DEVELOPMENT OF AN EMPIRICAL MODEL FOR CLAY SOIL STABILIZED WITH RECLAIMED ASPHALT PAVEMENT

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MEng/SEET/2017/6793

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AUGUST, 2021

ABSTRACT

A-6 lateritic clay is a weak soil, which requires some forms of stabilization and strength enhancement. The clay soil used in this research work was obtained from predetermined depths. It was first stabilized and then compacted with 0, 10, 20, 30, 40 and 50% of Reclaimed Asphalt Pavement respectively. The compaction parameters were correlated with the strength developed by clay soil stabilised with Reclaimed Asphalt Pavement. The compaction results of clay-RAP mix ranged from 0.553 - 1.995 g/cm^3 for MDD, with 7.9 to 4.10% for OMC using British heavy standard. The CBR values of unsoakedsamples ranged between 11% and 19.3%. Hence, using OMC as independent variable, an empirical model, MDD = - 0.348x+3.094 was developed using Microsoft Excel 2017. The model was satisfactory for sub-grade application. Also, the CBR value of 17.2 – 19.3% is satisfactory for both sub-grade use according to the Nigerian General Specification (1997). The Model gave an excellent strength correlation R^2 of 0.97.

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$\mathcal{H}$	The Percentage Finer for any Given Size
ς	The Specific Gravity of the Soil
$\mathcal{R}$	Corrected Hydrometer Reading
$\mathcal{M}_s$	Total Mass of the Soil
D_{60}	Particles with Diameter 60% Finer
D_{30}	Particles with Diameter 30% Finer
D_{10}	Particles with Diameter 10% Finer

CHAPTERONE

1.0 INTRODUCTION

1.1 Background to the Study

In civil engineering, soils with properties that cannot be safely and economically used for the construction of civil engineering structures without adopting some stabilization measures are known as problem soils. They are expansive and collapsible soils. To the geotechnical and highway engineers, a problem soil is one that poses problem to construction. Such problems may be as a result of instability of the soil which makes it unsuitable as a construction material in foundations, buildings, highways, water retaining structures and dams (Ola, 1987). Clay is predominant in most of the subgrade soil materials of Nigeria. The clay minerals attract and absorb water, thereby making it highly susceptible to swelling and shrinkage respectively.

Soils are classified into eight groups, A-1 through A-8. The major groups A-1, A-2, and A-3 represent the coarse grained soils and the A-4, A-5, A-6, and A-7 represent fine grained soils. A-8 are identified by visual inspection. A-6 group of soil classification system, includes those materials which have high plasticity indexes in relation to liquid limit and which are subject to extremely high volume change (AASHTO, 1986).

Reclaimed asphalt pavement (RAP) is an existing asphalt mixture that has been pulverized, usually by milling aggregate in recycled asphaltic pavement (Jeff and Miles, 2006). During pavement rehabilitation and reconstruction, large quantities of this materials are generated especially when asphalt pavement are removed. RAP is the term given to reprocessed and/or removed pavement materials containing asphalt and aggregates. These materials are generated when asphalt pavements are removed for reconstruction, resurfacing, or to obtain access to buried utilities. The binder in the RAP after several years of service, becomes aged and much stiffer than desired. Experience has indicated that the recycling of asphalt pavements is a beneficial approach from technical, environmental, and economical perspectives (Chen, 2007). This has made the recycling of pavement materials to become a very viable alternative to be considered in road maintenance and rehabilitation with the conservation of resources, preservation of the environment, and retention of existing highway geometrics; are some of the other benefits obtained by reusing pavement materials (Taha, 2002).

In Nigeria however, RAP recovered during highway reconstructions and rehabilitations are kept along road alignments and the statistics of the amount of RAP recovered is not documented. The use of waste materials, particularly RAP in the construction of pavements has benefits in not only reducing the amount of waste materials requiring disposal but can also provide construction materials with significant savings over new materials (Schroeder, 1994). Hence, the use of RAP can actually provide value to what was once a costly disposal problem. Initially, this recycling was limited to the re-use of materials removed from previous pavement structures such as: recyclable asphalt pavement, recyclable portland cement concrete and various base course materials but recently various other materials, not originating or associated with pavements, have come into use, either as additives or pozzolan to improve the particle size distribution, physical, chemical, engineering and mechanical properties of RAP (Hanks and Magni, 1989; FHWA, 2008; Chen, 2007).

However, the effect of RAP on Clay Soil, using a linear regression analysis model to determine the strength characteristics of the lateritic soil RAP mix; by carrying out a CBR test at different percentages is intended in this work. This will be very essential for Geotechnical engineers on proper application of RAP in the stabilization of Clay soil as material in pavement construction. Hence, the linear regression analysis shall reveal the strength characteristics of the soil RAP mix; at different percentages of the CBR test to be carried out.

1.2 Statement of the Research Problem

In geotechnical engineering, soil stabilization and improvement primarily aim at achieving lateritic clay soil strength properties that will ensure the durability and safety of civil engineering projects founded on soils (Onyelowe 2017a; Das, 2006 and Soban, 2012). In the developing world, for instance, road pavement decay resulting from failed subgrade and sub base layers is one of the major problems militating against traffic facilities (Fwa, 2006). The road sections comprises of the sub-grade, sub-base and base. Hence, this intended soil stabilization technique and model is expected to improve the bearing capacity of the lateritic clay soil before structures are constructed or pavement layers are laid upon the sub grade soil.

1.3 Aim and Objectives of the Study

The aim of this research work is to develop a linear regression analysis model of Clay Soil stabilized with Reclaimed Asphalt pavement. To achieve this aim, the objectives of this work are to;

- Obtain the index properties of lateritic soil; and
- Obtain the compaction characteristics of lateritic soil and lateritic soil RAP mix; and
- Obtain the California bearing ratio of lateritic soil RAP mix at 10%, 20%, 30%, 40% and 50%; and
- Develop a linear regression analysis model for the lateritic soil RAP mix.

1.4 Scope of the Study

The laboratory Test was carried out at the Federal University of Technology Civil Engineering department laboratory. The following preliminary tests was conducted which include; Sieve Analysis Test in accordance with BS 1377-2 (1990); Compaction Test (Standard Proctor Test) in accordance with BS 1377-2 (1992); California Bearing Ratio Test (CBR) in accordance with

BS 1377-2 (1992); at different mix percentages of 10, 20, 30, 40 and 50%, Atterberg Limit Test; shall be conducted using a 2013 Cassagrande apparatus, to get the index properties of Lateritic soil and Reclaimed Asphalt pavement. Finally, the Regression Analysis was done with Microsoft Excel 2017.

1.5 Justification of the Study

This research work is relevant to Geotechnical Engineers, having a vital role in predicting the strength gotten from a lateritic RAP mix by stabilization technique which depends on percentage addition of RAP. Hence, the need to consolidate on relevant thesis like "Prediction of Unconfined Compressive strength of a Stabilized Expansive Clay Soil using ANN and Regression Analysis (SPSS)" is quite important and necessary. However, this thesis would be of additional knowledge in order to reduce the time consuming process of laboratory investigation and for effective utilization of RAP.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Problematic Soil

In developing countries, the biggest handicap to provide a complete network of road system is the limited finances available to build road by the conventional methods. Therefore, there is a need to resort to one of the suitable methods of low cost road construction, followed by a process of stage development of the roads, to meet the growing needs of the road traffic. Thus, apart from affecting economy in the initial construction cost of lower layers of the pavement such as sub-base course it should be possible to upgrade the low cost roads to higher specification at a later date without involving appreciable wastage, utilizing the principle of pavement construction in stages. The construction cost can be considerably reduced by selecting local materials including soils for the lower layers of the pavement such as the sub-base course. If the stability of the local soil is not adequate for supporting wheel loads, the properties are improved by soil stabilization (Khanna and Justo, 2001).

The stability and serviceability of most engineering project or structure depends largely on their foundation and the bearing capacity of the soil that supports them (Chesworth, 2008). The stability and strength of structures would affect standard as stipulated in the engineering code of practices. A road pavement may serve its intended purpose if the foundation or sub grade meets the minimum standard in the highway codes. From Road Note 29 (1970), a subgrade with a California bearing ratio (CBR) of 2% or less is termed a weak material, while those with CBR values between 3 –15% are normal and those from 15% and above are said to be very stable.

However, Nigerian General Specification for Roads and Bridges in Nigeria (1997) however, recommended a minimum CBR value of over 80% for base materials, 30-80% for sub-bases and 10-30% for subgrade. Massive road construction has depleted once plentiful aggregate supplies and continuing to exhaust the valuable resources to rebuild existing roads only propagates and accelerates the problem (Hanks and Magni, 1989).

The soil particles of Clay Soils are extremely fine, they are 0.02mm in diameter. The particles are closely packed, and does not allow much "pore space" within the soil (Craig, 1992; Das, 2006). Common occurrences worldwide particularly the persistent difficulties in road construction are due to the expansive nature of clay soils. The common problem is the volumetric change associated with such clay soils when subjected to water content. In the light of the maintenance cost that will follow the road repairs accentuated by the presence of the expansive clay, methods of treatment must be evolved to eliminate or reduce the effect of soil volume change on the overall formation of the road structure (Bell, 1996; Nalbantoglue, 2000; National Lime Association, 2001).

2.2. Mathematical Modeling

Derived from its latin root “modus”, the word model is generally understood to stand for an object that represents a physical entity with a change of state or an abstract representation of real world processes, systems or sub-systems (Edwards and Hamson, 1989; Kapoor, 1993). Cheema (2006) defined a model as a representation of the essential aspects of an existing system or a system to be constructed which presents the knowledge of that system in a useable form. Thus, Box and Draper (1987) said that a model is a simplified depiction used to enhance our ability to understand, explain, change, preserve, predict and possibly control behaviour of a system which may be in existence or still awaiting execution. Nwaogazie (2006) referred to model as an imitation of something on a smaller scale and also that mathematical model stands as a mathematical representation of a set of relationship between variables and parameters. In case of an existing system, a model intends to improve on its performance, while it explores to identity the best structure/properties of a future system (Agunwamba, 2007). The trend of modeling is to collate existing records (data), establish relationships via mathematical equation(s), and calibrate the equation with experimental results and adopting such equation for forecasting and prediction. Prediction looks into the future for decision-making.

There is nothing mysterious about models; a house wife's shopping list, photographs, maps, organization charts, accounting statements and globe for earth planet are examples of ubiquitous use of a model. This is because each of them partially represents realities, simplifies complexities/uncertainties and portrays essential features of the represented systems in their own logical structure that is amenable to formal analysis (Kapoor, 1993), Models are broadly grouped into two; physical and mathematical models (Agunwamba, 2007).

A physical model is a three dimensional representation of an object which is tangible and made to look and perform like the system or some aspects of the system under study (Kapoor, 1993; Cheema 2006). Physical models are sometimes called iconic model because they are actually constructed and may be larger or smaller or identical in size to the object they represent (Kapoor, 1993). Physical models are very important in the development and analysis of complex engineering systems and processes such as ships, automobiles, aircrafts and complex chemical plants and so on. This is due to mathematical inputs of the boundary configuration and characteristics of these systems. Therefore, it is possible to select optimum design parameters of complex engineering systems by constructing and monitoring the performance of their physical models (Agunwamba, 2007).

A mathematical model is a simplified representation of a system or certain aspects of a real system, created using mathematical concept such as functions, graphs, diagrams/maps and equations to solve problems in the real world (Edwards and Hamson, 1989; Cheema, 2006), it is usually referred to as process model and can take many forms, including but not limited to algebraic equations, inequalities, differential equations, dynamic systems, statistical and game models (Ike and Mughal, 1997). Kapoor., (1993) classified mathematical models as critical, empirical or semi-empirical models based on how they are derived. Mathematical models can also be classified intoseveral other ways such as linear versus non-linear, static versus dynamic or steady versus non-steady, deterministic versus stochastic or probabilistic, lumped parameters versus distributed parameters, empirical versus mechanistic, continuous versus discrete, black-box versus white- box models (Aris, 1994; NIST/SEMATECH, 2006).

Critical models are developed using the principle of scientific laws. In empirical models the output is related mathematically to the input and a mathematical relationship is established between the two base on observed or experimental data from the system under study. Semi-empirical models are developed from a compromise between critical and empirical models with one or more parameters to be evaluated from the data generated from the system under study (Kapoor, 1993).

In a linear model, the objective function and constraints are in a linear form while in a non-linear model, part or all of the constraints and/or the objective function are non-linear. For deterministic models each variable and parameter is assigned a definite fixed number or a series of fixed numbers for any set of conditions. When variables and parameters in a model are difficult to define with unique values it is quite problematic. Static model does not explicitly take a variable time into account while dynamic models do. In lumped parameter model, the various parameters and dependent variables are homogeneous throughout the system while a distributed parameter model takes account of variations in behaviour from point to point throughout the system (Aris, 1994).

Furthermore, a mathematical model may be simple or complex. Although representing a real system mathematically is usually a complex process due to the presence of several variables and uncertainty associated with physical problems, approximations are often used to obtain a more robust and simple models (Agunwamba, 2007). This is because as the degree of complexity increases, so do the amount of information, time and cost required to develop a model and interpret its outcome.

Mathematical model is generally written as (Myer, 1990; Montgomery *et al.,* 2001; NIST/SEMATECH. 2006):

$$\gamma = f\ (\varkappa : \beta) + \varepsilon \qquad (2.1)$$

Consequently, there are three main parts to every mathematical model;

i. Response variable usually denoted by γ.

ii. Mathematical function usually denoted by $f\ (\varkappa : \beta)$.

iii. Random error usually denoted by ε.

It is based on this that NIST/SEMATECH (2006) expressed mathematical modeling as a concise description of the total variation in one quantity Y, by partitioning it into deterministic and random components. The response variable simply called "the response" or "dependent" variable is a quantity that varies in a way that we hope to be able to summarize and exploit via the modeling process. Generally, it is known that the variation of the response variable is systematically related to the values of one or more other variables before the modeling process is begun, although testing the existence and nature of this dependencies part of the modeling process itself (Dean and Voss, 1999; Wu and Hamanda, 2000).

The mathematical function, sometimes referred to as the regression function, regression equation, smoothing function or smooth consists of two parts. These are the predictor variables x_1, x_2…… X_n and the parameters, Z_0, Z_1…….. Z_n (Montgomery, 1991; NIST/SEMATECH, 2006; Nuran., 2007). The predictor variables are input to the mathematical function and usually observed along with response variable. Other names for the predictor variables include explanatory variable, independent variable, predictors or regressors. The parameters are the quantities that are usually estimated during the modeling process. Their true values are unknown and unknowable except in simulation experiments. The parameters and predictors are combined in different forms to give the function used to describe the deterministic variation in the response variable. For example, a straight line with an unknown intercept and slope goes with two parameters and one predictor variable and its equation is as follows:

$$f(x{:}\beta) = \beta_0 + \beta x \tag{2.4}$$

A straight line with an unknown intercept and a known slope of one goes with one parameter and is represented in equation (2.5)

$$f(x{:}\beta) = \beta_0 + x \tag{2.5}$$

For quadratic surface with two predictor variables, the full model goes with six parameters as described in equation (2.6)

$$f(x\beta) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_{12} x_1 x_2 + \beta_{11} x_1 + \beta_{22} x_2 \tag{2.6}$$

The random errors are simply the difference between the data and the mathematical function. They are unknown and assumed to follow a particular probability distribution which is used to describe their aggregate behaviour. The random errors cannot be characterized individually and the probability distribution that describes the errors has a mean of zero and an unknown standard deviation which is another parameter. Mathematical models are the most commonly used model for scientific and engineering applications because they are versatile, and easier to generalize from model to real life (Cheema., 2006). In many cases, the models are used to explain known facts and lay a foundation for the theory behind their phenomena and that of ambiguous processes. It is a booming engineering tool for design and optimization of systems/processes because it provides an avenue for understanding qualitative and quantitative aspects of phenomena of interests and also facilitates access to optimum design/performance parameters of systems/processes (Kapoor, 1993; Cheema, 2006; Agunwamba*.,* 2007).

2.3 Mathematical Model-Building Techniques

The bottom line for all data analysis problems is the selection of most appropriate method to apply which largely depends on the goal of the analysis and the nature of the data. However, model building is different from most other areas of data analysis with regard to method selection because there are more general approaches and more competing techniques available for this than most other data analysis methods. There is often more than one technique that can be effectively applied to a given modeling application. The large menu of methods applicable to modeling problems means that there are more potentials to perform different analysis on a given problem thereby resulting to more opportunity of obtaining effective and more efficient solutions (NIST/SEMATECH, 2006).

Most modern mathematical modeling methods vary considerably in their details but, their essentials fall within one or more of Analytical-Optimization technique, Statistical technique, Probabilistic technique and Simulation-Search /Sampling technique. Each of these model-fitting methods is not exclusively independent in application, that's why all modern mathematical modeling techniques involve other scientific advances peculiar to them in addition to elements of one or more if not all of these basic methods. For instance, response surface analysis consists of experimental strategy for exploring the space of the process or independent variables, empirical statistical modeling to develop an appropriate approximating relationship between a response or responses and the process variables and optimization methods (often simulation-search/sampling or analytical-optimization techniques or both) for finding the values of the process variables that produce desirable values of the response (Myers., 1990; Lawson and Madrigal, 1994; Neddermeijer*et al.,* 2000; Nicolai *et al.,* 2004).

Analytical technique applies classical calculus and Lagrange multiplier as well as other mathematical programming techniques which may be linear, non-linear and dynamic in its study. Statistical techniques include such methods as statistical inference, decision theory and multi-variable analysis like least square regressions which may be linear, non-linear or weighted. Probabilistic techniques such as queuing and inventory theory are used for studying stochastic system elements by means of appropriate statistical parameters. Simulation-Search/Sampling techniques are the most widely used for scientific and engineering applications. Simulation is a descriptive technique that incorporates the quantifiable relationships among variables and describes the outcome of operating a system under a given set of inputs/operating conditions. If the objective function is defined, the values of the objective for several runs generate a response surface. The sampling or search process explores the response surface to determine near-optimal and optimal solutions (Kathleen *et al.,* 2004; Nuran, 2007).

2.4 Physical Properties of Clay Soils

Clay soils when dry have greater strength, and they are also susceptible to waterlogging due to poor infiltration which results in poor aeration (Mullins, 1990). A clay soil tends to stay wetter for longer as the fine particles hold more water tightly, than a sandy or loamy soil does. Critical to the inter-relation of soil type and the potential for structure alteration is a soil's Plastic Limit water content (PL). Soil wetter than the PL is compressed and sheared when loaded, that is the soil is in a “plastic” state and is prone to structure alteration that can create poor soil physical conditions for plant growth. Soil modelled drier than PL fractures rather than smears. Soil strength is one aspect of soil structure, which is controlled by factors including grading or particle size distribution. The grading itself is controlled by several factors including particle size distribution, the amount and type of clay mineral, and particle shape (Young and Mullins 1991; Zhang, 2001).

2.5 Swelling Characteristics of Clay Soils

The shrinking-swelling characteristics of soils is one of the geotechnical problems. It is observed on the plastical clay soils under the various environmental conditions. This phenomenon affects all the weight construction realized on these soils without the knowledge of their properties. The damages resulting from this phenomenon are expensive in repairing damaged structures (Vincent *et al.,* 2008) . The swelling characteristics of expansive soils are due to several parameters associated to the initial soil conditions; which are the initial dry density and initial moisture content (Komine and Ogata 1994). Nonetheless, the first factor which conditions the swelling of a clay soil and affects these physical-chemical and mechanical properties is the mineralogical composition; the quantity and type of minerals present in clay soils. It's known for example, that the phenomenon of swelling depends not only on the granularity, but also on the superficial activity of the clay itself which will be all the more intense as the specific area of particles will be high. A montmorillonite will swell more than illite and this exhibits for such phenomenon more than the kaolinite; the properties of the soil will be all more marked as the percentage of expansive minerals (montmorillonite) will be higher (Binod and Beena 2011). But in nature, the soil is very heterogeneous; it composition depends on other elements in addition to swelling minerals. Several mineralogical researches on natural soils show the considerable presence of swelling clay and non-swelling clay in addition to the presence of bentonite and silt (Homand *et al.,* 2001).

2.6 Soil Stabilization

Stabilization of soil can be seen as the process of blending and mixing materials with soil to improve certain properties of the soil. This process may include the blending of soils to achieve a required gradation or the mixing of commercially available additives that may alter gradation, texture or act as a binder for cementation of the soil (Aris, 1994). O'Flaherty (2002), referred to soil stabilization as any treatment (including technically i.e compaction) applied to a soil to improve its strength and reduce its vulnerability to water. Sherwood (1993), also defined soil stabilization as the alteration of properties of an existing soil to meet the specified engineering requirements especially the strength properties which are taken to mean the requirements for use in the various layers of road pavements. The main properties that may be required to be altered by stabilization are:

Strength: To increase the strength and thus stability and bearing capacity.

The volume stability: To control the swell-shrink characteristics caused by moisture changes.

Durability: To increase the resistance to erosion, weathering or traffic usage and **Permeability:** To reduce permeability and hence the passage of water through the stabilized soil.

The foregoing definition covered much of the properties of soils that might be desired for a deficient soil to possess before embarking on soil stabilization for improvement. However, alteration of properties of soils by the process of soil stabilization is not limited to those mentioned in the definition by Sherwood (1993). Some other properties like the Atterberg limits, soil grading among others could also be altered through the process of soil stabilization.

2.7 Soil Classification

Soils may be classified in a general way as cohesionless or cohesive or a coarse or fine grained. As the terms are too general and cover too wide a range of physical and engineering properties, additional refinement or means of classification is necessary to determine the suitability of a soil for a specific engineering purpose and to be able to convey this information to others in an understandable way. Numerous classification systems have been proposed in the past several decades, which are helpful and guide in classifying soils. The most commonly used are the AASHTO and the Unified Classification Systems (Bowles, 1992).

2.7.1 AASHTO Soil Classification System

American Association of State Highway and Transportation Officials (AASHTO) formerly the Bureau of Public Road System is used worldwide. The AASHTO classification system started with the then U.S. Bureau of Public Roads in the years 1927-1929 and the system was revised in 1945. It classifies soils into eight groups, A-1 through A-8, and originally required the following data:

- Grain-size analysis
- Atterberg limits

The table that was used for the classification is shown Table 1.0 and to establish the relative ranking of a soil within a subgroup, the group index is a function of the percent of soil passing sieve No. 200 and the Atterberg limits.

Table 2.1: AASHTO soil classification system Note that *A*-8, peak or muck, is by visual classification and is not shown in the Table

General classification	Granular materials (35% or less passing No. 200)							Silt-clay materials (More than 35% passing No. 200)			
	A - 1		A - 3	A - 2				A - 4	A - 5	A - 6	A - 7
Group classification	A-1a	A-1b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5; A-7-6
Sieve analysis: Percent passing: No. 10 No. 40 No. 200	max 30 max max	max max	min. max.	max	max.	max.	max.	min.	min.	min.	min.
Characteristics of fraction passing No: 40: Liquid limit: Plasticity index	max.		N.P.	max max	min. max.	max. min.	min. min.	max. max.	min. max.	max. min.	min. min.
Group index	0		0 0	0		max.		max.	max.	max.	max
Usual types o significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty Soils		Clayey soils	
General rating as subgrade	Excellent to good							Fair to poor			

Source: Bowels (1992)

2.7.2 The unified classification system

This system was originally developed for use in airfield construction and it had already been in use since about 1942, but was slightly modified in 1952 to make it apply todams and other construction. The soils are designated by group symbols consisting of a prefix and suffix. The prefixes indicate the main soil types and the suffixes indicate the subdivisions within the groups.

Table 2.2: Unified Classification System

Soil type	Prefix	Subgroup	Suffix
Gravel	G	Well graded	W
Sand	S	Poorly graded	P
Silt	M	Silty	M
Clay	C	Clayey	C
Organic	O	W_L< 50 percent	L
Peat	Pt	W_L > 50 percent	H

Source: Bowles (1992)

A soil is well-graded gravel or non-uniform if there is a wide distribution of grain sizes present, if there are some grains of each possible size between the upper and the lower gradation limits. This could be ascertained by plotting the grain-size curve and either observing the shape and spread of sizes or computing the coefficient of uniformity and coefficient of curvature. A poorly graded, or uniform, if the sample is mostly of one grain size or is deficient in certain grain sizes.

The unified classification system defines a soil as:

1. Coarse-grained if more than 50 percent is retained on the No.200 sieve.
2. Fine-grained if more than 50 percent passes the No. 200 sieve.

The coarse-grained soil is either:

1. Gravel if more than half of the coarse fraction is retained on the No. 4 sieve
2. Sand if more than half of the coarse fraction is between the No.4 and No.200 sieve

Classification of coarse-grained soils depends primarily on the grain-size analysis and particle size distribution. Classification of fine-grained soil requires the use of plasticity chart; each soil is grouped according to the coordinates of the plasticity index and liquid limit. On this chart an empirical line (the A line) separates the inorganic clays (C) from silts (M) and organic (O) soils. Although the silty and organic soils overlapped areas, they are easily differentiated by visual examination and odour.

2.8 Characteristics of Reclaimed Asphalt Pavement

Reclaimed asphalt pavement (RAP) is being broadly used as a component of asphalt mixture for new highway pavement. Many research studies indicate that there are plenty benefits of using RAP for the new asphalt mixture, including reduction of total cost in pavement construction, natural resource conservation, environment protection, and rutting resistance improvement (Newcomb and Brown, 2007; Swiertz and Mahmoud, 2011; Mogawer and Austerman, 2012; Xiao., *et al* 2016).

Hong, (2010) indicated that the resistance to rutting of hot mix asphalt (HMA) with 35% RAP was better compared to only virgin asphalt due to the incorporation of aged binder. Daniel, (2010) found that the high-temperature performance grade remains the same or increases only one grade for the various RAP percentages. Attia and Abdelrahman, (2010) found that the effect of moisture on RAP is similar to the effect of moisture on granular material. Dynamic Shear Rheometer (DSR) testing was used to evaluate recycled RAP and virgin asphalt and indicated that the shear modulus (G*) and the G*/sin δ increased with the increasing percentage of RAP at both the high and intermediate test temperatures (Roque, *et al* 2015). In addition, it also could be found that RAP sources were vital factors to determine the shear moduli and other DSR parameters (NCHRP; and Stroup-Gardiner, 2016).

Low-temperature cracking is predominant result of distress in asphalt pavements because of the thermal stress that builds up in pavements in extreme climates (Li., *et al* 2004). These low temperature cracks result in transverse cracks and other distresses along the pavement and ultimately accelerate the deterioration of the asphalt pavement structure. Some research studies indicated that the involvement of RAP material in new asphalt pavement might result in noticeable damage of pavement surface (NCHRP 2009; Xiao and Amirkhanian 2007; West, *et al.,* 2013). Therefore, the evaluation of fracture resistance for asphalt mixtures containing RAP is of interest to owners and agencies seeking better performing pavements in cold climates (Li., *et al.,* 2004).Some articles reported that low temperature bending beam rheometer (BBR) stiffness increased with increasing RAP, the m-value decreased with the increasing percentage of RAP, and the magnitude of the changes were dependent on the RAP source (Amirkhanian., *et al* 2016). In addition, it was reported that the critical low performance grade (PG) temperature increased with the increased RAP materials (Scholz *et al.,* 2010). Mogawer*et al.,* (2012) found that the RAP mixtures performed similarly to their respective control mixture for all low-temperature cracking tests. These data suggest that plant-produced mixtures with up to 30% RAP may not be more susceptible to low temperature failures. Swiertz *et al.* (2011) reported that when using RAP, the low-temperature PG grade depended on fresh binder grade and source. Testing also showed that RAP source was not a significant factor for dynamic modulus at low temperatures, although it significantly affected the dynamic modulus at high temperatures. The addition of 40% RAP also significantly decreased the low-temperature fracture resistance (Li., *et al.,* 2004).

However, the conventional methods of classifying aged asphalt binders from RAP materials requires initial extraction of the asphalt binder from the RAP, which involves the use of harmful chemical solvents such as trichloroethylene. In recent years, a new testing procedure has been developed to estimate the low temperature properties of the RAP binder without extraction or chemical treatments (Ma *et al.,* 2010). This project provides a possibility to evaluate the properties of RAP binders by testing the RAP mortars (fresh binders blended with fine RAP materials) without extracting the RAP binders from them. With the respect to testing procedure, the modified bending beam rheometer test is employed with minor modifications to the equipment which do not alter the test method and general settings. The properties of the binder in RAP are then estimated from the mortar properties. Many initial trials of the materials and equipment involved were performed before conducting the testing procedures to determine the low temperature properties of the aged binders in RAP materials (Xiao *et al.,* 2011). The reclaimed asphalt pavement is graded as it have some basic properties. This are shown in Table 2.3 below.

Table 2.3 Gradation and Basic Properties of RAP Samples.

CODE	**CIR-4.7%**	**CIR-5.1%**	**CIR-5.5%**	**CIR-5.7%**	**CIR-6.3%**	**Target Gradation**
1-inch	100	100	100	100	100	100
3/4-icnh	95	95	94	96	95	95±2
1/2-inch	83	80	89	75	80	--
3/8 inch	73	75	83	65	67	--
No. 4	51	50	52	5	50	50 ± 2
No. 8	31	29	38	22	27	--
No. 16	20	16	25	12	15	--
No. 30	11	10	12	8	9	10±2
No. 50	6	6	7	3	5	--
No. 100	3	3	5	1	3	--
No. 200	0.4	0.6	0.8	0.3	1.1	0.8 ± 0.3
AC%	4.7	5.1	5.5	5.7	6.3	--
Gmm	**2.432**	**2.457**	**2.411**	**2.392**	**2.400**	**--**

CHAPTER THREE

3.0 MATERIALS AND METHODS

3.1 Introduction

The soil used in this study was collected using the disturbed sampling technique at depths of between 0.5 m and 2.0 metres from borrowed pits around Talba farm area, a suburb of Minna, Niger State, Nigeria. The natural moisture content was determined after which it was air-dried.It is a lateritic soil later classified using index properties tests conducted. The reclaimed asphalt pavement used in this work was sourced from the ongoing Minna to Suleja road Dualization/rehabilitation Nigeria. The Reclaimed Asphalt Pavement (RAP) was sieved through sieve No 200 of the BS sieve to get very fine grain. It was thereafter collected in a container as shown in plate I. However, Excel is a commercial spreadsheet application produced and distributed by Microsoft for Microsoft Windows and Mac OS. It features the ability to perform basic calculations, create pivot tables and create macros, also use graphing tools as well. Excel permits users to arrange data so as to view various factors from different perspectives. Visual Basic is used for applications in Excel, allowing users to create a variety of complex numerical methods. Programmers are given an option to code directly using the Visual Basic Editor, including Windows for writing code, debugging and code module organization.

3.2 Characterization of the Lateritic Soil

Soils have peculiarities, they vary in properties. In other words, no two soils can be similar in all properties but can behave alike in some cases. Therefore, it is necessary to identify a soil and properly classify it to the group it belongs. This can be achieved by conducting preliminary tests on the natural soil. The following tests were conducted on the lateritic soil:

3.2.1 Moisture content determination

Empty aluminium cans which were properly identified with labels were weighed. Representative samples of wet soil were placed in the cans and weighed after which it placed in the oven at 110^{0}C for 20 to 24 hours. The moisture content is computed as follows:

$$\text{Water Content} = (w_2 - w_1) \times 100/(w_1 - w_3) \qquad (3.1)$$

Where;

w_1 = Weight of dry soil + can

w_2 = Weight of wet soil + can

w_3 = Weight of can

3.2.2 Liquid limit

This is water content above which the soil behaves like a viscous liquid (a soil- water mixture with no measurable shear strength). The liquid limit testing apparatus (Cassagrande apparatus) was used for the determination of liquid limit as recommended in BS 1377: Part 2: 1990. The soil was sieved with 425µm sieve and water added in successive stages (drier to wetter). A grooving tool of tip width 2mm – 0.25mm and a drop of liquid limit machine cup of 10mm were also used. The liquid limit is the water content at which 25 bumps close a groove of about 13mm length.

3.2.3 Plastic limit

This is the moisture content below which the soil no longer behaves as a plastic material. The plastic limit was determined as specified in BS 1377: Part 2: 1990.The sample was sieved through 425µm sieve and water was added to about 20g of the filtrate soil in order to mould it. The moulded lump of soil was broken into smaller samples and each of them rolled on a glass plate using the fingers to obtain a thread of uniform diameter (3mm). The plastic limit is described as the water content when a thread of soil being rolled shear at 3mm diameter (the first crumbling point or appearance of little cracks). If the plastic limit could not be attained in first rolling, the thread will be broken into several other pieces, reformed into a ball and re-rolled.

3.2.4 Particle size analysis

The particle size grading was carried out as specified in ASTM 1992. A set of stack sieve (apertures ranging from 4.75mm – 0.075mm) was used. A pulverized soil sample was washed on sieve No. 200 (0.075mm) and the residue soil was oven-dried. The oven-dried soil sample of known weight was put in the set of stack sieves and then placed on a mechanical shaker to sieve for about 10 minutes. The weight of the materials remaining on each sieve was noted and the percentage retained computed as a percentage of the total weight. The percentage passing and cumulative percentage passing were computed for each sieve. The suspension passing the sieve No.200 after washing into a 1000ml jar was taken for sedimentation test for silt and clay sized particles quantitative determination. Enough water was added to make 1000ml of suspension and the deflocculant sodium-metaphosphate was used. The suspension was mixed thoroughly by placing a bung on the open end of the jar and turning upside down and back few times. The jar was placed on the table. The hydrometer was inserted into the suspension to measure the specific gravity and a stop-watch was used to record time. The percentage settling at any given time was recorded with the Equation (3.2).

$$\varkappa = [\varsigma / (\varsigma - 1] \times [\mathcal{R}/\mathcal{M}s\ ] \times 100 \qquad (3.2)$$

Where:

$\varkappa$ = The percentage finer for any given size

ς =The specific gravity of the soil

$\mathcal{R}$ = Corrected hydrometer reading

$\mathcal{M}_s$= Total mass of the soil

Coefficient of Uniformity, $C_u = D_{60} / D_{10}$ (3.3)

Coefficient of Curvature, $C_u = D_{30})^2 / (D_{60} \times D_{10})$ (3.4)

Where;

D_{60} = Particles with diameter 60% finer

D_{30} = Particles with diameter 30% finer

D_{10} = Particles with diameter 10% finer

3.2.5 Compaction test:

This test is to determine the maximum dry density and the optimum moisture content with a given compactive effort. This test established the optimum moisture content to be used for some other performance test like california bearing ratio and the unconfined compressive strength, which requires compaction. As specified by BS 1377: 1990 (Standard Proctor) was adopted. A cylindrical metal mould (Proctor mould) of about 1000cm^3 volume and a rammer of 2.5kg weight with a height drop of 300mm was used as the given compactive effort. Twenty-five (25) blows were given on each layer of three (3) and moisture content samples were taken from the top and bottom of the mould. The optimum moisture content was taken as the moisture content at which the maximum dry density was attained. The dry density was obtained with the expression shown below in Equations (3.5) and (3.6) respectively.

$$\gamma = \mathcal{W}/\mathcal{V} \tag{3.5}$$

$$\gamma_d = \left[\frac{\gamma}{1 + (M/100)}\right] \tag{3.6}$$

Where,

γ = Bulk density

$\mathcal{W}$ = Weight of wet soil

$\mathcal{V}$ = Volume of wet soil

γ_d = Dry density of soil

M = Moisture content of soil in decimal fraction

3.2.6 California bearing ratio

The California bearing ratio (CBR) test is an empirical test developed by the California State Highway Department for the evaluation of subgrade strengths. In the test as given in BS 1377: Part 2: 1990, a specimen which is 127mm in height and 152mm in diameter is compacted into the CBR mould. The specimens were prepared in 5 (five) layers and heavy rammer was used to give 56 (fifty-six) blows onto each layer. The load required to cause a circular, 49.65mm in diameter, to penetrate the specimen at a specified rate of 1.25mm per minute is then measured. From the test results, the CBR value is calculated. This is done by expressing the corrected values of forces on the plunger for a given penetration as a percentage of a standard force. The 2.5mm and 5.0mm penetration caused by 13.24KN and 19.96KN loads were used in comparing the loads that caused the same penetration on the specimens. The CBR value for the lateritic soil was obtained.

3.3 Method for Formulation of Linear Regression Model

Regression was done manually directly from Microsoft Excel and also by encoding. Encoding helps the visual Based Analysis to run in a particular way you want, to achieve a purpose. Macros also known as Visual Based Analysis (VBA) was used to run a code generated to accommodate the accepted and specified values, and a button was created such that when clicked; the regression is generated instantly.

3.3.1 Objective function

The Empirical model has the capacity to accommodate 50 samples with instant results to show if the MDD and OMC of each sample is OK for sub-grade materials in accordance to Nigerian general specification on roads and bridges (1997). Likewise, with variables which has been incorporated into the model, the results generated automatically can be used to ascertain the strength characteristics of the lateritic soil RAP mix, when related to the CBR results which ranged between 11-19%.

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Properties of Natural Clay Soil

The index properties of the natural clay soil is shown in Table 4.1 and particle size distribution curve in Figure 4.1. The fraction passing No 200 sieve is 42.57% with liquid limit of 35.5%, plastic limit of 28.9% and plasticity index of 6.5%. The soil is classified as A – 6 according to AASHTO soil classification systems respectively (AASHTO, 1986).

Table 4.1: Properties of Natural clay Soil

Properties (Average)	Sample A
Natural moisture content of soil (%)	12.06
Atterberg Limits	
Liquid limit (%)	35.5
Plastic limit (%)	28.95
Plasticity index	6.5
% Passing BS No. 200 sieve	42.57
Classification	
USCS	CL
AASHTO	A-6

4.2 Compaction Characteristics and CBR Results of Test Sample

Table 4.2: Compaction and CBR results of Test Sample

RAP(%)	Compaction Characteristics		
	MDD (g/cm^3)	**OMC** (%)	**CBR** (%)
0%	0.553	7.90	11.1
10%	0.645	6.70	13.5
20%	0.890	5.50	15.5
30%	1.120	5.80	17.2
40%	1.510	4.70	18.8
50%	1.995	4.10	19.3

Sieve Analysis Curve

The lateritic soil particles Size Curve is shown in Figure 4.1

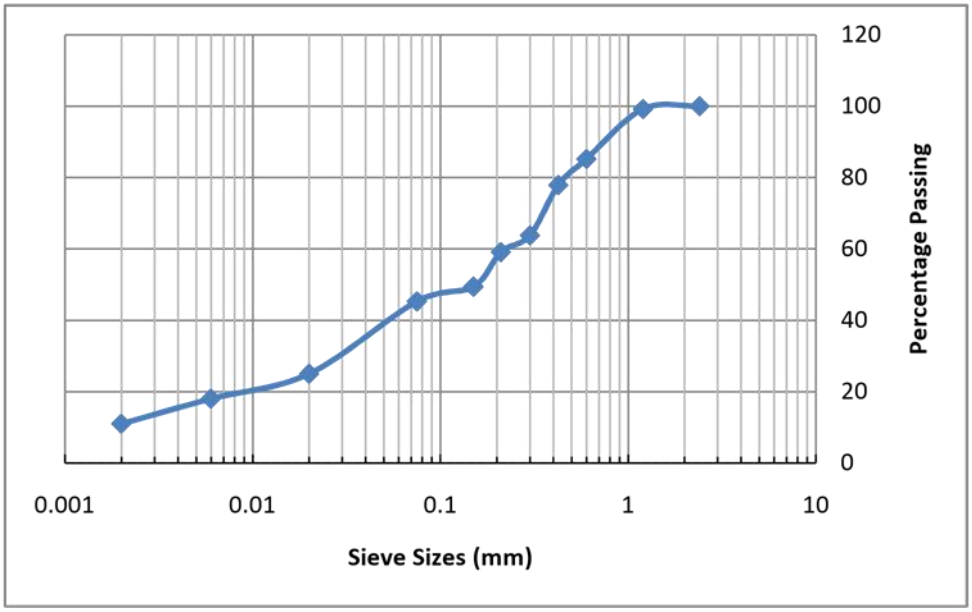


Figure 4.1: Particle Size Curve

4.2 Effect of RAP on Compaction Characteristics of Test Sample

Compaction test was carried out on the lateritic soil RAP mix at different percentages of Reclaimed Asphalt Pavement as shown from Figure 4.2 to Figure 4.7. It should be noted that four (4) samples were compacted for each RAP percentage content, before the average was considered. This makes it 24 samples that undergo compaction test. It was observed that for every percentage increase of RAP content, the MDD increases while the OMC decreases in value accordingly. This reveals the presence of RAP has increased cohesion by reducing the void spaces hereby having great effect on the geo-physical properties of the lateritic soil.

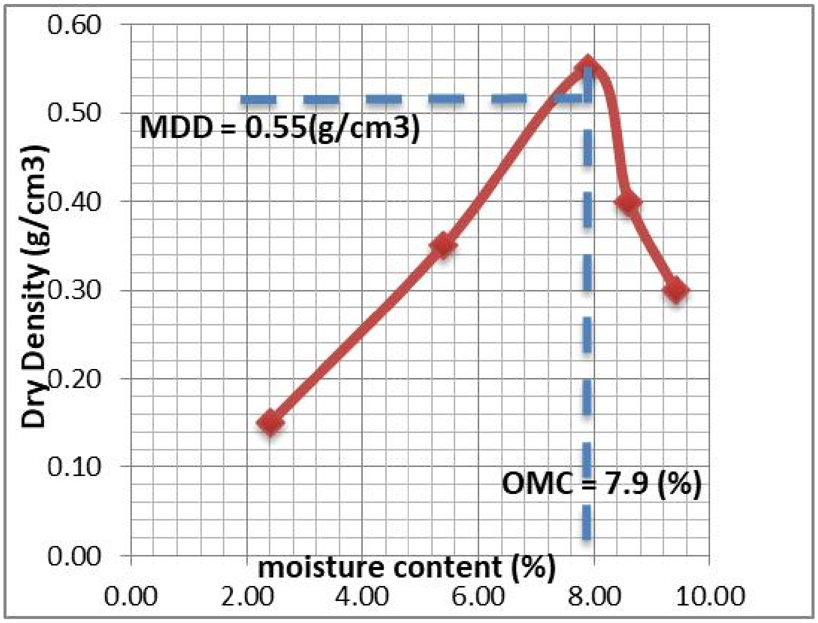


Figure 4.2: Compaction characteristics of stabilized clay 0% RAP

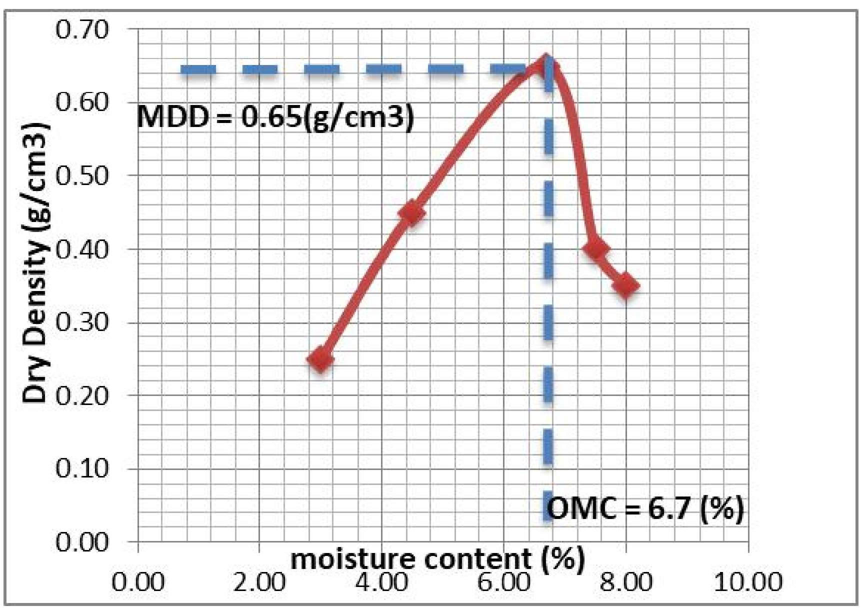


Figure 4.3: Compaction characteristics of stabilized clay 10% RAP

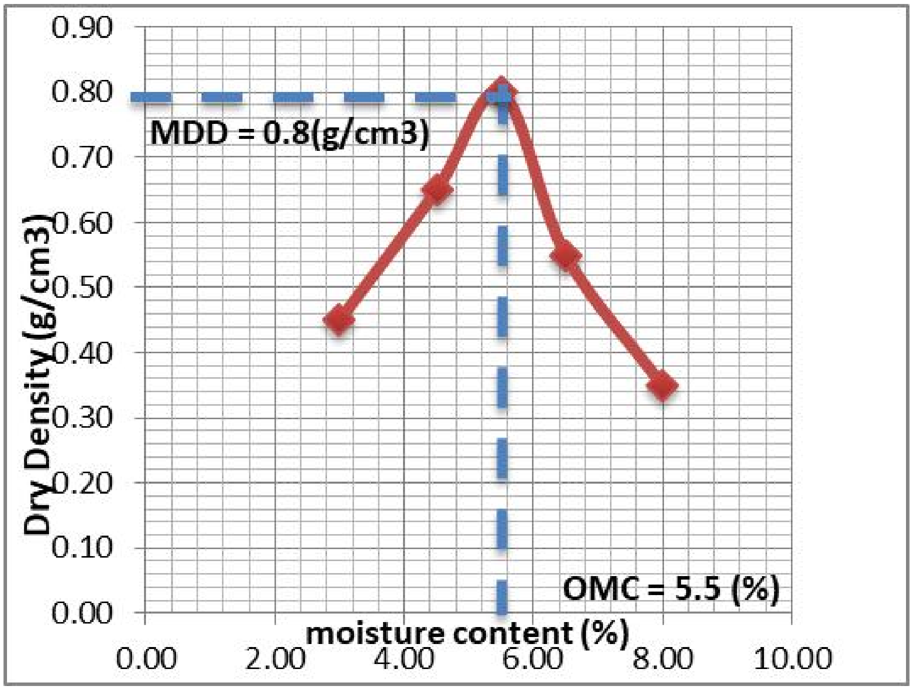


Figure 4.4: Compaction characteristics of stabilized clay with 20% RAP

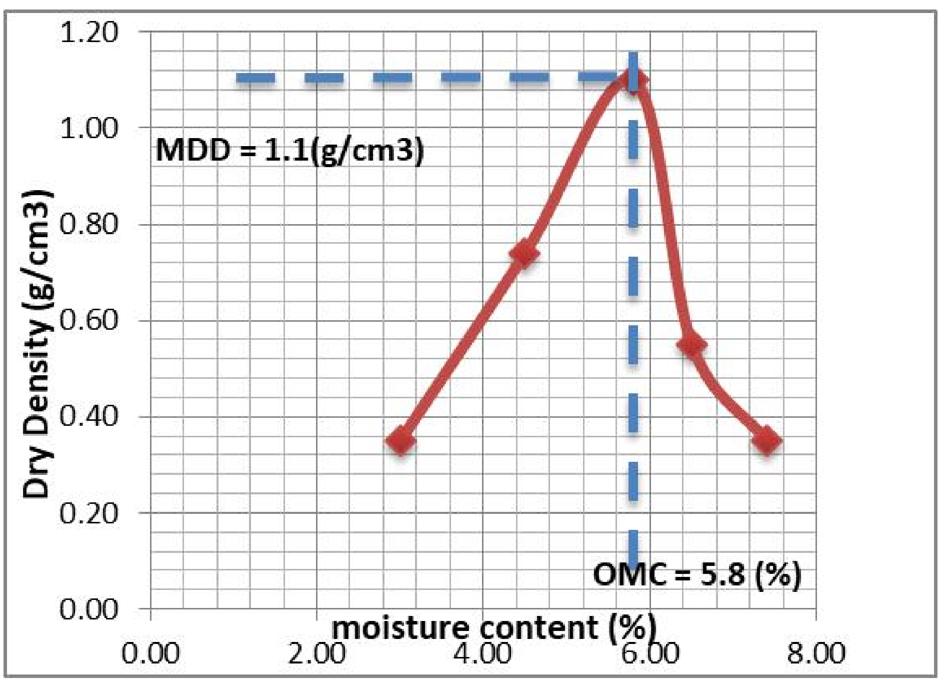


Figure 4.5: Compaction characteristics of stabilized soil with 30% RAP

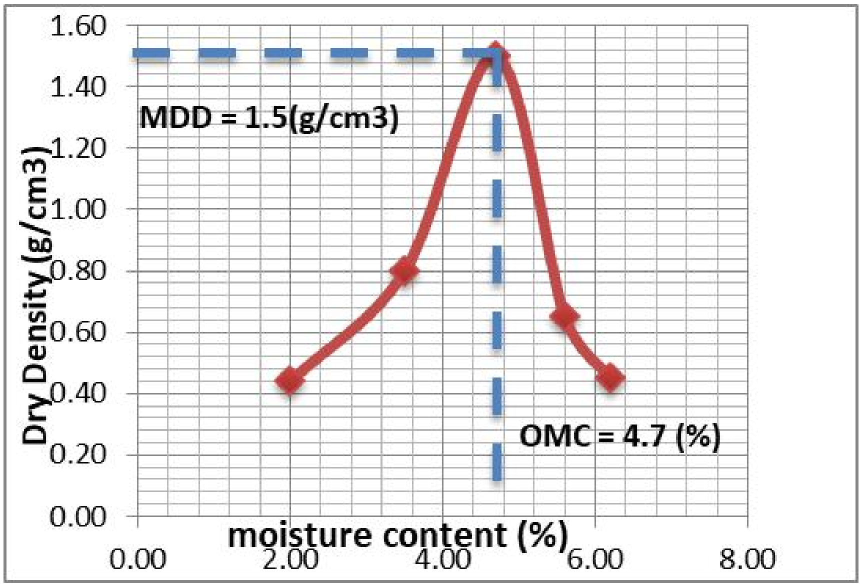


Figure 4.6: Compaction characteristics of stabilized soil with 40% RAP

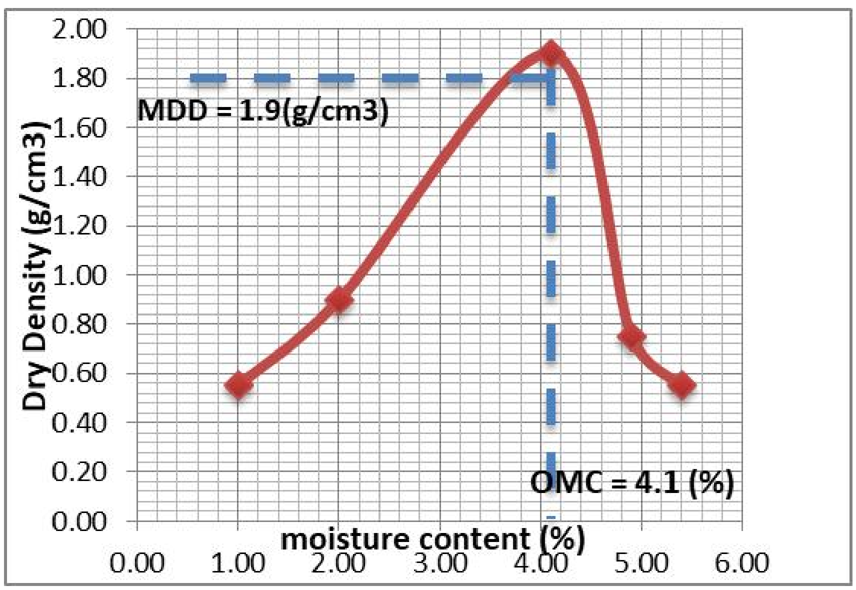


Figure 4.7: Compaction characteristics of stabilized soil with 50% RAP

4.3 Effect of RAP on California Bearing Ratio of Lateritic Soil

The California Bearing Ratio test was carried out on the lateritic soil RAP mix at different percentages of Reclaimed Asphalt Pavement. It should also be noted that four (4) samples were compacted for each RAP percentage content, before the average was considered. This makes it 24 samples that undergo CBR test. It was observed that for every percentage increase of RAP, the CBR value also increases accordingly. This reveals that Reclaimed Asphalt Pavement improves the strength characteristics behavior of the lateritic soil RAP mix based on percentage increase.

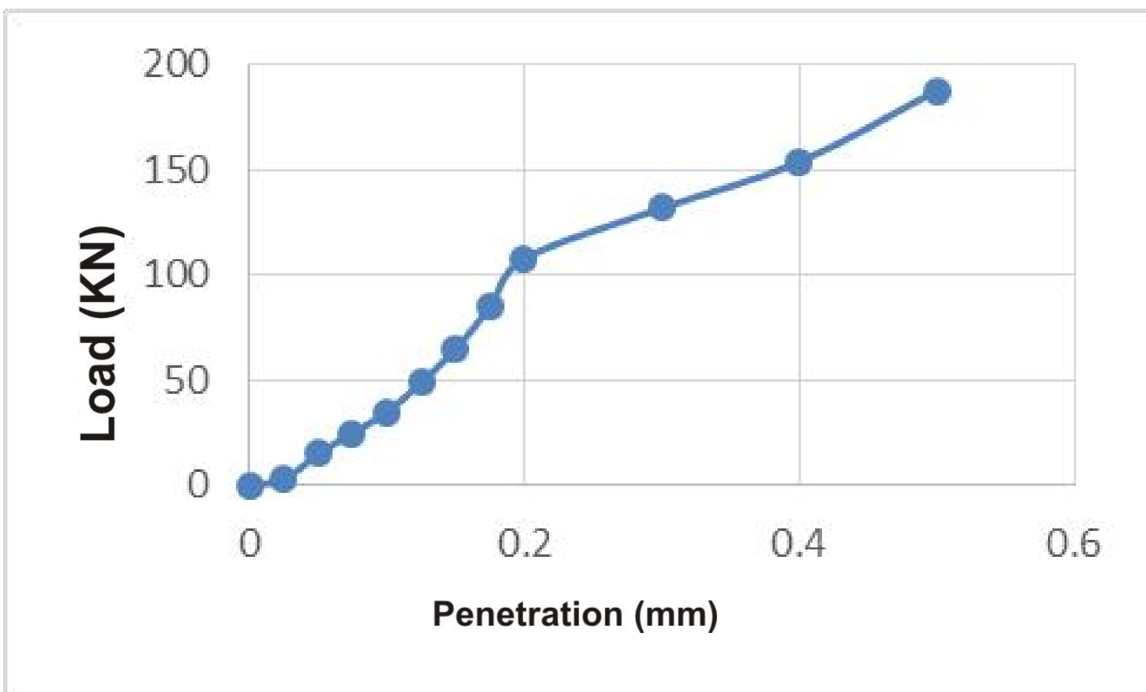


Figure 4.8: CBR effect on lateric soil with 0% RAP

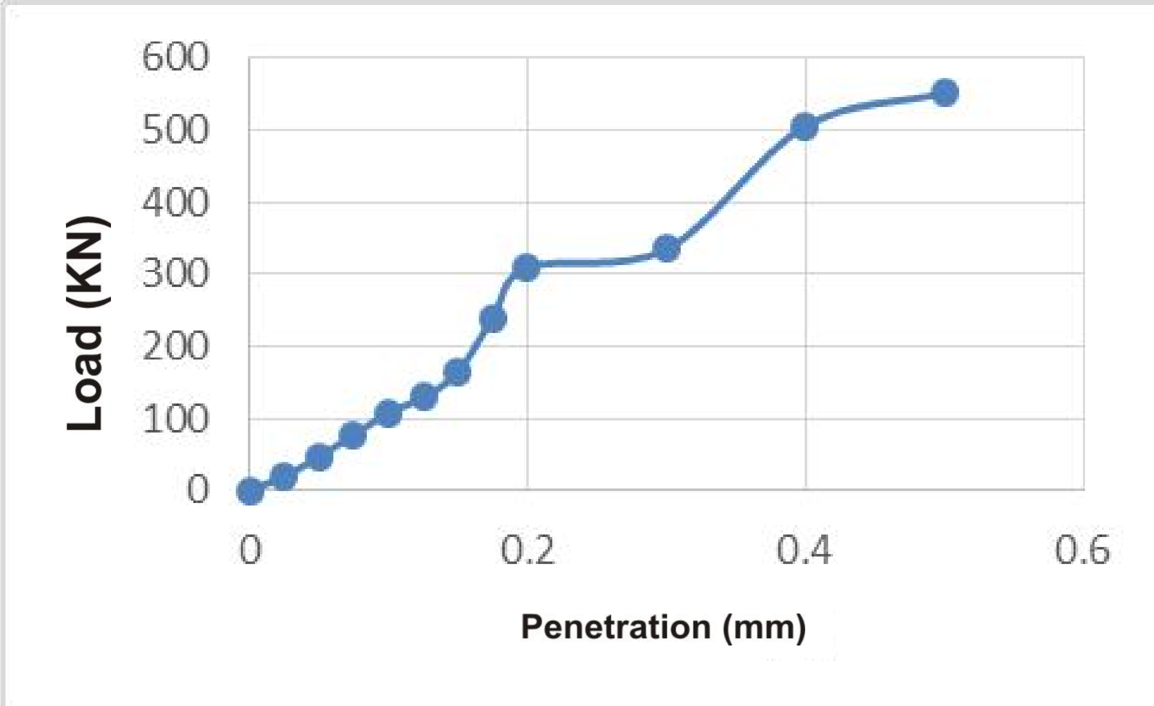


Figure 4.9: CBR effect on lateric soil with 10% RAP

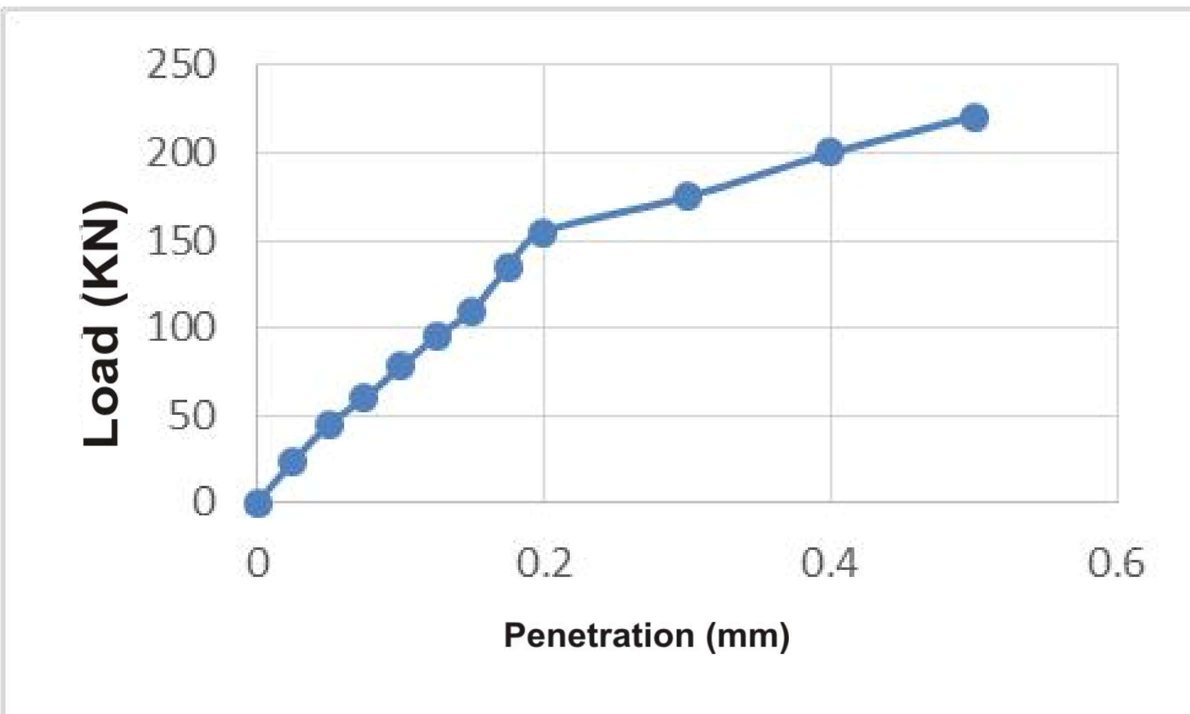


Figure 4.10: CBR effect on lateric soil with 20% RAP

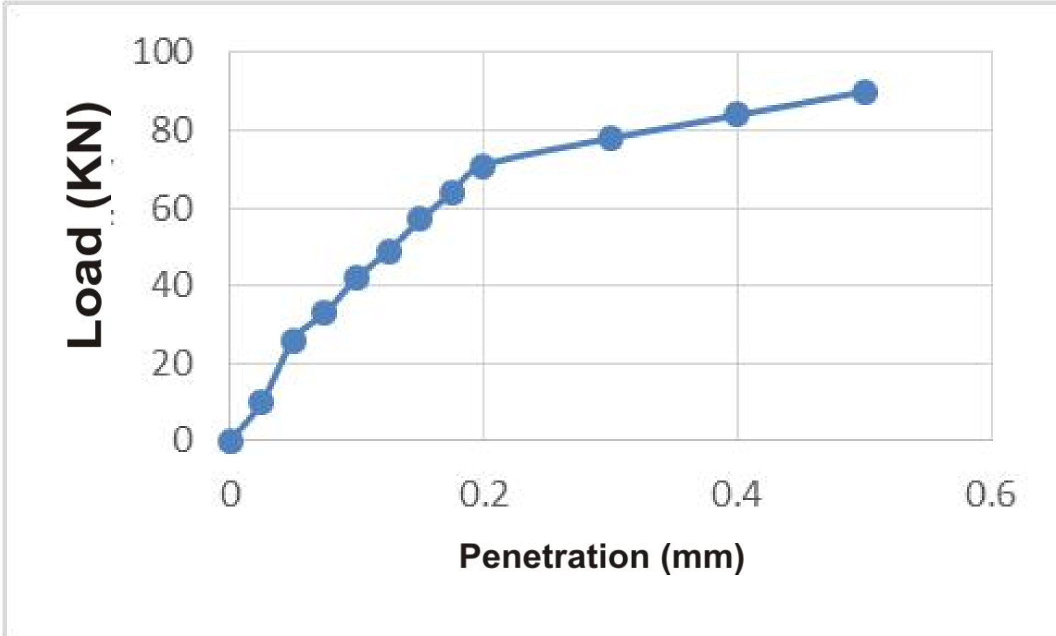


Figure 4.11: CBR effect on lateric soil with 30% RAP

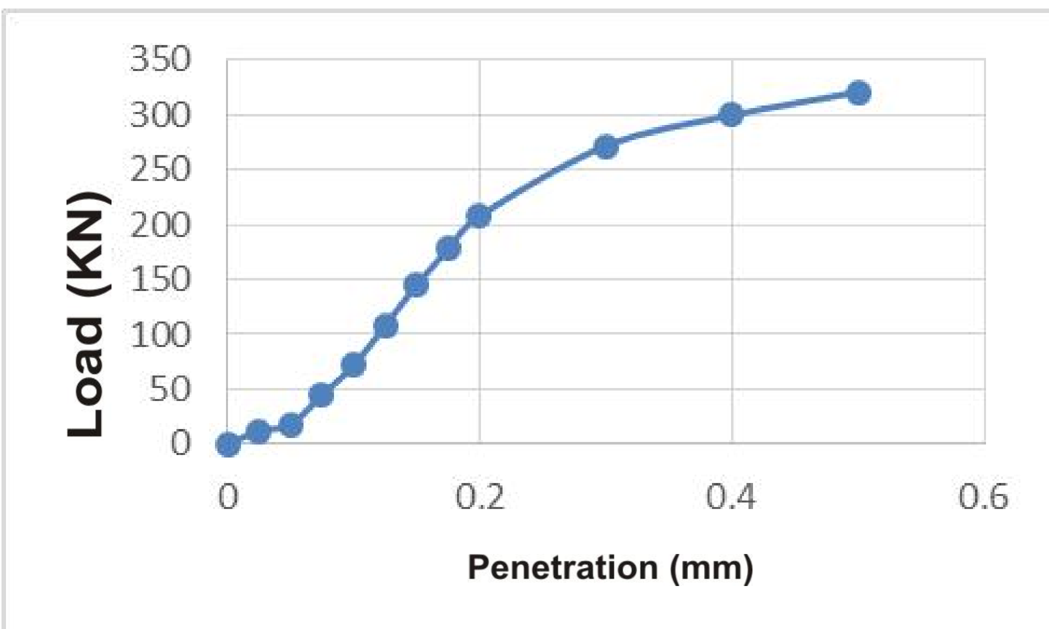


Figure 4.12: CBR effect on lateric soil with 40% RAP

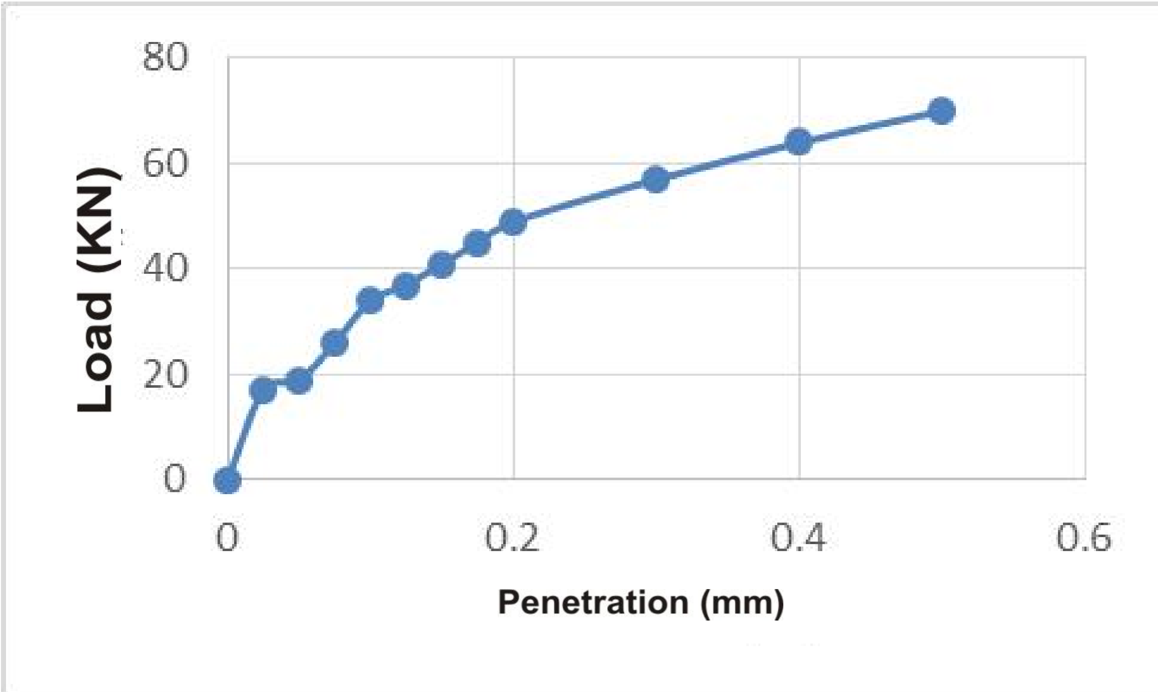


Figure 4.13: CBR effect on lateric soil with 50% RAP

4.4 Interface of Regression Model

The interfaces below reveals each section of the work that was done to bring about the model right from inputs of parameters

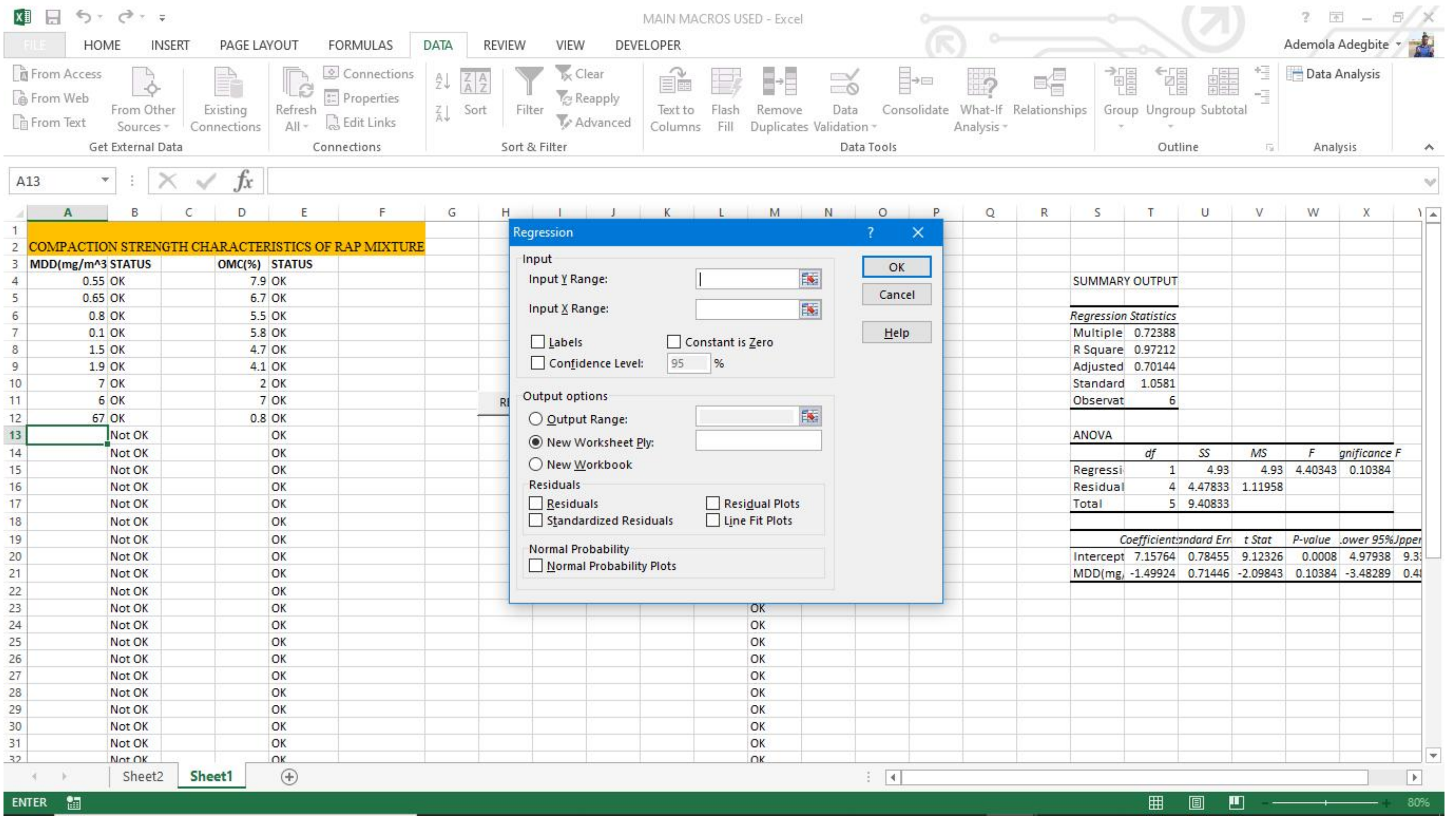


Plate I: Regression Model 1 for Test Sample

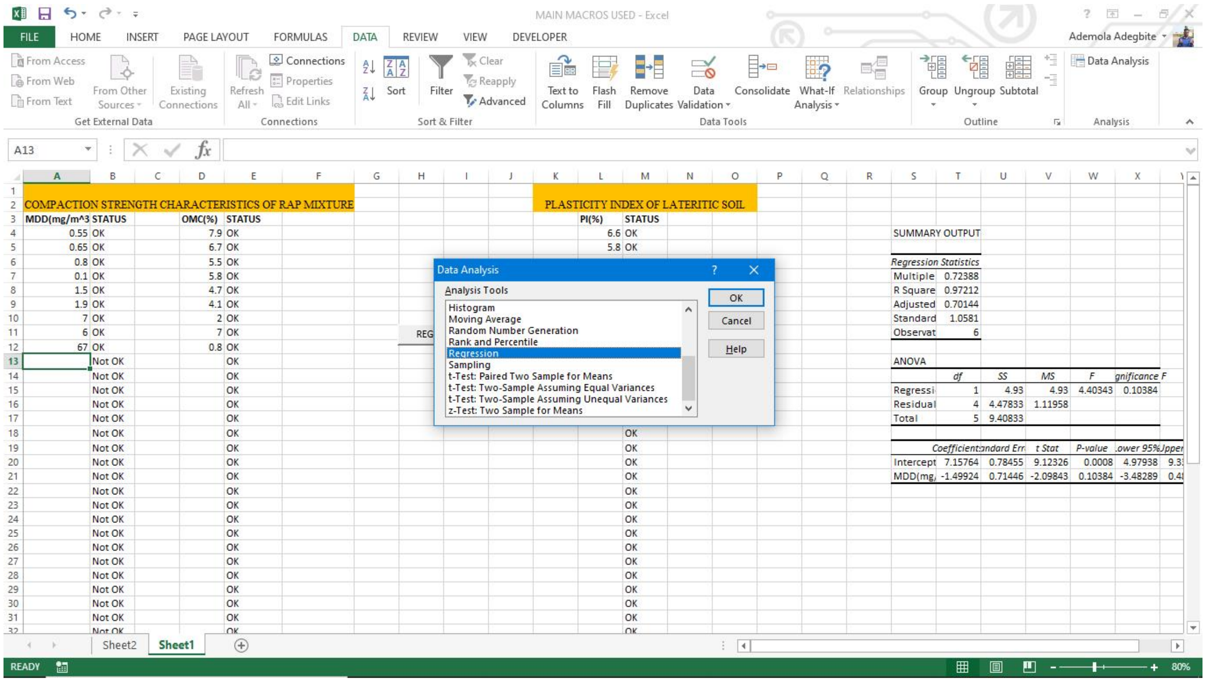


Plate II: Regression Model 2 for Test Sample

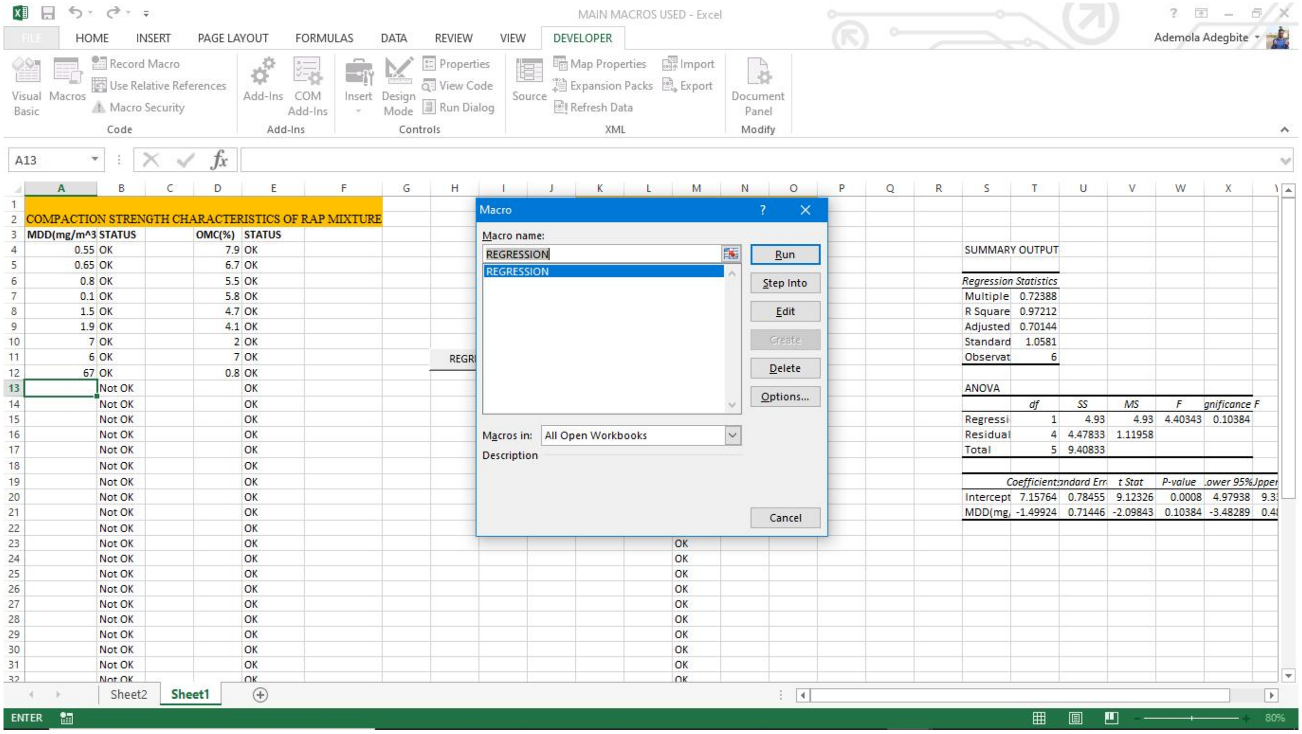


Plate III: Regression Model 3 for Test Sample

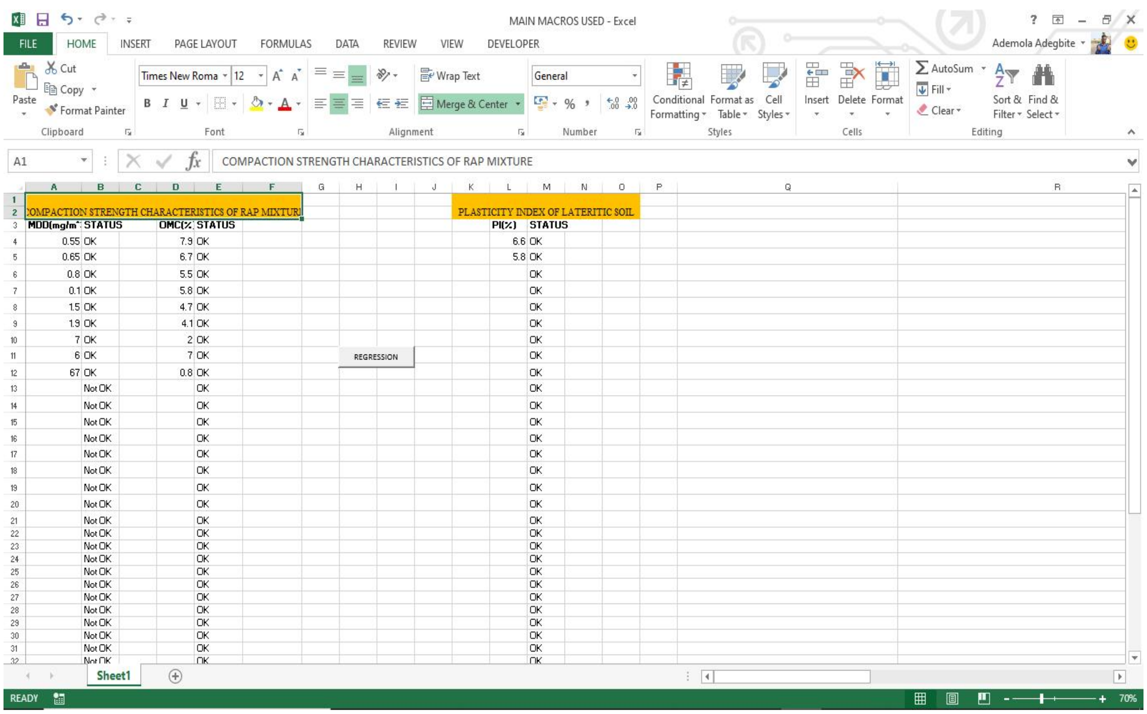
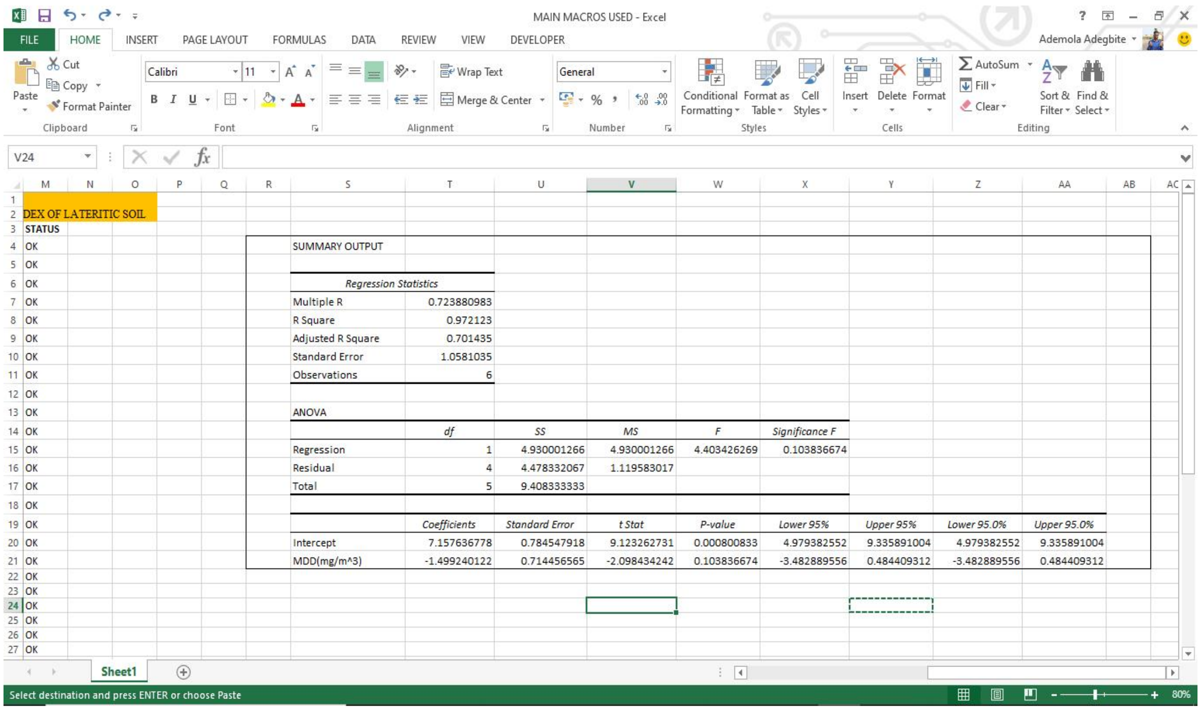


Plate IV: Regression Model 4 for test sample



Regression Statistics	
Multiple R	0.723880983
R Square	0.972123
Adjusted R Square	0.701435
Standard Error	1.0581035
Observations	6

ANOVA

	df	SS	MS	F	Significance F
Regression	1	4.930001266	4.930001266	4.403426269	0.103836674
Residual	4	4.478332067	1.119583017		
Total	5	9.408333333			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	7.157636778	0.784547918	9.123262731	0.000800833	4.979382552	9.335891004	4.979382552	9.335891004
MDD(mg/m^3)	-1.499240122	0.714456565	-2.098434242	0.103836674	-3.482889556	0.484409312	-3.482889556	0.484409312

Plate V: Regression Model 5 for test sample

4.5 Discussion

From the index properties test results, the percentage passing sieve 200 is 42.57% while the liquid limit is 35.5%. This has fulfilled the requirement according to AASHTO classification system of having a minimum of 36% soil particles passing through sieve 200 and liquid limit having 40% maximum. Hence, the lateritic soil was classified as A-6 which indicates that it is a Silt-Clay materials. However, samples with low moisture content, are suitable for road construction and this is expected to greatly increase the shear strength of soil. The liquid limit of the lateritic soil is 35.5% which indicate the probable absence of expandable clay materials; making them suitable for sub-grade which are not greater than 40%. Federal ministry of works and housing (Nigeria) recommended liquid limit of 40% maximum for sub-grade, 35% maximum for sub-base and 30% maximum for base course. Plasticity index of 20% maximum for sub-grade, 16% maximum for sub-base and 13% maximum for base course was also recommended by FMWH. Hence, plasticity index which is 6.6% for our lateritic soil is suitable for sub-grade (FMWH 1997).

However, the result for proctor compaction test are presented in Table 4.2; which indicate that at 0% of RAP, MDD is 0.553g/cm^3 and OMC is 7.90%; at 10% of RAP, MDD is 0.645g/cm^3 and OMC is 6.70%; at 20% of RAP, MDD IS 0.890g/cm^3 and OMC is 5.50%; at 30% of RAP, MDD is 1.120g/cm^3 and OMC is 5.80%; at 40% of RAP, MDD is 1.510g/cm^3 and OMC is 4.70%; at 50% of RAP, MDD is 1.995g/cm^3 and OMC is 4.10%. The Nigerian specification for Roads and Bridges Materials, shows that for a material to be suitable for construction, it should have MDD > 0.047mg/m^3 and OMC < 18%. Hence, for all percentages partially replaced with RAP, the results gotten are suitable for filling and embankment materials base on the results.

Accordingly, the CBR results are presented in Table 4.2; the results shows that at 0% of RAP, CBR is 11.1%; at 1% of RAP, CBR is 13.5%; at 7% of RAP, CBR is 15.5%; at 14% of RAP, CBR is 17.2%; at 28% of RAP, CBR is 18.8%; at 60% of RAP, CBR is 19.3%. Likewise, in accordance to the Nigerian General Specification for Roads and Bridges in Nigeria (1997) which recommends a maximum CBR value of 10-30% for sub-grade, the tested samples both at control and different percentage stages are excellent for sub-grade.

The Empirical model MDD = -0.348x+3.094 has the capacity to accommodate 50 samples with instant results to show if the MDD and OMC of each sample is OK for sub-grade materials. A quick Regression was done and the R square for satisfactory purpose also shows to be 0.97, multiple R is 0.72 and Adjusted R is 0.70. In accordance to the Nigerian Specification for Roads and Bridges Materials which recommends MDD > 0.047mg/m3 and OMC < 18% for a material to be suitable for construction, the model shows that lateritic soil at the various percentages are satisfactory for sub-grade use.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

Analysis of preliminary data obtained from the study indicates that from the Atterberg Limit (Liquid limit, plastic limit and plasticity index), the soil is classified according to the American Association of States Highway and transport officials (AASHTO) as A-6 and CL by the Unified Soil Classification System (USCS). Test result indicates that liquid limit (LL) is 35.5%, plasticity index (PI) is 6.5%. The compaction properties of the natural soil were substantially enhanced by the introduction of Reclaimed Asphalt Pavement (RAP). The test result shows that MDD is in the range of 0.553-1.995kg/cm3 with corresponding OMC which decreased from 7.9 - 4.1% . The California Bearing Ratio (CBR) increased with corresponding RAP percentages from 11.1-19.3%.

The model was developed and can accommodate as much as fifty (50) samples. The empirical model: MDD = -0.348x+3.094 can be used to ascertain the strength characteristics of the lateritic soil-RAP mix, when related to the CBR result which is satisfactory for sub-grade purposes.

5.2 Recommendations

The model can only be used when constraint sets in on the part of the expertise. It should be noted that this can only predict if the lateritic soil RAP mix is OK for construction purpose but cannot give result for the range (weak, normal & excellent) of any Lateritic soil RAP mix. However, at 50% of Reclaimed Asphalt Pavement used on the lateritic soil, the mixcould be considered for the sub-grade of the local light-trafficked roads.

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