, and fly ash. They undergo both rapid and slow chemical changes in response to water, which improve the soil matrix in terms of reduced swelling, increased shear strength, and resistance to the effects of wetting and drying. Cation exchange, flocculation, agglomeration, pozzolanic reaction, and carbonate cementation are the stabilizing processes often associated with conventional stabilizers. Soil physicochemical interactions result from the chemical reactions of non-traditional substances in the presence of adequate moisture. Bitumen emulsions, cement kiln dust, powdered granulated blast furnace slag, pulverized coal bottom ash, steel slag, mine tailings, sulphonated oils, and polymers are just few of the many examples. They are accomplished by the use of a number of compounds that serve as binders, water repellents, and/or compaction aids. These chemicals are often diluted with water before being sprayed into soil, which is then mixed and compacted.

Most people agree that cement stabilization is the gold standard in soil binding technology. Cement is excellent for stabilizing pavement, but it can't be used in places where there's a lot of clay in the soil or a lot of organic matter, so it has certain restrictions. Soils with a high concentration of clay are particularly vulnerable to having their soil-cement structure disrupted by the presence of salts like sulfate, thus it's important that these be kept to a minimum. Conversely, the process only works with a certain clay concentration. Biological matter of any kind is strictly forbidden. Complex application procedure. High-stakes procedures for determining the ideal timing for compaction and water-content regulation are required. From both a financial and ecological standpoint, cement manufacturing is a major energy hog.

Hydration of the cement particles causes them to develop into crystals with the ability to interlock with one another, resulting in high compressive strength, a process known as cement stabilization. Cement particles must cover most of the material particles for a strong connection to form. Likewise, the cement and soil must be mixed with a certain particle size distribution in order to promote excellent contact between soil particles and cement and successful soil cement stability. Highly compacted soil-cement, also known as cement-stabilized base or cement-treated aggregate foundation, consists of soil or aggregate, cement, and water. Because of the cement's hydration and subsequent strength development, soil-cement may be used as a long-lasting building material. Cement stabilization is complete when compaction is sustained; this is because cement fills the vacuum between soil particles, hence decreasing the void ratio of the soil. Subsequently, when water is added to the soil, the cement interacts with the water and hardens, increasing the unit weight of the soil. Cement hardening increases shear strength and bearing capacity as well. In cement reactions, the primary function is played by the cement's interaction with water, which may be present in any soil. Cement comes in a variety of varieties and uses, including Portland cement, blast furnace cement, sulfate resistant cement, and high alumina cement. Cement is often selected based on the soil type being worked with and the desired end strength. The cement reaction, known as the hydration process, begins with the addition of water and any other necessary ingredients to the dry cement. This causes the cement to harden and may be used in a variety of construction and engineering applications. Once cement has cured, it will surround dirt like glue, but it won't alter the earth's underlying structure. Since the hydration process proceeds slowly from the cement grain surfaces, the cement grains' centers may remain unhydrated (Shearwood, 1993). Depending on the factors involved, the ultimate effect on setting and gain in strength of cement stabilized soil may vary. Cement hydration is a complex process with a complex series of unknown chemical reactions (Hicks, 2002), but this process can be affected by the presence of foreign matters, water-cement ratio, curing temperature, presence of additives, specific surface of the mixture. Therefore, this should be considered during mix design to attain the appropriate strength. C3S and C2S calcium silicates are the primary cementitious characteristics in conventional Portland cement that contribute to strength development (MacLaren and White, 2003). Typically, just a modest quantity of cement is needed to boost the soil's engineering qualities and enhance the cation exchange of clay. The following are some of the enhanced qualities of cement-stabilized soils:

1. Decreased cohesiveness (Plasticity)
2. Decreased volume expansion or compressibility
3. Increased strength.

2.5.2 MECHANICAL STABILIZATION

Mechanical stabilization is a kind of stabilization that uses physical processes, such as compaction, soil mixing, or the installation of a barrier, to alter the physical qualities of the soil. To apply mechanical stabilization, layers of various materials are layered on top of one another until a compact substance is achieved, and binders are added to non-cohesive soils in minute quantities. What determines a soil's mechanical stability is its component materials' mechanical strength and purity, Mixture composition and ingredient proportions, how well dirt is being bound together, how well it is being mixed, rolled, and compacted, The weather and natural surroundings.

Soil gradation is altered by the process of mechanical stabilization, which enhances the soil's inherent qualities. Soil is compacted and densified by the use of rollers, rammers, vibration methods, and sometimes blasting to provide mechanical energy. In this technique, the soil's stability comes from the substance itself. When two or more distinct kinds of natural soil are combined, a composite material is created that is better than any of the individual soils. Mixing or combining soils of two or more gradations to generate a material fulfilling the specified standard constitutes mechanical stabilization (Habiba et al. 2017).

2.5.3 LIME STABILIZATION

In the process of lime stabilization, lime interacts with the soil, causing an exchange of cations in the absorbed water layer and a reduction in the flexibility of the soil. The end product is more friable than the starting clay and may be used as a subgrade with greater success.

Lime treatment is widely used as a chemical method of soil stabilization. Different types of lime, such as hydrated high-calcium lime, monohydrated dolomitic lime, calcitic quicklime, and dolomitic quicklime, may be utilized for various purposes. Lime's calcium reacts with the clay's adsorbed cations to trigger flocculation, lowering the clay's PI and making it easier to work with and blend. Dolomite (Ca(OH)2+MgO) and calcium hydroxide (Ca(OH)2) are the most common types of lime compounds. Lime production is a particularly energy-intensive process that results in significant carbon dioxide emissions. The optimum PI value for lime stabilization of clayey materials is below 10. Some clays undergo a pozzolanic reaction, which produces cementing agents that boost soil strength. For materials with a sulphate level over 0.3%, including silts, granular materials, and soils, it is not an effective stabilizer. Some of the lime used in the treatment process might be swept away into the environment and have an effect there if the area is not properly protected from runoff. Lime may be used to stabilize soil at a low cost. Lime stabilization refers to a technique for improving soil in which lime is introduced to the soil to enhance its qualities. Four different kinds of lime—hydrated high calcium lime, monohydrated dolomite lime, calcite fast lime, and dolomite lime—are often used to amend soil. Most soil stabilizers include lime at a concentration of between 5 and 10 percent. Cementing effect produced by pozzolanic reaction is favored above the cementing effect brought by cation exchange capacity in lime modification (Shearwood, 1993). Naturally plate-shaped clay particles are altered through soil modification to take on the form of needle-like interconnecting metalline structures when the clay flocculates. When clay soils dry up, they become less able to adapt to varying moisture levels. Pozzolana minerals react with lime in the presence of water to form cementitious chemicals in a process known as pozzolanic reaction or lime stabilization (White, 2005). CaO (quicklime) or Ca(OH)2 (hydrated lime) may provide the same result. In dry soils, when water is needed for compaction, slurry lime may also be employed (Rogers and Glendinning, 1993). The following are benefits of using quicklime instead than hydrated lime, notwithstanding quicklime's popularity (White, 2005) a greater percentage of active free lime per gram - creates heat that accelerates strength increase and a considerable drop in moisture content according to the reaction equation below; - is denser than hydrated lime (requiring less storage space) and produces less dust; -

$CaO + H_2O \rightarrow Ca(OH)_2 + \text{Heat } (65\text{kJ / mol})$

When added to moist soil, quicklime may absorb up to 32% of its weight in water, transforming into hydrated lime; the heat produced by this reaction also leads to the loss of water via evaporation, increasing the soil's plastic limit (its capacity to dry out and absorb water) (Al-Tabbaa and Evans, 2005). The reduction in flexibility was initially hypothesized by Sherwood to be the result of cation exchange, in which sodium and hydrogen cations are exchanged for calcium ions, for which the clay mineral has a larger affinity to water. Soil exchange capacity may be increased by adding lime even if the clay is already saturated with calcium ions (as in calcareous soils). Lime, like cement, raises the pH when it combines with moist clay minerals, making siliceous and aluminous chemicals more soluble. Cementitious products comparable to cement paste are produced when these chemicals react with calcium to create calcium silica and calcium alumina hydrates. Clay minerals, powdered fly ash, PFA, and blast furnace slag are all examples of natural pozzolanas materials that include silica and alumina and have the ability to react with lime. Applications in geotechnical engineering and environmental protection make up the bulk of the market for lime stabilization technology. Lime piles or lime-stabilized soil columns may be used for a variety of purposes, including the encapsulation of pollutants, the rendering of backfill (for example, wet cohesive soil), the capping of highways, the stability of slopes, and the upgrading of foundations. The lime stabilization process may be hampered by the presence of sulphur and organic components. Soil strength may be affected when sulfate (gypsum) reacts with lime to expand.

2.5.4 BITUMINOUS STABILIZATION

Bituminous stabilization systems are hydrocarbons that are insoluble in water but soluble in carbon disulfide. In this way, asphalt functions as a cementing or bonding agent, holding together otherwise cohesionless soils. Bituminous stabilization may be affected by many things, including soil type, asphalt quantity, mixing, and compaction. Soil stabilization using bitumen may take the form of either pure bitumen, cutback bitumen, or bitumen emulsions. It is important to consider the soil type, building technique, and weathering conditions when choosing the kind and grade of bitumen to use. Moisture content, bitumen viscosity, bitumen content, homogeneity in mixing, aeration, compaction, and curing are the most critical elements influencing bitumen stability. In order to make soil more weatherproof, bitumen binds to the soil particles. Subsequently, the soil's strength and weather resilience are greatly enhanced due to the absence of water penetration. Bitumen stability is impaired by the presence of organic matter, dissolved salts, and high pH values in soils. Higher than optimal levels of bitumen fill holes between soil or aggregate particles, leading to poor compaction, lower strength, and degraded deformation behavior of the stabilized soil. The ideal amount of bitumen to use ranges from 4 percent to 7 percent.

A sturdy foundation or Wearing surface may be created by incorporating a measured quantity of bituminous material into preexisting soil or aggregate material and properly blending the two together. Bitumen improves the soil's cohesiveness and load-bearing ability while also making it impervious to the effects of water. Bitumen stabilization may be achieved via asphalt cement, asphalt cutback, or asphalt emulsions. The soil to be stabilized, the building process, and the weather all play a role in determining the best kind of bitumen to utilize. The high temperature maximum susceptibility of tar as a binder makes it inappropriate for usage in frosty regions. Bituminous materials like asphalt and tar are often used in the building of roads and highways to stabilize the soil. The addition of bituminous materials to soil improves its cohesiveness and makes it less likely to wash away in a flood.

2.5.5 CHEMICAL STABILIZATION

Soil stabilization with the use of chemicals is known as chemical stabilization. It's more pricey here, but you have greater flexibility over the setting and curing times. Calcium chloride, sodium chloride, sodium silicate, polymers, and chrome lignin are only some of the compounds utilized in chemical stabilization. The use of admixtures in chemical stabilization causes changes to the soil's chemical characteristics. About 18 distinct chemical mechanisms have been observed in research, including ion exchange, ion adsorption, ion fixation, mineral formation, cementation, salt conversion, film modification, film adsorption, ion enrichment in pore water, capillary force modification, electrical surface tension modification in clay mineral, electrical force modification between particles, chemical force modification, and adsorption. Some of the advantages of chemical stabilization include the predictability of laboratory test results, the low minimum doses of chemical additives required to achieve stabilization, the speed with which the stabilization process can be accomplished, and the indifference of the stabilization process to soil engineering properties. The risk of groundwater contamination is very high due to the release of toxic compounds from some of the traditional agents; the leachate of toxic chemicals can affect the environment and human life in general; the cost of chemical soil stabilizer and the quantity required to achieve effective stifling are not always in harmony; and so on.

2.5.6 GROUTING STABILIZATION

Soil stabilizers are injected into the ground in this procedure. High-viscosity stabilizers work best on low-permeability soils, therefore this is often done under pressure. Some of the types of grout stabilization include; cement grouting, clay grouting, chemical grouting, polymer grouting, bituminous grouting.

2.5.7 FLY ASH STABILIZATION

Another common chemical stabilizer is fly ash. It's a waste product of power plants that burn coal to create electricity. The pozzolanic reaction and the filling of gaps in the mixture are the mechanisms by which fly ash stabilizes soil. Coarse grained particles with few or no fines may be stabilized using this method. The stabilization process works best on dry soil. Fly ash generated from the burning of harder, older bituminous, anthracite coal is pozzolanic but not self-cementing, hence an activator is often required after the right quantity of fly ash has been added to boost the mixture's pozzolanic reaction. Fly ash is activated with lime or Portland cement at a rate of 20-30 percent. Heavy metals and other toxic substances in fly ash can leach into the ground and water when released into the environment. The widespread availability of fly ash has led to a rise in its relevance in recent years for stabilizing purposes. In comparison to other ways, this one is quick and easy on the wallet. It has been used for decades in engineering and geotechnical applications with great success. Coal-fired power plants produce fly ash as a byproduct, but unlike lime and cement, it isn't very useful for cementation purposes. Most fly ashes are secondary binders, and thus are ineffective in producing the desired effect on their own. When combined with a tiny quantity of activator, however, it may undergo a chemical reaction to generate a cementations compound that helps strengthen fragile soil.

However, there are constraints to using soil fly ash for stabilization (MacLaren and White, 2003):

1. Since stabilization requires a lower moisture level in the soil, dewatering may be necessary.

2. Soil-fly ash mixture cured below zero and then soaked in water are highly susceptible to slaking and strength loss

3. Sulfur contents can form expansive minerals in soil-fly ash mixture, which reduces the long term strength and durability.

2.5.8 BONE ASH AS A SOIL STABILIZER

In addition to its mechanical and structural roles, bone also plays important biological and chemical roles in the body. Hydroxyapatite and other amorphous calcium phosphate compounds, perhaps containing carbonate, make up the bulk of bone. Age, diet, hormone levels, and illnesses all have an impact on the chemical and physical qualities of bones (Loveridge, 1999). Bone ash is created by burning cattle bones. Bone ash is a fine, powdery ash that is produced when bones are burned (calcinated). Calcium phosphate is its primary ingredient. The term "calcination" refers to a method of heating at high temperatures when atmospheric oxygen is present. The final byproduct is a substance similar to hydroxyapatite known as "bone mineral." All organic matter burns down to carbon dioxide. Bone ash is noteworthy because its key features stem from the retained cellular structure of bones throughout the calcination process (Ayininuola et al, 2016). Bone ash has extremely high heat transfer resistance and good nonwetting qualities since it is chemically inert and devoid of organic stuff. Calcined bone ash comprises CaO (45.53 %), P2O5 (38.66 %), MgO (1.18 %), SiO2 (0.09%), Fe2O3 (0.1 %), Al2O3 (0.06%), and moisture, as reported by Ayininuola and Shogunro (2013). (0.11).

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2.5.9 ORANGE PEEL AS A SOIL STABILIZER

The orange peel, the most abundant byproduct of orange juice processing, is rich in flavonoids that have been linked to antioxidant activity (Kanaze et al, 2008). Monoterpenes abound in orange peel oil, with d-limonene constituting over 90% of the total. It has several applications, including in the manufacture of orange juice and as an antioxidant in both culinary and pharmaceutical products. The engineering potential of orange peels is the subject of this research. In this case, the peels are to be oven-dried and ground into a powder that may then be used to stabilize laterite soil.

2.5.10 RICE HUSK ASH STABILIZATION

The amount of solid waste sent to landfills may be reduced if the trash has desired qualities that make it useful in geotechnical applications such land reclamation, embankment building, etc. Densification (including shallow compaction, dynamic deep compaction, and pre-loading), drainage, inclusions (including geosynthetics and stone columns), and stabilizations are all utilized to enhance the geotechnical attributes of troublesome soils. It is crucial to chemically stabilize the problematic soils while dealing with the treatment of soft fine-grained, expansive soils and collapsible loess deposits. Soil stabilization is the process of enhancing the soil's engineering features in order to increase its stability. The need for soil stabilization arises when the available building soil is unsuitable for the intended use. Methods like compaction, pre-consolidation, and draining are all a part of this. In spite of its low production and ready availability, rice husk ash (RHA) is a pozzolanic substance with potential utility in soil stabilization. After being burned at a regulated temperature, rice husk produces ash that accounts for around 17 percent to 25 percent of the husk's original weight. The ash from rice husks, together with rice straw and bagasse, is a great pozzolana because of its high silica content. Pozzolanas are siliceous and aluminous minerals that, by themselves, have little to no cementations value but which, when finely split and combined with water at room temperature, chemically react to produce compounds having cementations qualities. As the silica in Rice Husk Ash seems to be in crystalline form, suggesting that the substance is inert, it is very improbable that the ash would react with lime to generate calcium silicates. It's also probably not going to be as reactive as fly ash, which is more finely separated. As a consequence, Rice Husk Ash is a fantastic stabilizer. Ash seems like a wonderful lightweight filler that won't cause any major problems.

2.5.11 THERMAL STABILIZATION

As a consequence of temperature increases, the soil's properties greatly improve. You may use heat or cold to stabilize the soil. Soil drying out because to heating. Raising soil strength through decreasing electrostatic repulsion between clay particles. A little amount of strength is lost in clayey soils when they are frozen; this is because to an increase in interparticle repulsion. When the temperature of the soil is lowered below freezing, the pore water freezes and the soil is stabilized.

2.5.12 ELECTRICAL STABILIZATION

Electro-osmosis is a technique for improving the structure of clayey soils by use of electrical current. When direct current (DC) is run through clayey soil, pore water moves to the negative electrode (cathode). This occurs because the positive ions (cations) in the water are drawn to the cathode. The removal of water has a dramatic effect on the durability of the soil. Although effective, draining cohesive soils through electro-osmosis is an expensive method. Not only that, but the soil's inherent properties are now higher.

2.5.13 STABILIZATION BY GEO-TEXTILE AND FABRICS

Porous fabrics consisting of synthetic materials such as polyethylene, polyester, nylons, and polyvinyl chloride are known as geotextiles. Geotextiles are offered in woven, nonwoven, and grid form. Geotextiles are highly durable. It helps in the stability of the structure when correctly entrenched in the soil. It's used to build unpaved roads on soft soils. Metallic strips are inserted into the soil for support, and an anchor or tie back is provided to constrain a facing skin element. Nonbiodegradable reinforcing materials, such as fibers, geotextiles, geogrids, and geocomposites, have been demonstrated to increase the strength and load-bearing capability of subgrades and base course materials in previous study. These materials have the potential to increase the performance and longevity of future roadways while also lowering building costs. The majority of current research on these materials is based on laboratory testing that are only half completed. To produce design standards based on material attributes, more laboratory testing and assessments will be required, and these specifications will need to be confirmed using large-scale field tests. (Habiba, 2017)

2.5.14 RECYCLED AND WASTE PRODUCTS

Chemical and mechanical stabilization methods must be refined for a variety of materials, including recycled rubber tire chips, copper and zinc slag, muck from paper mills, and crushed asphalt. Since many potentially dangerous materials must be recycled, it is important to develop a practical, efficient, and effective approach to evaluating the potential for contamination from leachates and emissions resulting from their processing and disposal. In certain cases, the scope of a risk assessment may be constrained by the state of the environment.

2.6 FACTORS AFFECTING THE STRENGTH OF STABILIZED SOIL

Organic matter, sulphates, sulphides, and carbon dioxide in stabilized soils may contribute to unsatisfactory stabilized material strength. (Shearwood, 1993)

2.6.1 ORGANIC MATTER

Most soils have a substantial amount of organic matter in their top layers. But in well-drained soils, organic matter may penetrate down to depths of 1.5 m or more. Those findings may be seen in (Shearwood, 1993). Soil pH is lowered when organic matter combines with hydration products such calcium hydroxide ($Ca(OH)\ 2$). Because of the low pH value, the hydration process may be slowed, and the hardness of stabilized soils may be influenced, making compacting difficult or impossible.

2.6.2 SULPHATES

In the presence of considerable moisture, sulfate-rich soils that have been treated with a calcium-based stabilizer create calcium sulphoaluminate (ettringite) and/or thamausite, a product with a greater volume than the reactants combined.

However, it's possible that dissolving the sulfate and allowing the reaction to take place would need more water than was originally available during mixing. (Hicks, 2002).

2.6.3 SULPHIDES

Iron pyrites ($FeS2$), a kind of sulphide, may be found in many types of garbage and by-products from factories. By following the processes (1) and (2) below, sulphuric acid, which is formed when $FeS2$ is oxidized, may be converted into gypsum (hydrated calcium sulphate).

(1) $2FeS_2 + 2H_2O + 7O_2 = 2FeSO_4 + 2H_2SO_4$

(2) $CaCO_3 + H_2SO_4 + H_2O = CaSO_4 \cdot 2H_2O + CO_2$

Hydrated sulfate has the potential to damage the stabilized material in the same manner as sulfate does when present in high enough concentrations. Despite this, gypsum could be a component of organic soil (Shearwood, 1993).

2.7 COMPACTION

In actual construction, the impact of a binder on soil density is crucial. Maximum dry density is lower for the stabilized combination at a given degree of compaction compared to unstabilized soil. The optimum moisture level increases as the amount of binders increases. Those findings may be seen in (Shearwood, 1993). As soon as cement stabilized soils come into contact with water, the hydration process starts. Compaction of the soil mix as soon as possible is essential since the hardening of the soil mix is necessary for this technique. The stabilized soil mass might harden over time if compaction is delayed, requiring more work to accomplish the same result. This might cause significant bond breakage and thus a weakening of the material. Lime-stabilized soils, in contrast to cement, could benefit from postponing compaction. To get maximum plasticity benefits from lime stabilized soil, a mellowing period is necessary to enable the lime to penetrate the soil. After this period, the lime-stabilized soil may be mixed and compacted to achieve maximum strength.

2.8 MOISTURE CONTENT

Both the hydration process and proper compaction rely on the presence of sufficient moisture in stabilized soils. When compared to Quicklime (CaO), which absorbs only around 32 percent of its own weight in water, fully hydrated cement absorbs roughly 20 percent. As reported by (Hebib & Farrell, 1999). If the soil's moisture level is too low, the binder will absorb some of it instead. Soil strength may be negatively impacted by inadequate moisture in high soilwater affinity soils (such clay, peat, and organic soils). (Hicks, 2002).

2.9 TEMPERATURE

The pozzolanic reaction is temperature-dependent. The temperature out in the field varies throughout the day. Pozzolanic interactions between binders and soil particles weaken at low temperatures, making the stabilized mass less robust. In colder areas, stabilizing the soil throughout the growing season may be important. according to (Shearwood, 1993).

2.10 FREEZE-THAW AND DRY-WET EFFECT

Stabilized soils are vulnerable to repeated periods of freezing and thawing. Therefore, protecting stabilized soils in the field against frost damage may be crucial. The forces of shrinkage in stabilized soil will depend on the chemical reactions of the binder. Repeated dry-wet cycles, brought on by diurnal temperature changes, may put a strain on cement-stabilized soils. Therefore, safeguards against such effects are needed. Reference: (Hebib & Farrell, 1999).

2.11 ENGINEERING CLASSIFICATION OF SOIL

Subclassification of soils is possible depending on their engineering characteristics and behaviour. Systems for classifying soils are able to identify the main characteristics of the many different soil types without providing detailed descriptions of each. The American Association of State Highway and Transportation Officials (AASHTO) created both the AASHTO system and the USCS (Unified Soil Classification System). The majority of AASHTO's users are soil engineers working for state and municipal transportation departments. Both models account for the particle size distribution and the Alterberg constraints. The Unified system of classification is widely used in the geotechnical industry.

2.12 AAS HTO CLASSIFICATION SYSTEM

The AASHTO system of soil classification was developed in 1929 as the Public Road Administration Classification System. According to this system, soil is classified into seven major groups: A-1 through A-7. Soils classified under groups A-1, A-2, and A-3 are granular materials of which 35% or less of the particles pass through the No. 200 sieve. Soils of which more than 35% pass through the No. 200 sieve are classified into groups A-4, A-5, 4-6, and A-7. These soils are mostly silt and clay-type materials. The classification system is based on the following criteria:

1. Grain size:

a. Gravel: Fractions passing the 7-5-mm (3-in.) sieve and retained on the No. l0 sieve

b. Sand: Fractions passing the No. 10 sieve and retained on the No.200 sieve

c. Silt and clay: fraction passing the No. 200 U.S. sieve

2. Plasticity: The term silty is used when the fine fractions of the soil have a plasticity index of 10 or less. The term clayey is used when the fine fractions have a plasticity index of 11 or more.

3. If cobbles and boulders (a size larger than 75 mm) are encountered, they are excluded from the portion of the soil sample from which classification is made. However, the percentage of such material is recorded.

2.12.1UNIFIED SOIL CLASSIFICATION SYSTEM

Casagrande first introduced the concept of a unified soil classification system for use by the Army Corps of Engineers in building airfields during World War II. In 1952, the Corps worked with the US Bureau of Reclamation to update this system. Many engineers nowadays rely on it (ASTM designation D -2481). When applying the categorization system, keep the following in mind.

1. The classification is based on material passing a 75 mm (3 in.) sieve.
2. Coarse fraction: percent retained above No. 200 sieve = 100–F200 = R200
3. Fine fraction: percent passing No. 200 sieve = F200
4. Gravel fraction: percent retained above No.4 sieve = R4.

According to the Unified Soil Classification System, the soils are divided into two major categories:

1. Coarse-grained soils that are gravelly and sandy in nature with less than 50% passing through the No. 200 sieve. The group symbols start with prefixes of either G or S. G stands for gravel or gravelly soil, and S for sand or sandy soil.
2. Fine-grained soils with 50% or more passing through the No. 200 sieve. The group symbols start with prefixes of M, which stands for inorganic silt, C for inorganic clay, and O for organic silts and clays. The symbol Pt is used for peat, muck, and other highly organic soils. Other symbols used for the classifications are:

W- Well graded

P- Poorly graded

L - Low plasticity (liquid limit less than 50)

H- High plasticity (liquid limit more than 50)

2.13 PREVIOUS WORKS ON STABILIZATION OF LATERITE

Kasthurba et al. (2014) looked at the viability of laterite as a construction material in low-income areas. Their study provides a concise overview of laterite's occurrence and features before highlighting the necessity for a global database of qualities and standardized testing methodologies to promote its wider assessment and implementation. Azeko (2015) looked on recycling waste PE and using it as reinforcement in laterite bricks for eco-friendly construction.

The finest balance of flexural/compressive strength and fracture toughness was found in a composite with 20% PE, according to his research. Compressive/flexural strengths and fracture toughness values are also seen to decrease with 20 vol. percent of PE. AB is able to effectively fill the gaps between soil particles thanks to geopolymer. Aziz and Mukri (2016) looked at how much of a geopolymer may be used to make a difference in the compaction parameter of laterite soil. The findings show that when using both high and normal compaction efforts, the dry density and moisture content of soil are best served by a laterite soil mix containing 15 percent geopolymer. Mustapha (2015). Mechanical parameters such as compressive strength, flexural strength, and fracture toughness were tested for natural fiber (straw) reinforced laterite and found to be enhanced.

Marto et al. looked at an innovative soil stabilization method called SS299, a liquid polymer soil stabilizer (2013). The results show that the shear and unconfined compressive strengths of laterite soil may be greatly improved by using SS299 soil stabilizer. According to research conducted by Ayininuola and Shogunro (2013), bone ash has a fascinating effect on improving soil shear strength. Laterite soil stabilized using liquid soil stabilizers, canlite, was studied for its geotechnical qualities by Mohd Yunus et al (SS 299). (2015). The geotechnical qualities of laterite soil were shown to be enhanced when treated with the SS 299 soil stabilizer. During the first 28 days of curing, there was the most fluctuation in the unconfined compression strength. Achampong et al. (2013) studied laterite soils in Legon, Ghana, to learn more about the chemical stabilization of laterite soils for road building. Soil stabilization was shown to be optimal with only 6% lime input, when tested against the requirements of the Ghana Highway Authority (GHA)

CHAPTER THREE

MATERIALS AND RESEARCH METHODOLOGY

3.1 Materials and Sample Preparation

The samples utilized in this study are from Nigeria, where laterite is widely employed in geotechnical engineering projects. Soil composed of laterite was dug out from a layer 1–1.3 meters deep. The two types of laterite used in this study were obtained from the engineering department at the University of Benin's ugbowo campus. The laterite was gathered, allowed to dry in the air, and then milled into a powder with a particle size of around 80 nm. The laterite components were stabilized in a different way by using orange peel that was available for purchase. A locally built engine was used to dry and crush the orange peel. Figure 2(a), 2(b), and 2(c) below show raw laterite samples and orange peels, respectively.

Figure 2(a): Raw laterite sample **1**

Figure 2(b): Raw laterite sample **2**

Figure 2(c) Grinding of the orange peels with a locally made engine.

In this study, the raw laterite and the stabilized laterite were used to mold bricks of different material matrices and compositions as shown in Figure 3. The hydrated lime and bone ash were first mixed dry with the laterite respectively. Subsequently, a calculated amount of water needed to satisfy the moisture requirement of the soil and the hydration needs of the stabilizer was added. The samples were obtained with the aid of a locally made mold of 50 x 50mm cube size. The mold was constructed to correspond to the shape and size of the samples to be made. A hydraulic press was used to press the material firmly so that the resulting sample could take the shape of the mold. The samples prepared were to be mechanically tested to obtain their compressive strengths.

3.2 Preliminary Laboratory Tests

In order to understand the behavior of the laterite samples used in this research properly, major preliminary tests were taken. The necessary preliminary tests considered included, Sieve analysis, Moisture Content Test, Specific Gravity Test and Atterberg Limits Test.

3.2.1 Sieve Analysis

When testing aggregate sizes, a known weight of material was placed on top of a series of nested sieves (the top sieve had the largest screen openings and the screen opening sizes decreased with each sieve down to the bottom sieve, which had the smallest opening size screen for the type of material specified) and mechanically shaken for a set amount of time. The material retained on each nested sieve was then weighed after being shaken through the sieves. Each sieve was emptied into a previously weighted pan (the tare weight), the pan was weighed, the contents of the next sieve were put to the pan, and the total was weighed using the cumulative technique. This procedure was continued until the contents of the bottom pan and all sieves had been added and weighed. Particle size distribution information was obtained by washing a known amount of the sample and doing sedimentation analysis.

3.2.2 Water /Moisture Content Test

Soil, rock, and soil-aggregate mixtures' water (moisture) content percentages were measured in accordance with ASTM D 2216, the "Standard Test Method for Laboratory Determination of Water (Moisture) Content." This analysis was carried out to ascertain the relative amount of moisture in the laterite sample. The water content of a soil may be a very useful indicator for determining the nature of the soil's behavior and its qualities in many cases. The water content of a fine-grained soil greatly affects its consistency. The phase relationships of air, water, and solids in a certain volume of soil may also be expressed in terms of the water content. The number of the moisture container and its lid were recorded in order to determine the relative humidity. We weighed and measured a dry, wiped-down, and empty moisture container (MC). We filled the moisture container with damp soil and closed the top. The MCMS was calculated by weighing the can (which was now full of damp dirt) and its lid. The container holding the wet dirt had its cover removed, and it was put into a drying oven preheated to 105 degrees Celsius. After leaving the oven without the moisture container overnight, we took it out the next day. After removing the lid from the moisture container using gloves and making sure it was on tightly, it was set aside to cool to room temperature. The MCDS was calculated by weighing the can and its lid (which contained the dry dirt). The percentage of water or moisture in the laterite was then determined.

3.2.3 Specific Gravity test

The soil's specific gravity (Gs) is defined as the mass of one cubic meter of soil solids at a given temperature divided by the mass of one cubic meter of gas-free distilled water at the same temperature. Properties and Soil Behavior: An Electronic Laboratory Manual. Two samples were utilized; hence four density bottles were required for this study. Empty bottles were weighed first, followed by those filled with water, then with sand, and finally with water plus sand. Various measurements were taken and their averages determined.

3.2.4 Atterberg Limits Test

A fine-grained soil's plastic and liquid limitations were calculated using the Atterberg Limits Test. Liquid limit, plastic limit, and plasticity index of soils were tested in accordance with ASTM D 4318. The Atterberg limitations depend on the amount of moisture in the ground. When a soil can no longer be bent by rolling into threads of 3.2 mm (1/8 in.) diameter without disintegrating, that's when you know it has reached its PL, expressed as a percentage of water. The transition point between the plastic and the viscous fluid states of soil is defined by its liquid limit (LI). The shrinkage limit is the moisture content below which no further reduction in soil volume may be achieved. Soil plasticity is quantified using the Plasticity index (PI). The size of the water content range across which plasticity in the soil is seen. Nonplastic soil has a PI of 0, somewhat plastic soil has a PI of 7, medium plastic soil has a PI of 7-17, and extremely plastic soil has a PI of >17. A high PI indicates a clay-rich soil, a low PI indicates silt, and a PI of 0 indicates a nonplastic soil with little or no clay or silt. Atterberg limits are used to categorize fine-grained soils in the Unified Soil Classification System or AASHTO system, and have been associated to a variety of soil engineering features.

In addition to the California bearing ratio test and compaction, the samples were also (CBR). The soil tests performed in this research adhere to the west African criteria for civil engineering soil testing, and the study's categorization system follows the guidelines established by the American Association of State Highway and Transportation Officials (AASHTO).

3.3 Compaction

The tools include a sample tray, mold with a capacity of 0.002124 m3, sample extruder, measurement cylinder, moisture content tin, hot-air oven, scale, and rammer with a 4.5 kg capacity.

The empty mold was weighed beside the base to achieve an accurate reading. After measuring out 3 kilos of dirt, 6 percent water was added and blended in. Three layers of the dirt mixture were compressed in the mold. A 4.5-kilogram rammer dropped from a height of 450 millimeters. After compression, the mold's collar was cut away, and the top was scarped with iron rod. The weight of the compressed sample was measured.

The soil sample was broken up after being extruded from the mold using a sample extruder. Moisture content was calculated by taking samples from the top and bottom. The process was continued until the compacted soil's weight began to decrease, at which point we stopped adding 3 percent weight of water to the sample. Moisture content samples were collected during the journey. The same methods were used.for both samples, and a percentage of 2,4,6,8 of grinded orange peels were added for each compaction.

Figure 3: Raw laterite sample **1 and 2,** with orange peel stabilizer, at 2,4,6 and 8% for both samples

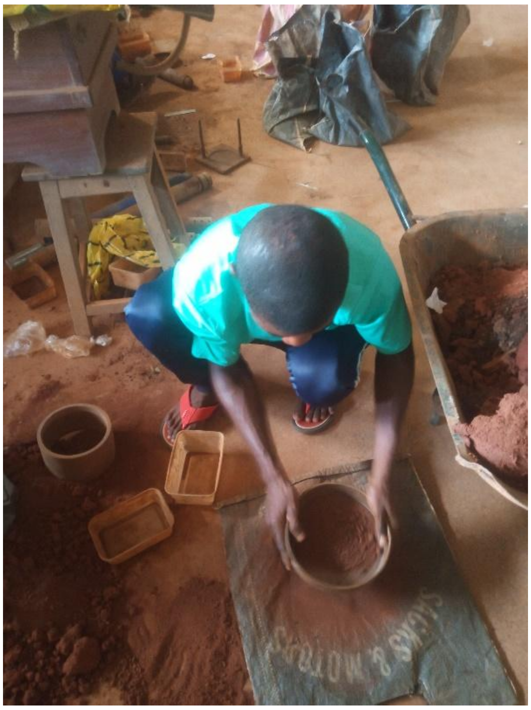

Figure 4: sieving of the sample for compaction

Figure 5: mixing of sample

Figure 6: weighing of soil and mould

Figure 7: sample for oven-drying

3.4 California Bearing Ratio Test (CBR)

A cylinder metal mold with a 0.002124 cm3 capacity, a 4.5 kg rammer, a measuring cylinder, a sample tray, and a California Bearing Ratio testing machine are the instruments you'll need to conduct your tests.

We utilized BS 1377's approach 1 described in Section 5.1.3.3. The mold was put together by attaching the base plate and the collar. The maximum dry density of the soil sample was found by weighing five kilos of the soil and calculating the amount of water needed to moisten the whole sample to that density. Afterward, using the 4.5 kg rammer, the earth was compressed in three layers into the mold with the collar connected. Each layer received a sufficient number of blows, dispersed equally throughout the surface, to bring the dirt up to the rim of the mold. Using a steel straight edge, we cut off the collar and leveled the dirt with the top of the mold.

The specimen mold was placed on the bottom plate of the testing machine with the base plate in place and the top face exposed. The force was measured at 0.25 mm increments up to a maximum penetration of 7.5 mm.

The mold's base plate was unscrewed from the bottom and reattached to the top, and the mold's contents were turned upside down. Both ends of the specimen were subjected to the same battery of tests as stated above. Both samples went through the aforementioned steps, with orange peels added to the soil at 2, 4, 6, and 8 percent.

Figure 8: Raw laterite sample **1 and 2,** with orange peel stabilizer, for CBR.

Figure 9: Oven

Figure 10: Raw laterite sample **2**

Figure 11: CBR Machine.

CHAPTER FOUR

RESULT AND DISCUSSION

The result presented in this chapter covers all the tests carried out on the soil samples and the stabilizers toward achieving the stated aims and objectives.

4.1 Laboratory Test Results

4.1.1 Control Samples

Table 4.1: Summary of the geotechnical test results (**Control sample1 & 2**)

S/No	**Laboratory Test**			**Result for Sample 1**	**Result for Sample 2**
1	**Natural Moisture Content (%)**			16.27	14.36
2	**Specific Gravity**			2.69	2.73
3	**Atterberg Limit**	**Liquid limit (%)**		39.14	36.18
		Plastic limit (%)		24.65	23.69
		Plasticity index (%)		19.50	19.65
4	**Sieve analysis**	**Sample 1**		**Sample 2**	
		Sieve size (mm)	**Percentage passing (%)**	**Sieve size (mm)**	**Percentage passing (%)**
		2.36	99.59		

		2	99.59	2	99.95
		1.18	98.37	1.18	98.7
		0.6	86.58	0.6	84.25
		0.425	80.78	0.425	75.37
		0.3	64.21	0.3	55.93
		0.212	54.38	0.212	44.32
		0.15	50.51	0.15	39.78
		0.075	46.59	0.075	34.72
				Sample 1	**Sample 2**
5	**Compaction on the raw samples 1&2**	**OMC (%)**		16.80	14.20
		MDD (g/cm^3)		1.55	1.66
6	**CBR**	**Top (%)**		3.96	3.74
		Bottom (%)		15.11	15.22

Detailed results can be found at Appendix A – F

From the sieve analysis, the percentage passing the 0.075mm sieve is 46.59%, the percentage passing the 0.425mm sieve is 80.78% and the percentage passing the 2.00mm sieve is 94.7%.

From Atterberg limit test, Liquid limit is 39.14% and the plastic index is 19.50% for laterite sample one (1), and the percentage passing the 0.075mm sieve is 34.72%, the percentage passing the 0.425mm sieve is 75.37% and the percentage passing the 2.00mm sieve is 99.95%. From Atterberg limit test, Liquid limit is 36.18% and the plastic index is 19.65% for laterite sample two (2).

Since, the percentage passing 0.075mm sieve is less than 35% therefore it is a granular soil. Considering all other parameters, the soil is classified as an A-2-6 soil. Based on AASTHO specifications, soils in the A-2-6 class are good as subgrade materials (Table 2.5)

The CBR for the bottom unsoaked was obtained to be 15.11% showing that it is suitable only to be used as subgrade material (Table 2.2). If it is to be used for subbase or base course, it would have to be stabilized or reinforced with other materials. The plasticity index is 19.49% and from Table 2.4, this shows the soil is of clayey type and has medium plasticity.

4.1.2 Natural soil and orange peel.

Table 4.2: Summary of the geotechnical test results (Natural soil sample with orange peel)

S/No	**Percentage Replacement (%)**	**Test**		**Result**	**Result**
1	**2**	**Compaction**	**OMC (%)**	16.50	16.65
			MDD (g/cm^3)	1.49	1.55
		CBR	**Top (%)**	7.43	7.43
			Bottom (%)	1.57	1.65

2	**4**	**Compaction**	**OMC (%)**	18.00	18.50
			MDD (g/cm^3)	1.48	1.45
		CBR	**Top (%)**	25.19	25.16
			Bottom (%)	13.38	14.30
3	**6**	**Compaction**	**OMC (%)**	20.90	20.50
			MDD (g/cm^3)	1.63	1.70
		CBR	**Top (%)**	16.60	17.30
			Bottom (%)	15.18	15.94
4	**8**	**Compaction**	**OMC (%)**	23.00	23.05
			MDD (g/cm^3)	1.27	1.29
		CBR	**Top (%)**	4.76	5.06
			Bottom (%)	6.11	6.87

Detailed results can be found at Appendix G – N

During the compaction test, I observed that at the addition of each percentage of orange peel, (2, 4, 6, and 8%) they is an increase in the compaction. The OMC increased in each sample and in the sample with addition of 6% orange peel to the natural soil sample 2, they is an increase in MDD of 1.70. Also, the maximum CBR was obtained in the sample with 4% orange peel with a value of 25.19% and 25.16% at the top. According to the Federal Ministry of Works and Housing, this value is below the minimum requirement of subbase material. Hence, it cannot be used as subbase on Trunk A roads and can be used as subbase in Trunk D (rural) roads.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The poor durability and associated with short service life of earth-based bricks affect the use of the material as a sustainable construction material. Engineers recognize modification (stabilization) of engineering properties of soil as an important process of improving the performance of problematic soils and this makes marginal soils perform better as a civil engineering material. Based on the laboratory tests, orange peel showed a good effect on stabilization treatment of laterite soil. It was observed from laboratory testing that the mechanical properties of stabilized soil vary and depend on the percentage of stabilizer used. From the critical examinations and analyses, the following conclusions have been drawn:

1. The natural laterite soil is not medium in plasticity, indicated by its plasticity index of 39.49 & 38.65% for both samples. According to AASTHO system of soil classification, the results from the wet sieve analysis show that the soil is an A-2-6 soil. The OMC and MDD were 16.80 and 14.20% and 1.55 and 1.66g/cm^3 respectively. And the maximum value for the soaked CBR was obtained as 15.11%.
2. Considering the optimum mix proportions; for orange peel with natural soil at 6%, the OMC was increased to 20.90% and 20.50, the MDD was also increased to 1.63g/cm^3 and 1.70g/cm^3.

The appropriate mix ratio for orange peel in this experiment was gotten at 6% orange peel mixed with the sample.

5.2 Recommendations

Based on the research findings, the following recommendations are made:

1. The use of soil reinforced with orange peel for subbase should be restricted to Trunk D roads

2. Orange peel increase CBR of the soil but not in substantial amounts so a binder e.g., cement can be added to make it suitable for its use in the subbase and basecourse of the road.

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APPENDIX

A: Results of moisture content test on Control Sample 1&2

MOISTURE CONTENT TABLE FOR BOTH SAMPLES				
SAMPLES	**Sample 1**		**Sample 2**	
Can no.	**COC**	**CN**	**TA7**	**JA2**
Can weight (W1)	17.0	15.9	17.6	17.3
Can weight +wet soil (W2)	125.6	119.3	123.6	130.1
Can weight +dry soil (W3)	107.9	102.5	109.1	113.9
Moisture content	16.29	16.24	13.68	14.36
Moisture content Average	16.27		14.36	

B. Results of Specific gravity test on Control Sample 1&2

SPECIFIC GRAVITY TABLE FOR BOTH SAMPLES				
SAMPLES	**Sample 1**		**Sample 2**	
Bottle No.	**ZC**	**ZH**	**VC**	**JA**
Weight of empty bottle (W1)	23.01	17.09	24.54	20.36
Weight of bottle + soil (W2)	61.27	49.94	53.09	46.10
Weight of bottle + soil + water (W3)	97.56	89.51	94.28	89.76
Weight of bottle + water. (W3)	73.75	69.18	76.17	73.46
Specific Gravity (SG)	2.65	2.74	2.73	2.73
Specific Gravity (SG) Average	2.69		2.73	

C. Table 4.2: Results of Wet Sieve Analysis on Control Sample 1

SIEVE ANALYSIS FOR SAMPLE 1			
Sieve no.	**Mass Retained (g)**	**Mass passing (g)**	**Percentage Passing %**
2.36	0.41	0.41	99.59
200	6.00	0.41	99.59
1.18	1.22	1.63	98.37
600	11.79	13.42	86.58
425	5.80	19.22	80.78
300	16.57	35.79	64.21
212	9.83	45.62	54.38
150	3’87	49.49	50.51
75	3.92	53.41	46.59

D. Table 4.3: Results of Wet Sieve Analysis on Control Sample 2

SIEVE ANALYSIS FOR SAMPLE 2			
Sieve no.	**Mass Retained (g)**	**Mass passing (g)**	**Percentage Passing %**
2.36			
200	0.05	0.05	99.95
1.18	1.25	1.30	98.7
600	14.45	15.75	84.25
425	8.88	24.63	75.37
300	19.44	44.07	55.93
212	11.61	55.68	44.32
150	4.54	60.22	39.78
75	5.06	65.28	34.72

E: Results of Liquid limit and plastic limit test on Control Sample 1

Type of test	Liquid limit					Plastic limit		
	LL	**LL**	**LL**	**LL**	**LL**	**PL**	**PL**	**PL**
Number of blows	46	35	23	18	13			
Can no.	NO2	DEN	0.37	IKH	BLE	TK	FUR	FA4
Weight of can. (g)	18.12	18.08	17.86	18.56	16.93	14.99	18.48	17.34
Weight of wet soil +can. (g)	54.40	54.01	53.09	56.81	58.42	29.88	31.95	28.50
Weight of dry soil +can. (g)	45.49	44.88	44.23	46.67	46.83	26.73	30.29	26.21
Weight of wet soil (g)	36.28	35.93	35.23	38.25	41.49	14.89	13.47	11.16
Weight of dry soil (wd) (g)	9.11	8.95	9.00	8.42	5.34	11.84	16.82	15.05
Weight of moisture (wm) (g)	8.91	9.13	8.86	10.14	11.59	3.15	1.66	2.29
Moisture content, M=wm/wd×100%	97.80	102.01	98.44	120.43	217.04	26.60	9.87	15.22

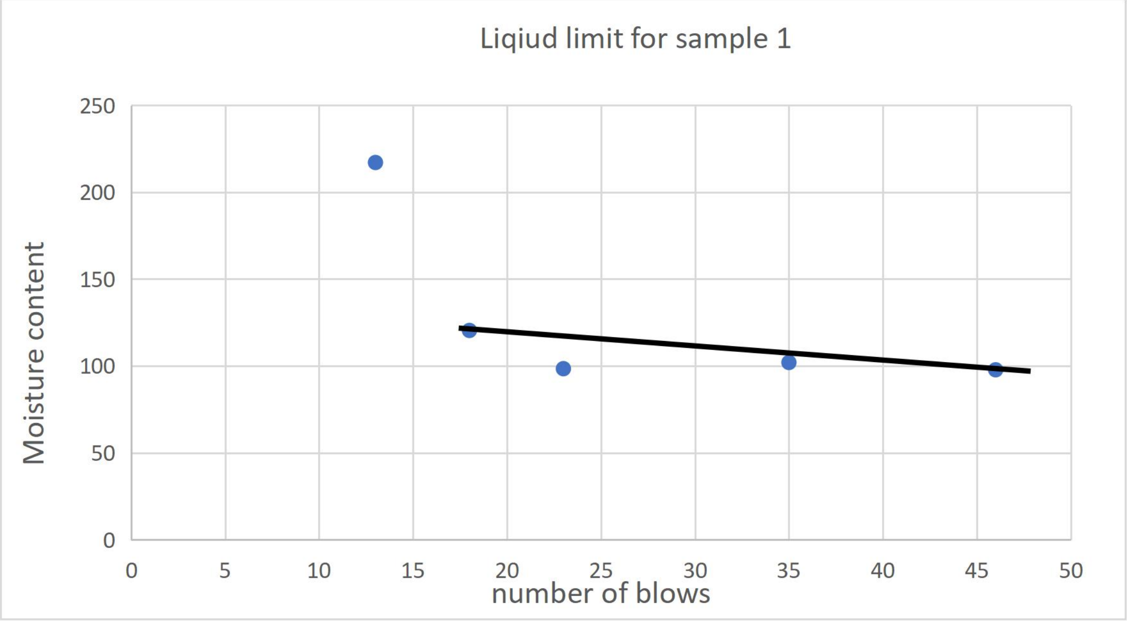


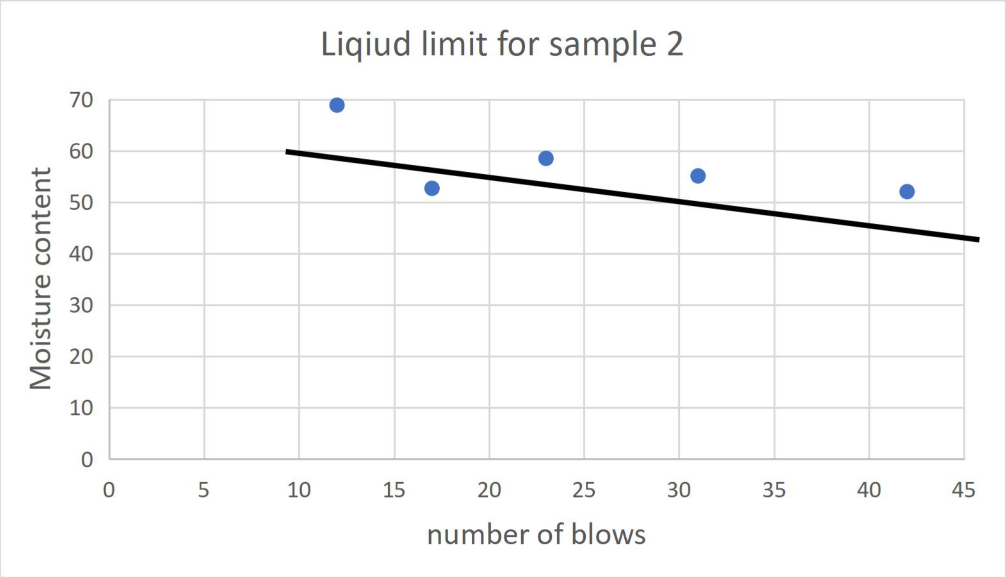
Fig: 12

F: Results of Liquid limit and plastic limit test on Control Sample

ATTERBERG LIMIT TEST
SAMPLE 2

Type of test	Liquid limit					Plastic limit		
	LL	**LL**	**LL**	**LL**	**LL**	**PL**	**PL**	**PL**
Number of blows	42	31	23	17	12			
Can no.	GRE	XA	0.32	2	A	0.5	B	0.21
Weight of can. (g)	25.01	23.70	21.40	23.39	21.65	23.07	21.52	23.25
Weight of wet soil +can. (g)	66.74	63.54	59.20	60.97	61.23	38.12	34.95	38.35
Weight of dry soil +can. (g)	58.18	55.12	51.30	52.90	52.40	35.85	32.93	36.16
Weight of wet soil (g)	41.73	39.84	37.80	37.58	39.58	15.05	13.43	15.10
Weight of dry soil (wd) (g)	16.45	15.28	13.50	15.32	12.82	20.80	19.50	21.06
Weight of moisture (wm) (g)	8.56	8.42	7.90	8.07	8.83	2.27	2.02	2.19
Moisture content, M=wm/wd×100%	52.04	55.10	58.51	52.68	68.88	10.91	10.36	10.39

Fig: 13



G: Results of Compaction test (Control) sample 1

SAMPLE 1 COMPACTION TABLE										
Weight of mold +soil W2 (g)	6359		6431		6465		6435		6425	
Weight of mold +soil W1 (g)	4622		4622		4622		4622		4622	
Weight of wet soil (W2-W1) (g)	1737		1809		1843		1813		1803	
Bulk density (pb) (W2-W1)/g/cm	1.74		1.81		1.84		1.81		1.80	
MOISTURE CONTENT DETERMINATION										
Can no.	**OZB**	**REE**	**24R**	**TA2**	**TNK**	**OOO**	**F.F**	**TAB**	**BO7**	**TA8**
Weight of can + wet soil (g)	67.57	56.33	42.75	45.99	43.35	47.04	41.18	38.92	47.94	42.04
Weight of can + dry soil (g)	61.53	51.63	39.44	41.98	38.93	42.54	37.16	35.30	42.56	37.49
Weight of can (g)	17.23	17.64	18.72	17.24	15.32	18.29	17.40	17.36	18.58	17.42
Weight of dry soil (wd)g	44.30	33.99	20.27	24.74	23.61	24.25	19.76	17.94	23.98	20.07
Weight of Moisture (wm)g	6.04	4.70	3.31	4.01	4.42	4.5	4.02	3.62	5.38	4.55
Moisture content 100(wm/wd)%	13.63	13.83	16.33	16.21	18.72	18.56	20.34	20.18	22.44	22.67
Average Moisture Content +m %g/cm	13.73		16.27		18.64		20.26		22.55	
Dry Density=pb/1+averMc/100	1.53		1.54		1.55		1.51		1.47	

H: Results of Compaction test (Control) sample 2

SAMPLE 2 **COMPACTION TABLE**										
Weight of mold +soil W2 (g)	6133		6477		6539		6482		6437	
Weight of mold +soil W1 (g)	4622		4622		4622		4622		4622	
Weight of wet soil (W2-W1) (g)	1511		1855		1917		1860		1815	
Bulk density (pb) (W2-W1)/g/cm	1.51		1.86		1.92		1.86		1.82	
MOISTURE CONTENT DETERMINATION										
Can no.	**P.P**	**TED**	**IRR**	**BBI**	**DEC**	**100**	**B.5**	**IZO**	**STA**	**MAG**
Weight of can + wet soil (g)	57.72	58.01	59.14	53.72	54.95	57.63	49.49	46.49	56.62	52.11
Weight of can + dry soil (g)	53.77	54.21	54.16	49.56	49.89	52.33	44.89	42.27	50.01	46.77
Weight of can (g)	18.04	18.28	17.43	17.80	18.77	16.59	17.32	17.65	17.54	18.60
Weight of dry soil (wd)g	35.73	35.93	36.73	31.76	31.12	35.74	27.57	24.62	32.47	28.17
Weight of Moisture (wm)g	3.95	3.80	4.98	4.16	5.06	5.30	4.60	4.22	6.61	5.34
Moisture content 100(wm/wd)%	11.06	10.53	13.56	13.09	16.26	14.83	16.68	17.14	20.36	18.96
Average Moisture Content +m %g/cm	10.82		13.33		15.55		16.91		19.66	
Dry Density=pb/1+$\frac{averMc}{100}$	1.36		1.64		1.66		1.59		1.52	

Fig: 14

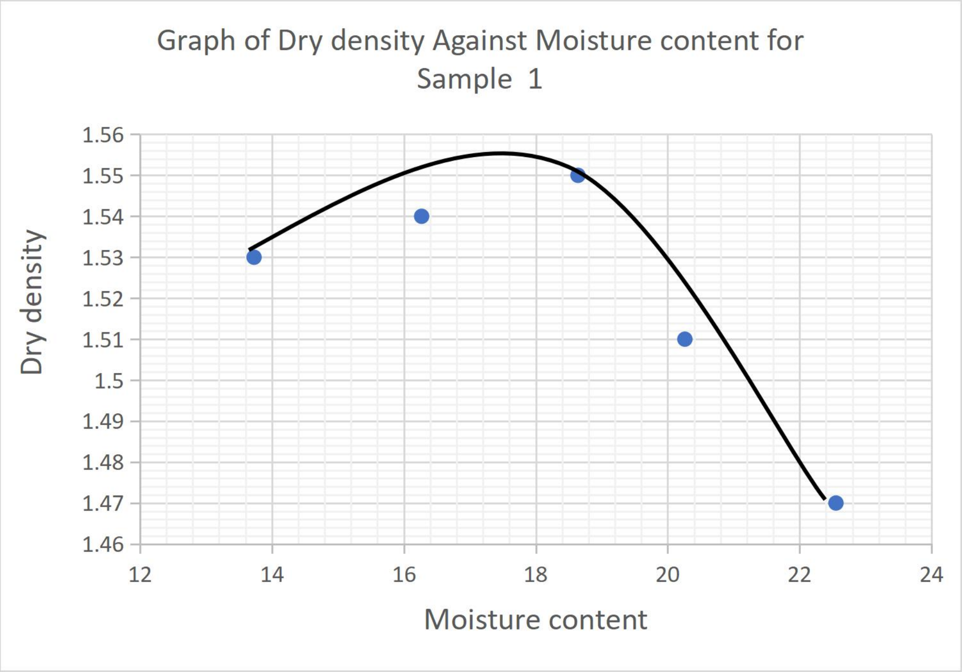


Fig: 15, Graph of Dry density Against Moisture content for Sample 2

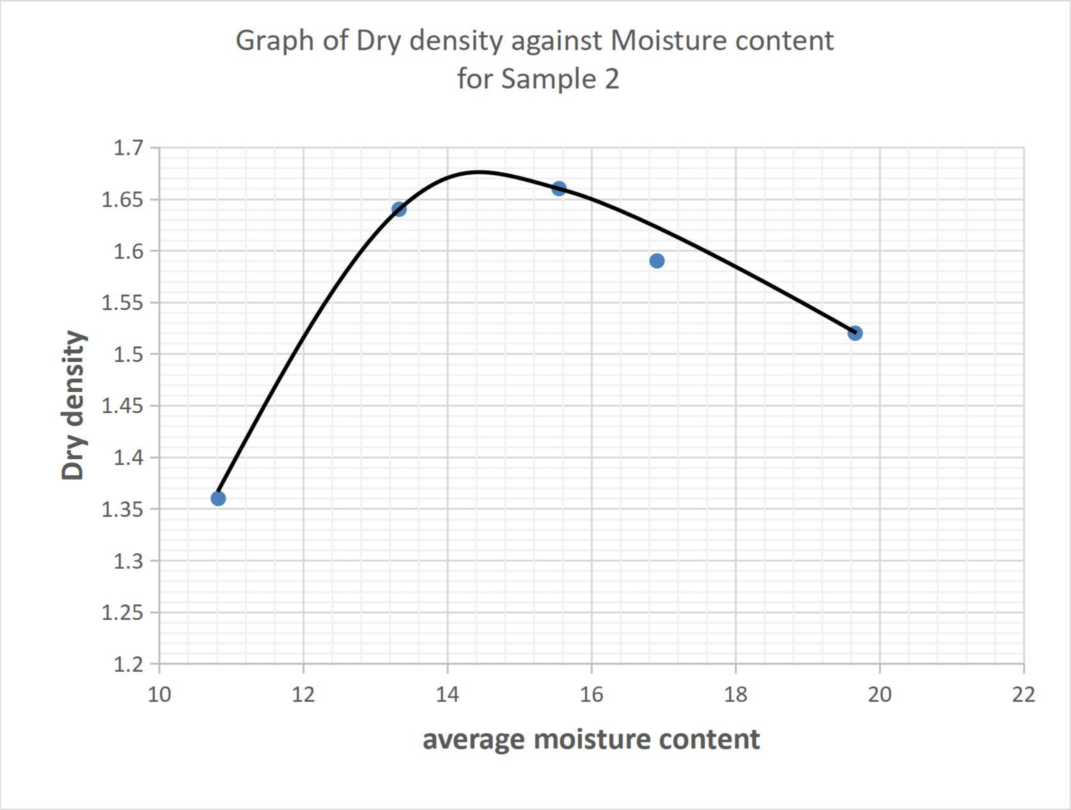
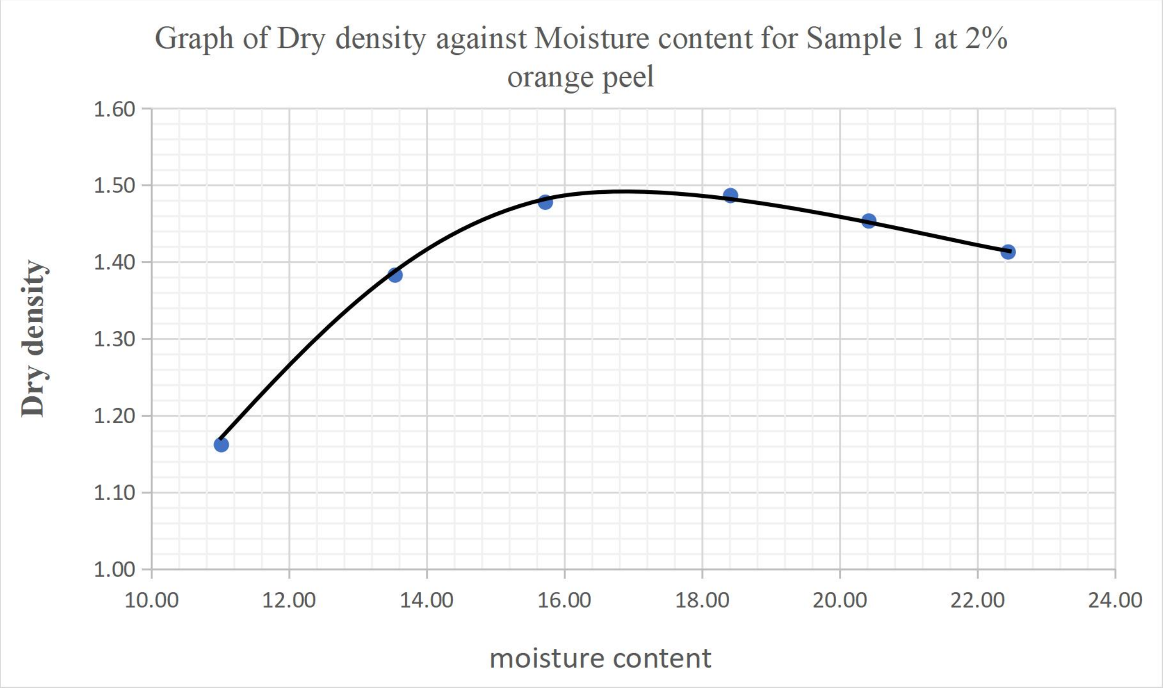


Fig: 16, Graph of Dry density Against Moisture content for both Sample 1 at 2,4,6 and 8% orange peel stabilizer



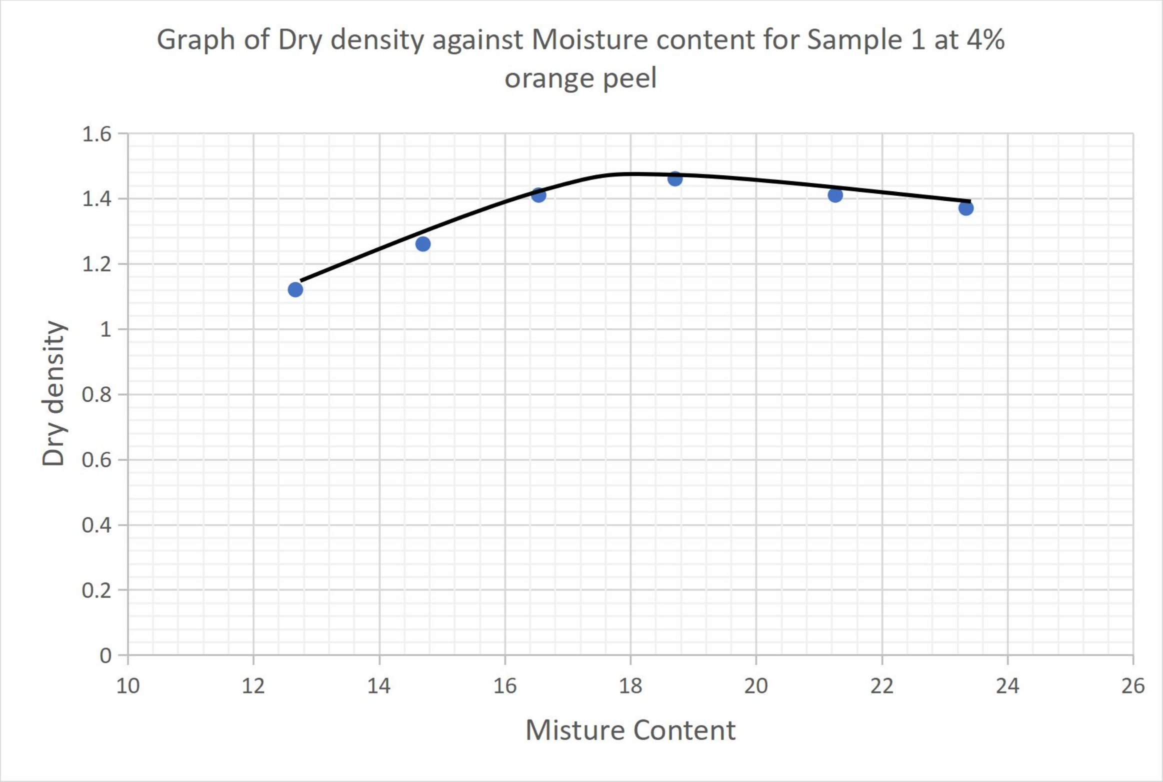


Fig: 17.

Fig: 18.

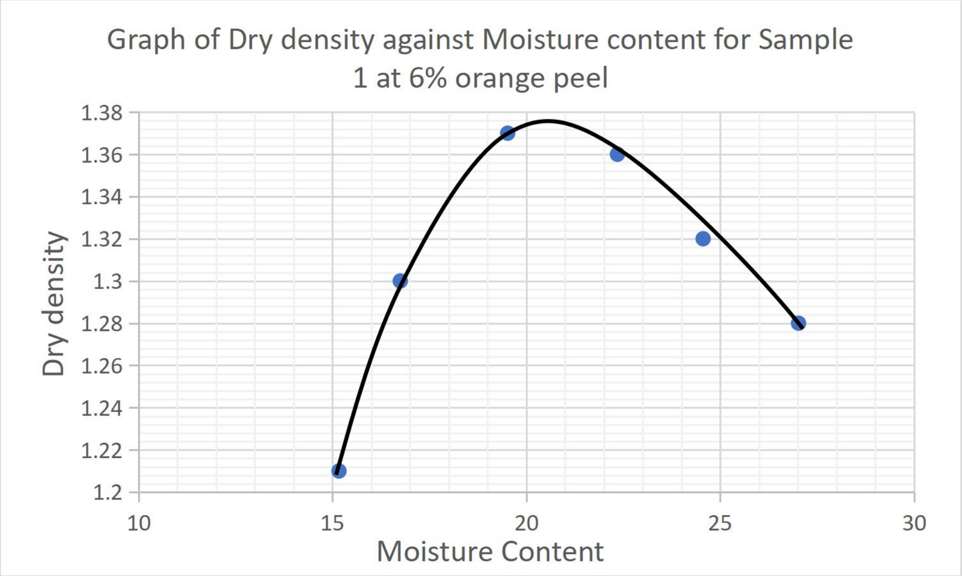

Graph of Dry density against Moisture content for Sample 1 at 6% orange peel
Dry density
1.38
1.36
1.34
1.32
1.3
1.28
1.26
1.24
1.22
1.2
10
15
20
25
30
Moisture Content

Fig: 19.

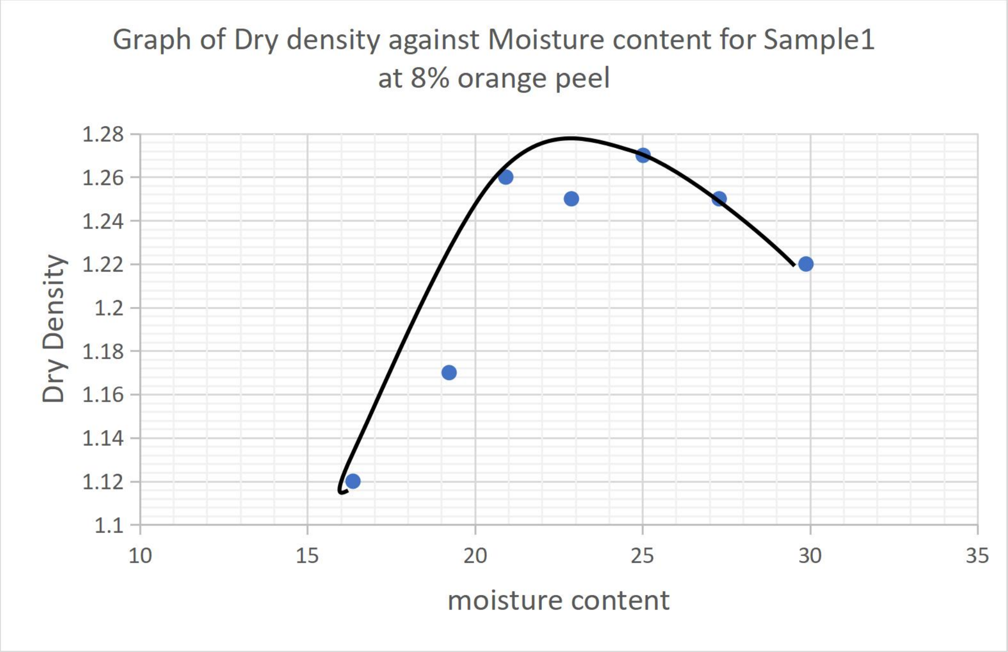

Graph of Dry density against Moisture content for Sample1 at 8% orange peel
Dry Density
1.28
1.26
1.24
1.22
1.2
1.18
1.16
1.14
1.12
1.1
10
15
20
25
30
35
moisture content

Fig: 20. Graph of Dry density Against Moisture content for Sample 2 at 2,4,6 and 8% orange peel stabilizer.

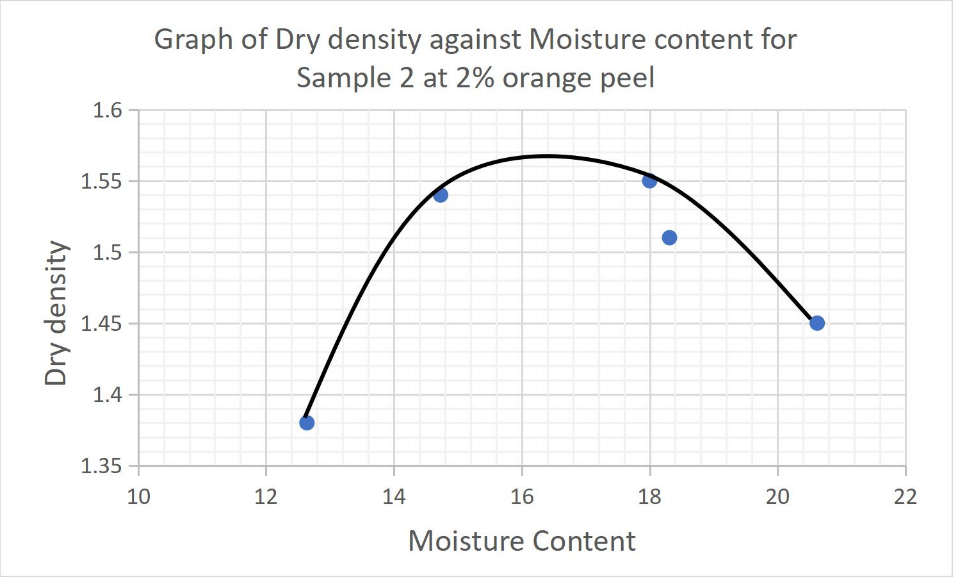
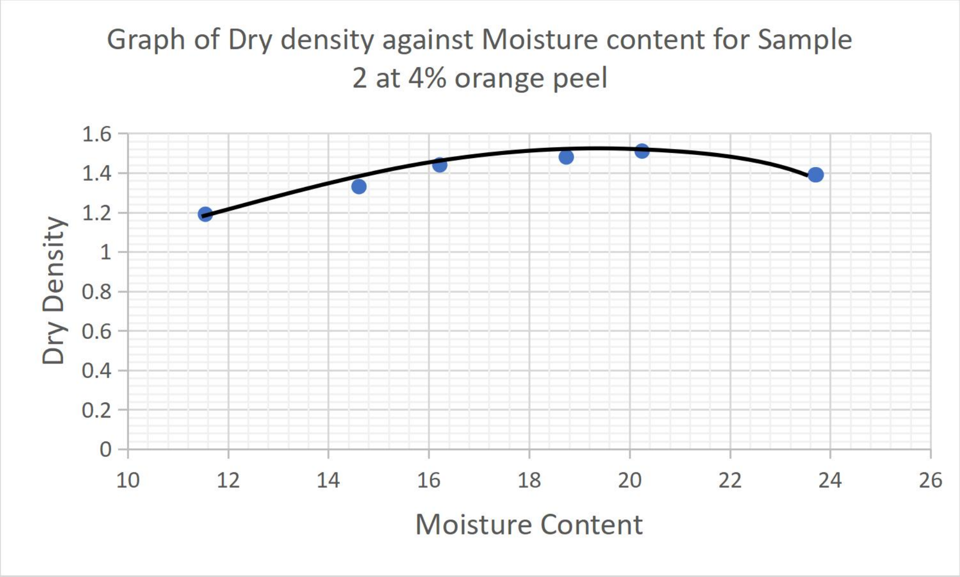


Fig: 21.



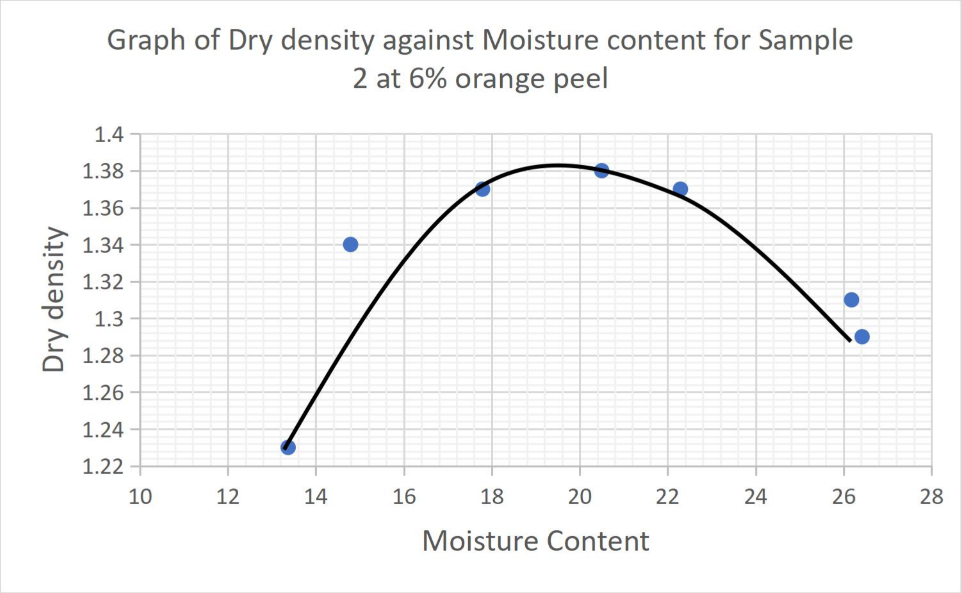

Graph of Dry density against Moisture content for Sample 2 at 6% orange peel
Dry density
1.4
1.38
1.36
1.34
1.32
1.3
1.28
1.26
1.24
1.22
10
12
14
16
18
20
22
24
26
28
Moisture Content

Fig: 22.

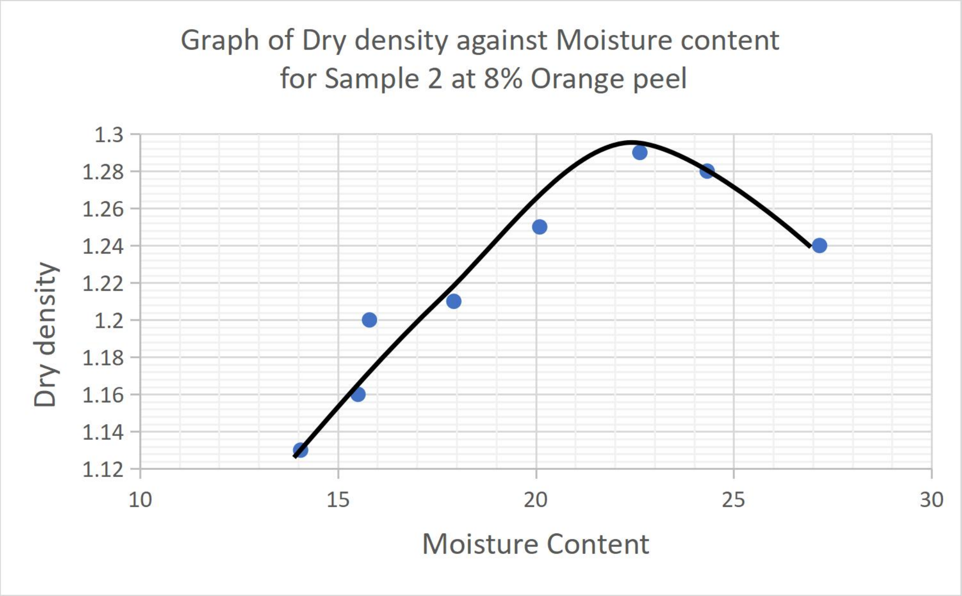

Graph of Dry density against Moisture content for Sample 2 at 8% Orange peel
Dry density
1.3
1.28
1.26
1.24
1.22
1.2
1.18
1.16
1.14
1.12
10
15
20
25
30
Moisture Content

Fig: 23.

R: Result of CBR on Soil Sample 1 without stabilizers

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	85.00	148	217	268	309	349	420	484	629	735	845
Ditto corrected												
Load (KN)	0	0.92974	1.61885	2.37358	2.93142	3.37989	3.81741	4.59402	5.29406	6.8801	8.03954	9.24274
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	53.00	72	91	122	168	215	312	386	548	670	831
Ditto corrected												
Load (KN)	0	0.57972	0.78755	0.99537	1.33445	1.83761	2.3517	3.4127	4.22213	5.99411	7.32856	9.0896
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator		20.00	25	36	46	48	52	60	65	72	83	96
Ditto corrected												
Load (KN)	0	0.21876	0.27345	0.39377	0.50315	0.52503	0.56878	0.65629	0.71098	0.78755	0.90787	1.05006
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	77.00	135	171	180	183	194	194	194	194	194	201
Ditto corrected												
Load (KN)	0	0.84224	1.47665	1.87042	1.96887	2.00168	2.122	2.122	2.122	2.122	2.122	2.19857
C. B. R. %												

	UNSOAKED		SOAKED	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	25.52	26.5233	3.96399	3.56201
TOP	13.8739	21.1529	15.1127	10.6312

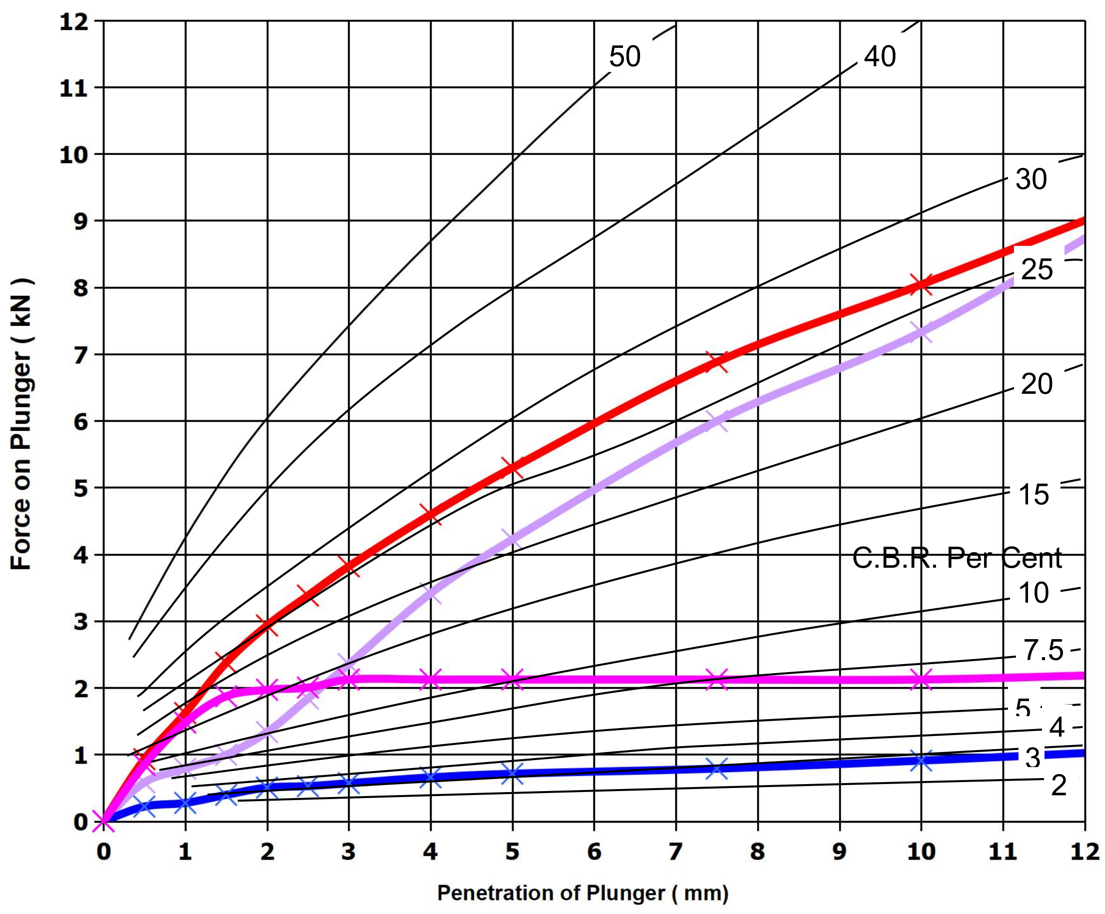


Fig 24: Graph of CBR Test (Control sample 1)

S. CBR: Natural soil Sample 1 and 2% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration	0.0	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50

(mm)	0										
Load Indicator	0	59.00	92	110	129	146	161	193	220	304	382	523
Ditto corrected											
Load (KN)	0	0.64535	1.00631	1.2032	1.41102	1.59697	1.76104	2.11106	2.40639	3.3252	4.17837	5.72065
C. B. R. %											

TEST ON TOP (UNSOAKED)

SURCHARGE

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	86.00	131	160	190	214	227	260	281	337	392	430
Ditto corrected												
Load (KN)	0	0.94068	1.4329	1.7501	2.07825	2.34076	2.48296	2.84392	3.07362	3.68616	4.28775	4.7034
C. B. R. %												

TEST ON BOTTOM (SOAKED)

SURCHARGE

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator		18.00	18	19	19	19	19	19	19	19	19	19
Ditto corrected												
Load (KN)	0	0.19689	0.19689	0.20782	0.20782	0.20782	0.20782	0.20782	0.20782	0.20782	0.20782	0.20782
C. B. R. %												

TEST ON TOP (SOAKED)

SURCHARGE

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	50.00	70	81	85	90	86	71	71	76	84	92
Ditto corrected												
Load (KN)	0	0.54691	0.76567	0.88599	0.92974	0.98443	0.94068	0.77661	0.77661	0.8313	0.9188	1.00631
C. B. R. %												

	UNSOAKED		SOAKED	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	12.06	12.056	1.56908	1.0412
TOP	17.6728	15.3988	7.43247	3.89081

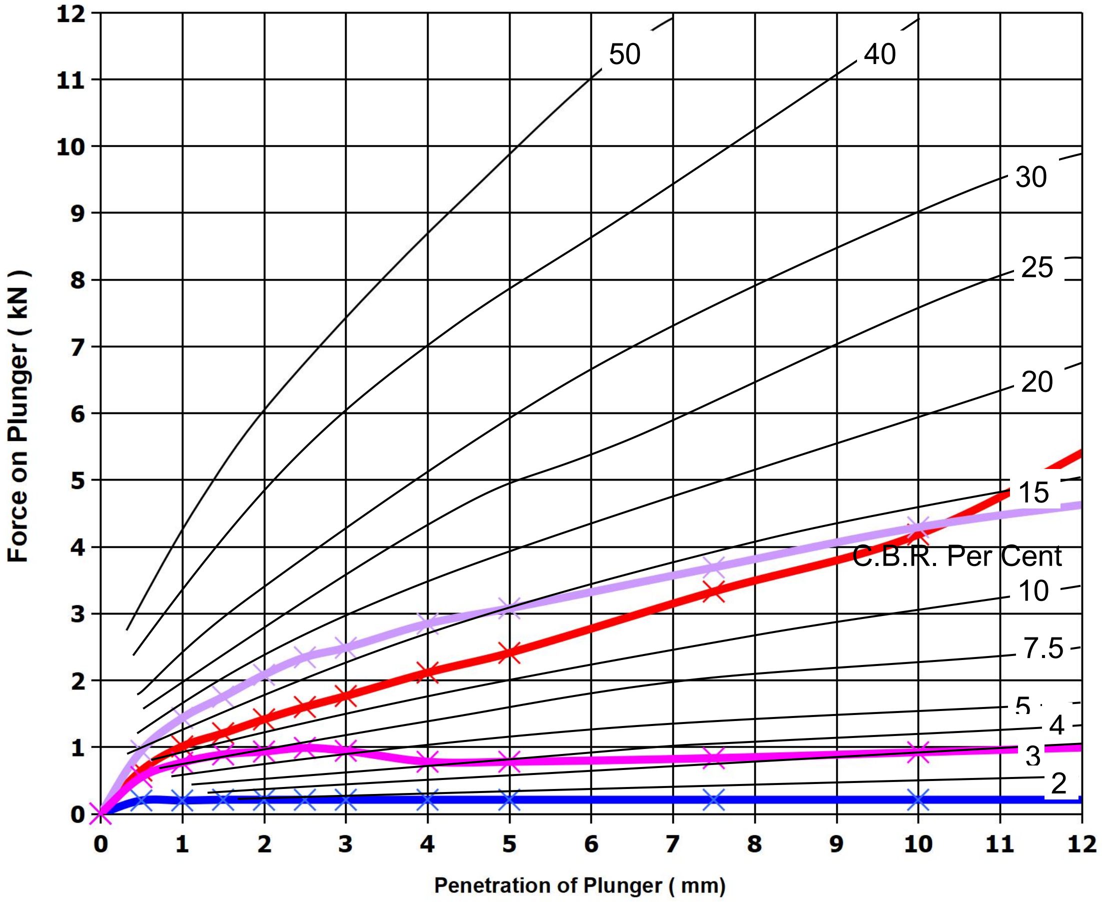


Fig 25: Graph of CBR Test (2% Orange Peel + Natural soil)

T: CBR: Natural soil Sample 1 and 4% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	63.00	124	178	225	274	315	388	445	607	740	863
Ditto corrected												
Load (KN)	0	0.6891	1.35633	1.94699	2.46108	2.99705	3.44552	4.244	4.86748	6.63946	8.09423	9.43962
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	42.00	54	83	111	137	168	230	292	403	505	601
Ditto corrected												
Load (KN)	0	0.4594	0.59066	0.90787	1.21413	1.49853	1.83761	2.51577	3.19394	4.40807	5.52377	6.57383
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator		95.00	115	131	146	162	176	204	230	272	308	333
Ditto corrected												
Load (KN)	0	1.03912	1.25789	1.4329	1.59697	1.77198	1.92511	2.23138	2.51577	2.97518	3.36895	3.6424
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	60.00	91	153	230	305	350	372	377	382	390	406
Ditto corrected												
Load (KN)	0	0.65629	0.99537	1.67354	2.51577	3.33614	3.82835	4.06899	4.12368	4.17837	4.26588	4.44089
C. B. R. %												

	UNSOAKED		**SOAKED**	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	22.63	24.3861	13.3785	12.604
TOP	11.3139	16.0016	25.1878	20.6596

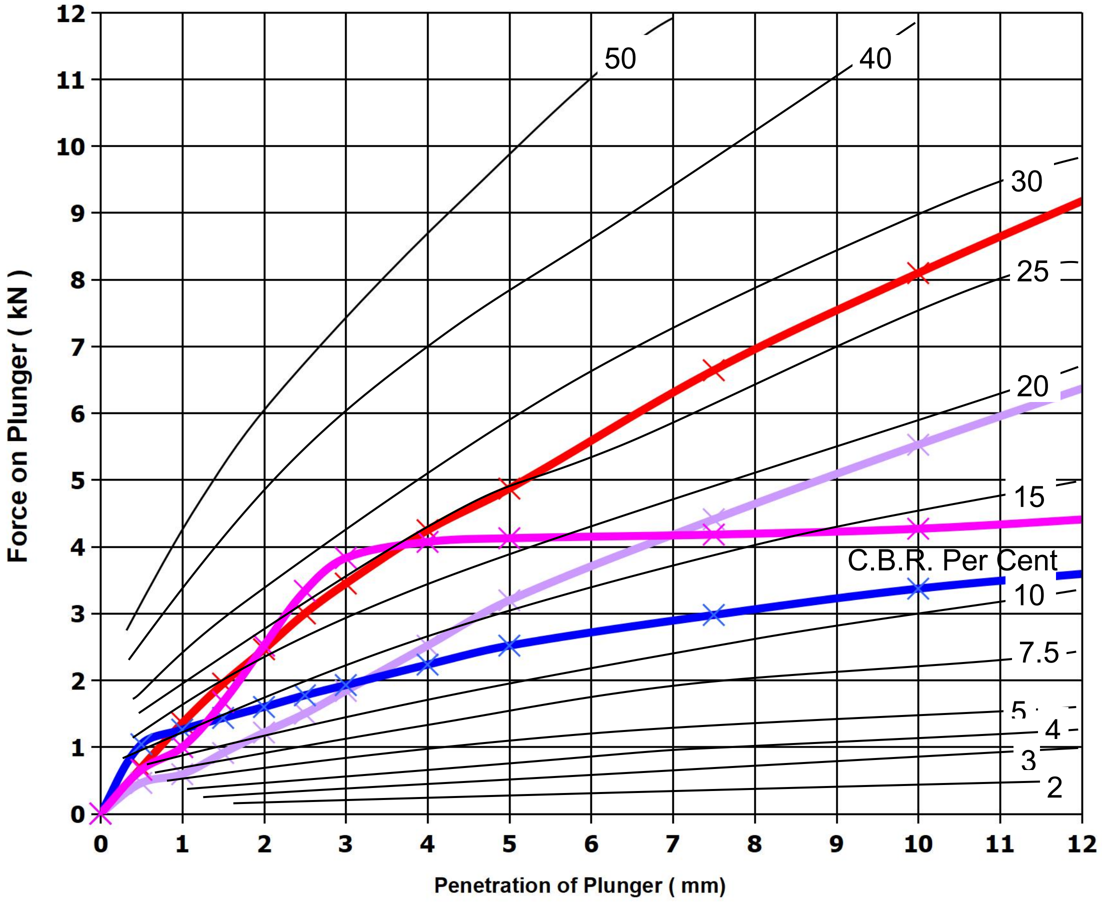


Fig 26: Graph of CBR test (Natural soil + 4% Orange Peel)

U: CBR: Natural soil Sample 1 and 6% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	43.00	78	100	122	157	182	234	277	374	479	675
Ditto corrected												
Load (KN)	0	0.47034	0.85318	1.09382	1.33445	1.71729	1.99074	2.55953	3.02987	4.09087	5.23937	7.38325
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	30.00	62	104	141	176	207	262	303	406	562	649
Ditto corrected												
Load (KN)	0	0.32814	0.67817	1.13757	1.54228	1.92511	2.2642	2.8658	3.31426	4.44089	6.14724	7.09886
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator		0.00	0	0	0	3	5	10	15	23	33	42
Ditto corrected												
Load (KN)	0	0	0	0	0	0.03281	0.05469	0.10938	0.16407	0.25158	0.36096	0.4594
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	13.00	13	15	30	40	50	61	66	75	83	91
Ditto corrected												
Load (KN)	0	0.1422	0.1422	0.16407	0.32814	0.43753	0.54691	0.66723	0.72192	0.82036	0.90787	0.99537
C. B. R. %												

	UNSOAKED		**SOAKED**	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	12.97	15.1796	0.24775	0.822
TOP	14.5346	16.6044	3.30332	3.61681

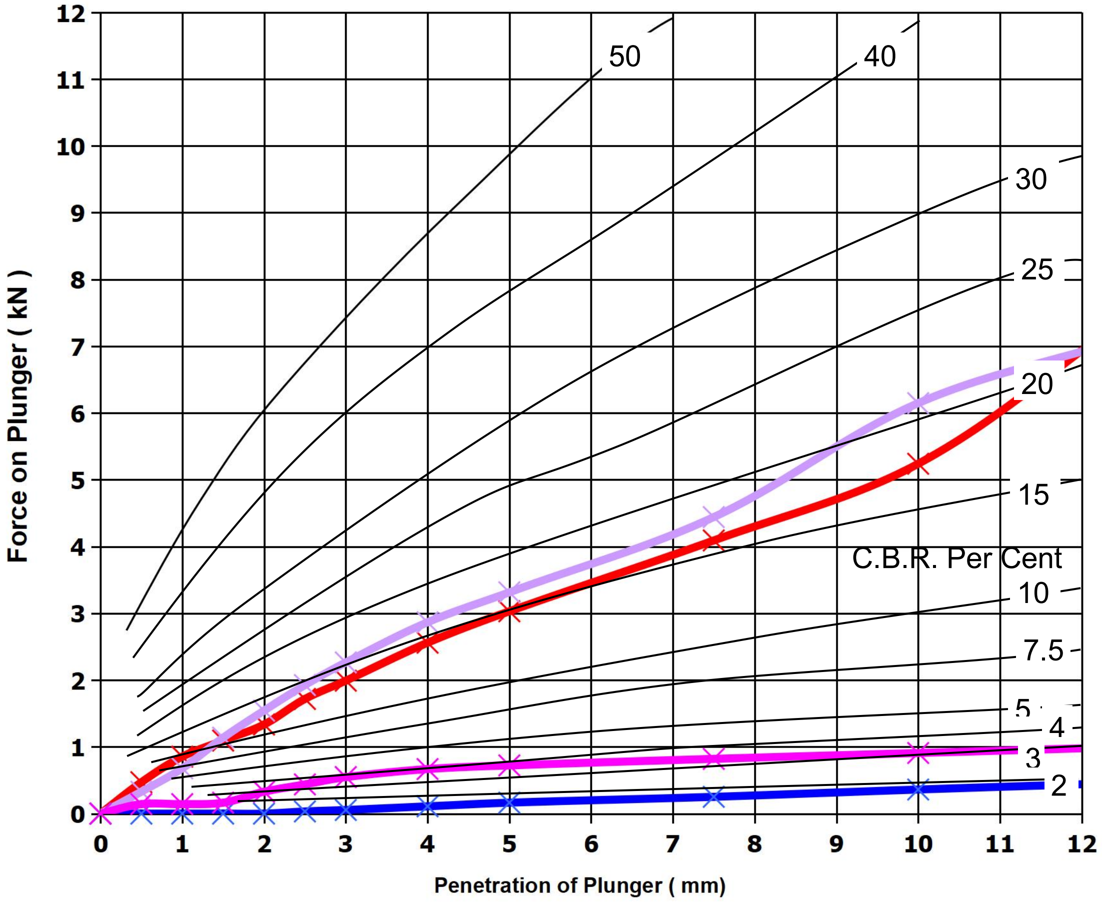


Fig 27: Graph of CBR test (Natural soil + 6% Orange Peel)

V: CBR**:** Natural soil Sample 1 and 8% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	195.00	305	384	468	533	591	698	790	997	1185	1585
Ditto corrected												
Load (KN)	0	2.13294	3.33614	4.20025	5.11905	5.83003	6.46445	7.63483	8.64114	10.9053	12.9617	17.337
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	111.00	142	225	270	318	365	443	516	687	864	1109
Ditto corrected												
Load (KN)	0	1.21413	1.55322	2.46108	2.9533	3.47833	3.99242	4.8456	5.64409	7.51451	9.45056	12.1304
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator		47.00	50	52	55	57	62	70	87	49	64	85
Ditto corrected												
Load (KN)	0	0.51409	0.54691	0.56878	0.6016	0.62347	0.67817	0.76567	0.95162	0.53597	0.70004	0.92974
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	26.00	35	51	64	74	81	92	100	117	133	145
Ditto corrected												
Load (KN)	0	0.28439	0.38284	0.55785	0.70004	0.80942	0.88599	1.00631	1.09382	1.27976	1.45477	1.58603
C. B. R. %												

	UNSOAKED		**SOAKED**	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	44.02	43.2921	4.70723	4.76761
TOP	26.2614	28.2769	6.11114	5.48001

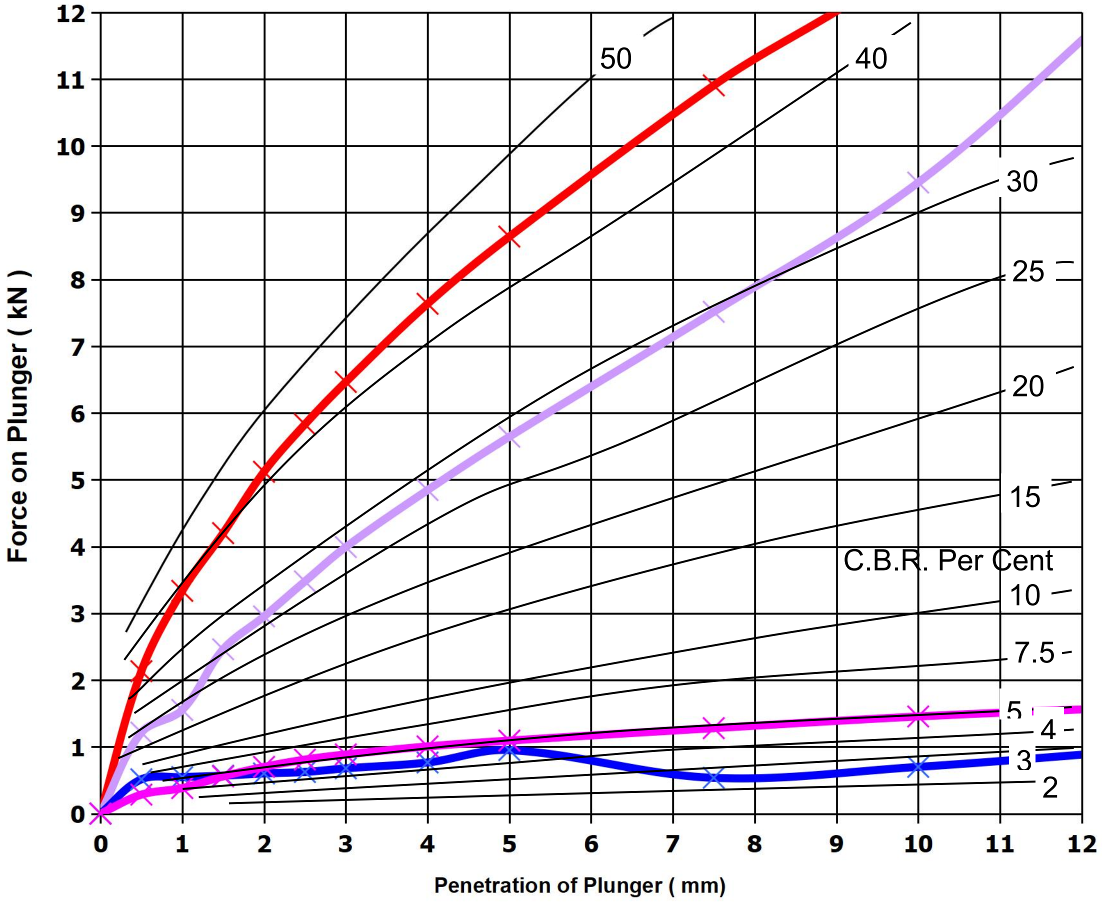


Fig 28: Graph of CBR Test (Natural soil + 8% Orange Peel)

W: Result of CBR on Soil Sample 2 without stabilizers

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	85.00	148	217	268	309	349	420	484	629	735	845
Ditto corrected												
Load (KN)	0	0.929771	1.61885	2.37358	2.93142	3.37989	3.81741	4.59402	5.29406	6.8801	8.03954	9.24271
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	53.00	72	91	122	168	215	312	386	548	670	831
Ditto corrected												
Load (KN)	0	0.57972	0.78755	0.99537	1.33445	1.83761	2.3517	3.4127	4.22213	5.99411	7.32856	9.0896
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator		20.00	25	36	46	48	52	60	65	72	83	98
Ditto corrected												
Load (KN)	0	0.21876	0.27345	0.39377	0.50315	0.52503	0.56878	0.65629	0.71098	0.78755	0.90787	1.05006
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	77.00	135	171	180	183	194	194	194	194	194	202
Ditto corrected												
Load (KN)	0	0.84224	1.47665	1.87042	1.96887	2.00168	2.122	2.122	2.122	2.122	2.122	2.19858
C. B. R. %												

	UNSOAKED		SOAKED	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	25.53	26.5233	3.96398	3.56203
TOP	13.8738	21.1528	15.1128	10.6313

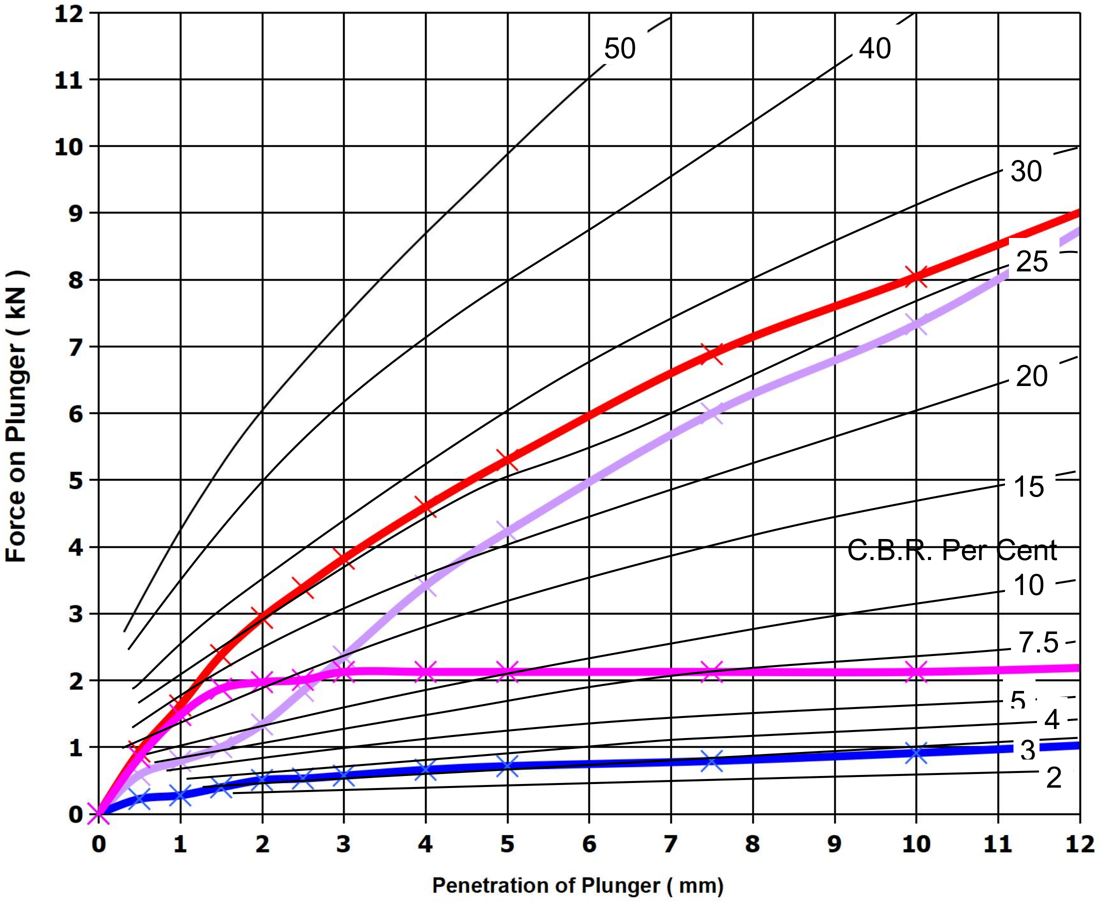


Fig 29: Graph of CBR Test (Control sample 2)

X: CBR**:** Natural soil Sample 2 and 2% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	90.00	140	180	232	302	356	437	501	645	756	863
Ditto corrected												
Load (KN)	0	0.98443	1.53134	1.96887	2.53765	3.30332	3.89398	4.77997	5.48001	7.05511	8.26924	9.43962
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	58.00	85	117	157	191	222	280	331	439	554	658
Ditto corrected												
Load (KN)	0	0.63441	0.92974	1.27976	1.71729	2.08919	2.42827	3.06268	3.62053	4.80185	6.05974	7.1973
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	47.00	59	66	72	77	81	87	94	109	125	142
Ditto corrected												
Load (KN)	0	0.51409	0.64535	0.72192	0.78755	0.84224	0.88599	0.95162	1.02819	1.19226	1.36727	1.55322
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	21.00	39	40	51	63	74	97	115	152	177	198
Ditto corrected												
Load (KN)	0	0.2297	0.42659	0.43753	0.55785	0.6891	0.80942	1.061	1.25789	1.6626	1.93605	2.16575
C. B. R. %												

	UNSOAKED		**SOAKED**	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	24.94	27.4549	6.35889	5.15121
TOP	15.7734	18.1388	5.20273	6.30202

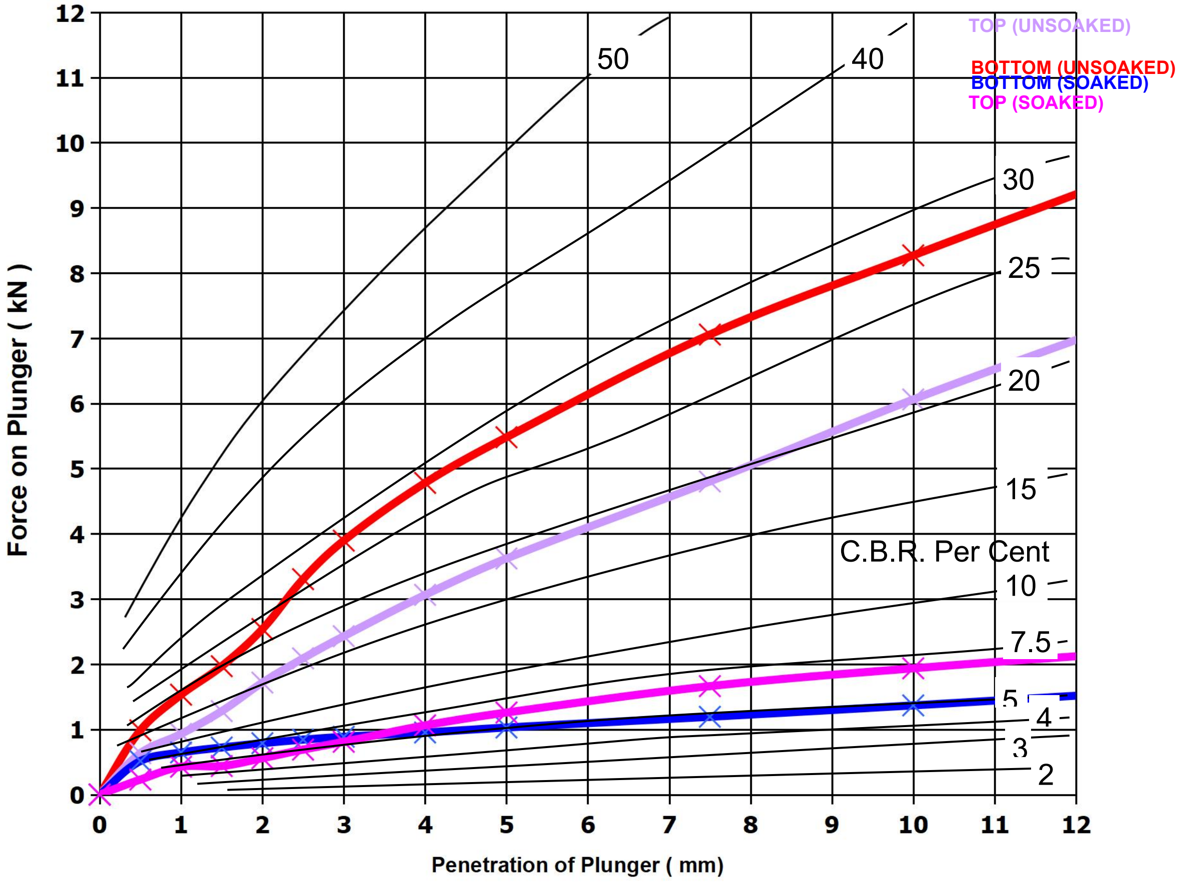


Fig 30: Graph of CBR Test (Natural soil + 2% Orange Peel)

Y: CBR: Natural soil Sample 2 and 4% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	122.00	162	205	244	275	309	362	427	567	598	847
Ditto corrected												
Load (KN)	0	1.33445	1.77198	2.24232	2.66891	3.00799	3.37989	3.95961	4.67059	6.20193	6.54101	9.26461
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	116.00	153	182	208	235	257	302	342	427	548	648
Ditto corrected												
Load (KN)	0	1.26883	1.67354	1.99074	2.27514	2.57047	2.8111	3.30332	3.74085	4.67059	5.99411	7.08792
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	29.00	34	37	41	44	47	52	54	64	68	70
Ditto corrected												
Load (KN)	0	0.31721	0.3719	0.40471	0.44846	0.48128	0.51409	0.56878	0.59066	0.70004	0.74379	0.76567
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	40.00	57	74	90	102	115	131	143	165	178	196
Ditto corrected												
Load (KN)	0	0.43753	0.62347	0.80942	0.98443	1.11569	1.25789	1.4329	1.56416	1.80479	1.94699	2.14388
C. B. R. %												

	UNSOAKED		**SOAKED**	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	22.71	23.3997	3.63365	2.95921
TOP	19.407	18.7416	8.42347	7.83642

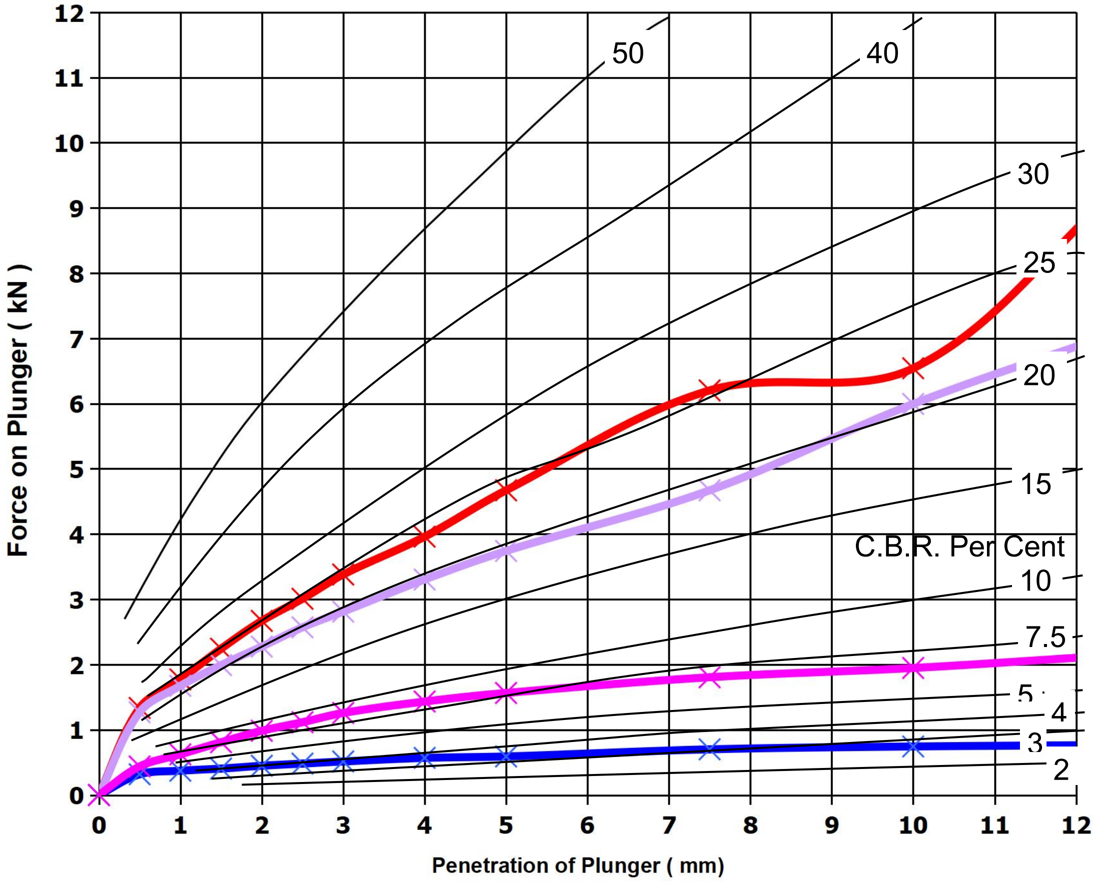


Fig 31: Graph of CBR Test (Natural soil + 4% Orange Peel)

Z: CBR Test: Natural soil Sample 2 and 6% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	127.00	184	234	276	313	345	401	453	573	684	786
Ditto corrected												
Load (KN)	0	1.38915	2.01262	2.55953	3.01893	3.42364	3.77366	4.3862	4.95498	6.26756	7.48169	8.59739
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	91.00	135	171	197	223	243	282	318	392	472	552
Ditto corrected												
Load (KN)	0	0.99537	1.47665	1.87042	2.15482	2.43921	2.65797	3.08456	3.47833	4.28775	5.16281	6.03786
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	20.00	26	30	34	37	39	43	47	54	58	64
Ditto corrected												
Load (KN)	0	0.21876	0.28439	0.32814	0.3719	0.40471	0.42659	0.47034	0.51409	0.59066	0.63441	0.70004
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	65.00	75	82	86	89	91	95	98	106	112	122
Ditto corrected												
Load (KN)	0	0.71098	0.82036	0.89693	0.94068	0.9735	0.99537	1.03912	1.07194	1.15944	1.22507	1.33445
C. B. R. %												

	UNSOAKED		**SOAKED**	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	25.85	24.8245	3.05557	2.57561
TOP	18.416	17.4264	7.34989	5.37041

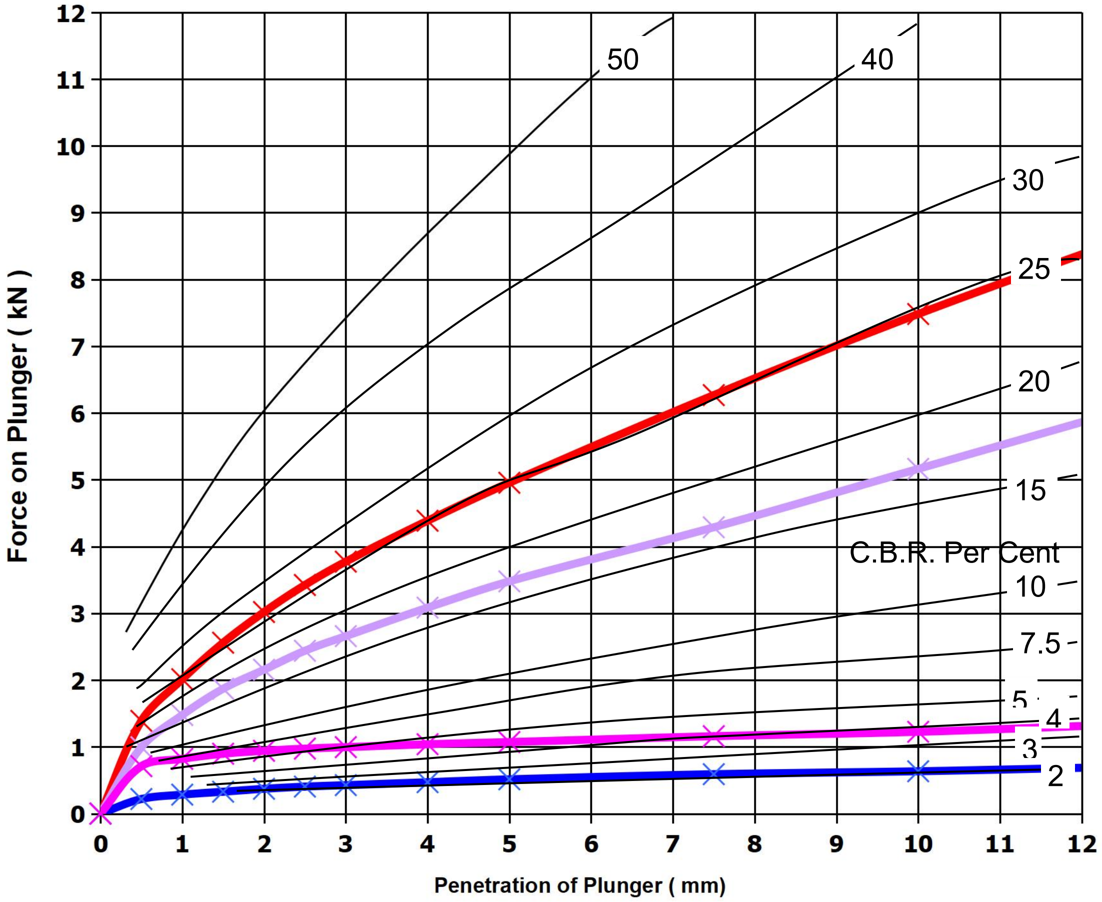


Fig 32: Graph of CBR Test (Natural soil + 6% Orange Peel)

Z1: CBR: Natural soil Sample 2 and 8% Orange Peel

TEST ON BOTTOM (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	101.00	149	193	225	261	293	349	392	493	574	661
Ditto corrected												
Load (KN)	0	1.10475	1.62978	2.11106	2.46108	2.85486	3.20488	3.81741	4.28775	5.39251	6.2785	7.23012
C. B. R. %												

TEST ON TOP (UNSOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	68.00	95	120	153	178	197	238	274	344	431	509
Ditto corrected												
Load (KN)	0	0.74379	1.03912	1.31258	1.67354	1.94699	2.15482	2.60328	2.99705	3.76272	4.71434	5.56752
C. B. R. %												

TEST ON BOTTOM (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	42.00	52	54	57	60	63	66	71	80	88	92
Ditto corrected												
Load (KN)	0	0.4594	0.56878	0.59066	0.62347	0.65629	0.6891	0.72192	0.77661	0.87505	0.96256	1.00631
C. B. R. %												

TEST ON TOP (SOAKED) **SURCHARGE**

Penetration (mm)	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	7.50	10.00	12.50
Load Indicator	0	97.00	107	109	110	110	111	113	116	120	124	138
Ditto corrected												
Load (KN)	0	1.061	1.17038	1.19226	1.2032	1.2032	1.21413	1.23601	1.26883	1.31258	1.35633	1.50946
C. B. R. %												

	UNSOAKED		**SOAKED**	
	2.5mm	5.0mm	2.5mm	5.0mm
BOTTOM	21.55	21.4817	4.95498	3.89081
TOP	14.6998	15.0152	9.08413	6.35682

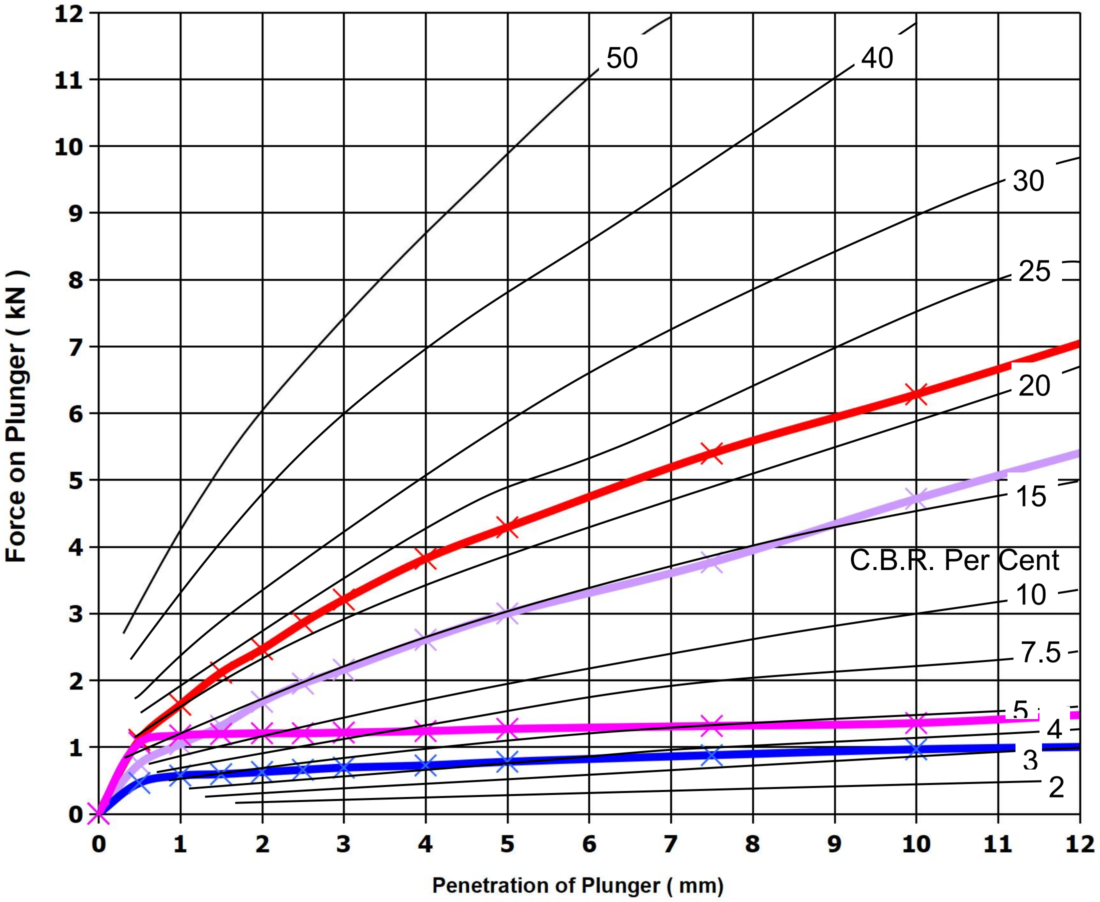


Fig 33: Graph of CBR Test (Natural soil + 8% Orange Peel)