Geotechnical Properties and Effects of Palm Kernel Shell Ash and Cement on Residual Soils in Pavement Construction along Owo-Ikare Road, Southwestern Nigeria

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ABSTRACT

Demand for good engineering materials for sustainable roads has necessitated an investigation into the stabilizing potentials of palm kernel shell ash and cement on the geotechnical properties of residual soils as highway construction materials. This study aims at establishing the optimum content of stabilizers required for soil improvement.

Fifteen bulk soil samples were collected from trial pits at depths of 1.5 m in different locations of the study area. Index and engineering strength tests were performed on the soils in their natural states and those stabilized with 2, 4, 6, 8, and 10% of palm kernel shell ash (PKSA) and cement contents. The soils are classified as A-2-7 of the AASHTO classification system and inorganic silt of high plasticity and compressibility (MH-OH) on Unified soil classification system.

The plasticity indices of the soils decreased with increase in PKSA and cement contents with percentage reduction in the interval of 24.94% - 46.26% for PKSA and 48.41% - 56.51% for cement stabilized soils. Stabilization of the soils with PKSA resulted in increase in the optimum moisture content (OMC) with corresponding decrease in Maximum dry density (MDD).

Addition of 6% PKSA optimally improved the CBR of the soils (soaked and unsoaked) and 4% PKSA optimally improved the UCS. Whereas increase in percentage by weight of cement decreased the OMC with increase in MDD. The maximum improvement in the MDD, CBR and UCS including maximum reduction in OMC was observed on the addition of 8% by weight of cement, thus, the optimum content for improvement of the soils. More than 8% by weight of cement caused reduction in these strength indices but increases the OMC. Increase in MDD, CBR and UCS values of all the soils improved the soils richly to the satisfaction of meeting the minimum requirements that guarantee the soils as subgrade and subbase materials in the construction of road pavement.

The CBR - UCS models established are in form of second order polynomial ($ax^2 – bx + c$) with the R^2 values. It was observed that 0.158 and 0.121 correlation was obtained for CBR (unsoaked and soaked) and UCS at PKSA content. While, a stronger positive correlation (0.591 and 0.597) was established for CBR and UCS at cement content. The results indicate good correlation between CBR and UCS. Thus, it can be concluded that between 4 and 6% PKSA and 8% cement serve as optimum amount required for effective stabilization. Therefore, PKSA and cement are recommended for stabilizing poor soil for road works.

(Keywords: Stabilization, residual soil, palm kernel shell ash, cement, road pavement, geotechnical properties)

INTRODUCTION

The depletion of natural resources arising from continuous and constant exploration has called for substitutions and preservation of these resources. Hence, the quest for the use of readily available, cheaper materials and industrial waste materials in infrastructure development is increasing, especially when these materials are tested, and they meet the required specifications and standards. Rapid increase in population and industrialization cause production of large quantities of waste.

Stabilization involves the different methods employed for modifying the properties of a soil to improve its engineering performance. It is a process which basically involves changing the chemical properties of soft soils by adding binders or stabilizers, either in wet or dry conditions, to increase the strength and stiffness of the originally weak soils. Recently, soil stabilization has developed into new ways of using local available environmental and industrial waste materials to improve the engineering properties of weak or clayey soil for their appropriate performance. Soil stabilization and modification in this form emphasize the optimum utilization of these local materials and wastes to achieve low cost construction of engineering projects.

Palm kernel shells can be obtained from the oil palm tree. It is a notable tree with all its parts useful for domestic and industrial purposes. It is commonly found in western part of African and throughout the world. The demand for palm oil is greatly increasing everyday due to its benefits to human body system, thus more cultivation of oil palm is expected in future. Also, the production of palm oil result on wastes like palm kernel shell (PKS), palm kernel fiber (PKF), palm oil mill effluent (POME) and empty fruits bunches (EFB).

Palm kernel shell is a domestic and industrial waste available in large quantities in palm oil producing areas of Nigeria creating storage problem and constituting threat to the environment, since they are generated on a daily basis. Palm kernel shells have very low ash (about 3% weight) (ASTM D3174-02, 2002) and sulphur (about 0.09% weight) (ASTM D4239-02, 2002) contents.

Road failures are often associated with poor construction materials or inadequate design without cognisance of the underlying soils (Ademila, 2018). Normal concrete is produced with mining or river sands and stones, while, lightweight concrete are produced using environmental solid wastes. Recycling materials have been reported to be utilized in different composition in different layers of road structure from the top surfacing layer to the underneath layers (El-Assaly and Ellis, 2001).

Ademila (2017) investigated the effect of rock flour on the geotechnical properties of lateritic soils. The results show significant reduction in plasticity and linear shrinkage of the soil with increasing amount of rock flour. The strength characteristics (maximum dry density, optimum water content, CBR and shear strength) all increased with increasing rock flour content. This improvement in the geotechnical properties of the soils with rock flour shows that rock flour is a good stabilizing agent for weak soil.

Palm kernel shells are used as aggregates in concrete (Olutoge, 2010); Muntohar and Rahman (2011) used oil palm shell as partial replacement of fine aggregates for the production of masonry blocks (shellcrete). Portland cement is the most important hydraulic cement utilized extensively in various types of cement stabilization of soils. Cement acts as a binder and provides the much-desired hardening and strengthening properties.

The addition of cement also increases compressive strength, the resistance of lateritic soils to freezing and thawing, wetting and drying. It also affects the particle size of fine particles (Bello, 2011). However, cement can be applied to stabilize any type of soil, except those with organic content greater than 2% or having pH lower than 5.3 (ACI, 1990).

Road accidents with continuous loss of lives and properties are increasingly becoming a major concern due to bad roads (Ademila, 2017). Performance of highway pavement mostly the flexible one depends on the functions of the component layers especially subgrade and subbase which are compacted layer of soil that provides lateral support to the pavement. Construction over weak/soft subgrade affects the performance of pavement and results in instability of pavement.

Failures on Nigeria highways are generally due to poor geotechnical properties of the underlying soils which constitute the entire road pavement (Ademila, 2018). Due to the variability in the engineering properties of soils, soil testing and information of the sub-soil condition of an area is very important to determine the most suitable method of soil stabilization for a specific soil in the laboratory before the design and construction of highways.

This study presents the effects of palm kernel shell ash and cement on the engineering properties of residual soils for constructing quality and stable highway. This is in a view to reduce the construction cost, prevent the usage of substandard materials, prevent the incessant road failure and more importantly secure the lives of citizens across the country. Furthermore, constant utilization of these non-hazardous solid wastes for construction would eliminate threat and negative impact on the environment for sustainable development.

LOCATION AND GEOLOGY OF THE STUDY AREA

Study Area (Geology and Hydrogeology)

Owo-Ikare Road is strategically located within the North Senatorial district of Ondo State, Nigeria. It lies between latitudes 7° 10' N and 7° 35' N and longitudes 5° 36' E and 5° 50' E as shown in Figure 1. The area is situated in the humid tropical region of Nigeria, characterized by alternating wet and dry seasons with a mean annual rainfall of over 1,500 mm. The area is also characterized with a fairly uniform temperature and high relative humidity. Inhabitants of the towns around the highway are mainly students, indigenes of the towns and government workers. This restricts their occupations mainly to schooling, farming and civil service.

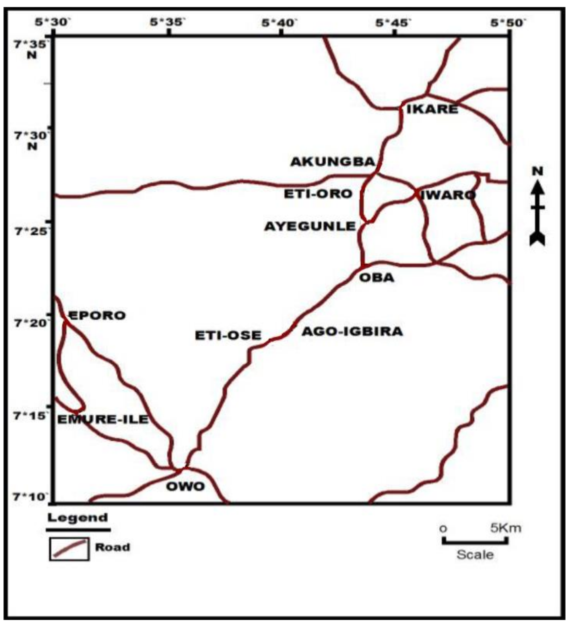

Figure 1: Location Map of the Study Area showing Owo-Ikare Road.

The road covers an area extent of about 65 square kilometers. It is bounded by Ikaram Akoko at the core north, Ido-Ani at the south east, Emure-Ekiti at the south west, and Akure at the core south. The area is drained by the River Ogbese, River Alatan, River Ako, River Isakare, River Awara, River Ose and River Agbo, which are seasonal. The rivers dominate the drainage system of the study area, and it is mainly dendritic. The area is located within western uplands of southwestern Nigeria. The topography varies from flat terrain to low lands and hilly terrain with topography elevation ranging between 345 and 370 meters above the sea level (Oluwafemi and Oladunjoye, 2013).

The study area falls within the Precambrian basement complex rocks of southwestern Nigeria. It is underlain by the migmatite-gneiss-quartzite complex characterized by migmatite gneiss, granite gneiss, grey gneiss and quartzite (Rahaman, 1989). In the study area, migmatite gneiss is the dominant rock unit with a minor amount of quartzite (Figure 2).

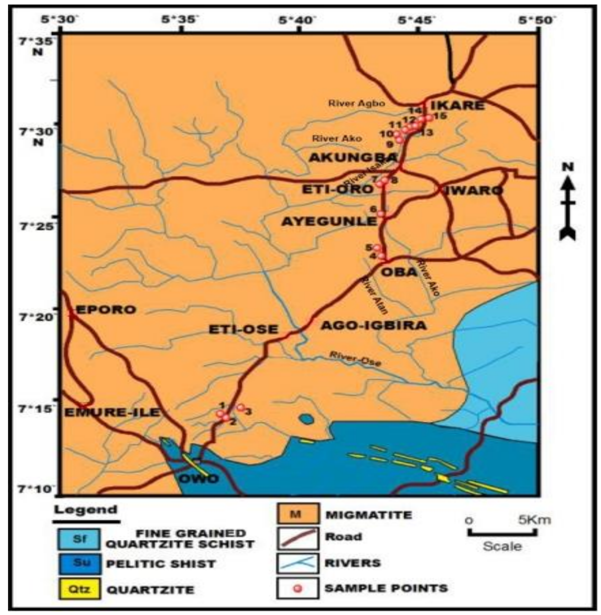

Figure 2: Geological Map of the Study Area.

Pegmatites are also found associated with the migmatite gneiss as an intrusion, which consist of microcline and quartz. Granite gneiss is metamorphosed granite and two major types are distributed in the study area; biotite rich gneiss and banded gneiss. Biotite rich gneiss is fine to medium grained, shows strong foliation trending westwards and is usually dark in colour. Banded gneiss shows parallel alignment and alteration. It occurs mostly as hills, boulders and flat lying exposures, which are dark to light grey in colour and porphyroblastic in texture. Grey gneiss in the study area varies from light to dark grey. Different textural varieties have been recognised, but the most common type is a medium grained rock with regular and persistent banding of varying thickness.

Sources of surface water supply to the study area are the River Ogbese, River Alatan, River Ako, River Isakare, River Awara, River Ose and River Agbo and their major tributaries. In the basement complex area, groundwater is contained within the weathered and/or fractured/jointed basement columns. A static water level of the hand-dug wells in the area ranges from 2.14 m to 6.35 m.

The unconfined nature and the near-surface occurrence of the aquifer system in the area expose the subgrade soil to ingress of water. Groundwater moves from a region of high concentration and altitude to a region of low concentration and low altitude. It implies that groundwater in the study area flows in the N–S and NE–SW directions.

MATERIALS AND METHODS

Materials

Soil: This study was conducted on fifteen bulk soil samples collected from fifteen different locations from the residual soil deposit which stretched along the Owo-Ikare Road. The samples were obtained at sampling depths of 1.5 m in order to obtain true representative samples of the subgrade which is the placement level of flexible highway pavement. On the field, the rock and soil exposures were observed and described. The sample collection was done systematically to ensure proper collection of samples and total coverage of the study area. A global positioning system (GPS) was used at each sampling point to measure coordinates of the station and heights above sea level. All the soil samples were carefully labeled in sample bags and then taken to the laboratory in sealed polythene bags to prevent contamination and loss of moisture.

Palm Kernel Shell Ash (PKSA): Bulk samples of palm kernel shell were obtained from local oil producing firm in Akungba Akoko, Ondo State, Nigeria. The shells were thoroughly washed with water to remove impurities. They were sun dried and kept in waterproof sacks. The washed quantities of palm kernel shells were burnt properly to ashes in a blast furnace to about 900°C - 1000°C in the laboratory. The ash passing through a No. 200 sieve mesh, with 0.075 mm aperture BS 1377 (1990) and BS 1924 (1990) was used for this study.

Cement: Ordinary Portland cements were used. Portland cement is the most common type of cement in general use in the country, it was used as stabilizing agent in this study and conformed to the requirements of BS 12 (1996).

Water: The water used for this study was obtained from a borehole. The water was clean and free from any visible impurities. It conformed to the requirements of BS 3148 (1980).

Test Methods

The natural moisture content was determined immediately in the laboratory. Soil samples were air dried for two weeks to allow partial removal of natural water before other analysis. After the drying, lumps in the samples were gently grounded with minimal pressure as not to reduce the sizes of the individual particles. The following laboratory tests were conducted on the samples: natural moisture content, specific gravity, consistency limits, linear shrinkage, grain size distribution, compaction, California bearing ratio (CBR) and unconfined compressive strength. Samples for grain size analysis were soaked in a weak Calgon solution to facilitate disaggregation during wet sieving.

However, for the soil stabilization, five representative bulk soil samples were selected from the fifteen samples, ensuring total coverage of the study area and the effects of palm kernel shell ash and cement as stabilizing agents on the geotechnical properties of the samples were determined. These laboratory analyses were carried out according to British Standard Methods of test for soils for civil engineering purposes (BS 1377:1990) and ASTM Standard D1557 (2009).

The compaction test was conducted on soil samples that were compacted in three layers in a CBR mold each 25 mm thick and applied 56 numbers of blows of 4.5 kg rammer falling freely through a height of 450 mm. The CBR test was carried out with a mold of capacity 945×10^{-6} m^3 at optimum moisture content and 96 hours of soaking period which simulated the prolonged inundation and submergence encountered during the peak of rainy season between July and October.

The procedure of unconfined compressive strength test was defined as cylindrical specimen of cohesive soil where a steadily increasing axial load was subjected to the soil specimen until failure (BS 1377:1990). The specimen was placed centrally on the lower platen of the compression testing machine. The force was applied with a controlled strain rate of approximately 1 mm/minute. The force was recorded during the test until the specimen failed.

Consolidation Test

One dimensional consolidation test was carried out in accordance with British Standard (BS 1377:1990) and as described by Head, 1994. The compacted soil samples were cored into the oedometer ring and placed in the consolidometer setup. Pressure increment was 100 kN/m^2, 200 kN/m^2, 400 kN/m^2 and 800 kN/m^2 during the loading stage and unloaded up to 200 kN/m^2. Compression readings were recorded between 10 seconds and 24 hours during the loading stage for each incremental load.

Shear Strength Test

Horizontal load was applied as soon as vertical load has been imposed in this test and shearing continued at the rate of approximately 1 mm/minute, until the shear force goes beyond its maximum value and becomes constant or decreases, representing failure condition. The results of the direct shear tests for the soils are presented in the form of stress-strain curves and plots of shear stress versus normal stress. From these, the shear strength parameters (angle of cohesion (c) and angle of internal friction (ϕ) were obtained.

Mathematically, the shear strength of soil is governed by the Mohr-Coulomb failure criterion:

$$S = C + P \tan\phi$$

Where S is the shear stress at failure along any plane and it is the normal stress on that plane. C and ϕ are the shear strength parameters; cohesion and angle of shearing resistance.

Soil-PKSA Mixtures

PKSA was added to the soil in 2, 4, 6, 8, and 10% proportions by dry weight of the soils. Index properties and strength tests were performed on soil-PKSA mixtures. The influences of PKSA as stabilizing agent on the samples were determined. The various tests were carried out in accordance with BS 1377 (1990) and BS 1924 (1990) procedures.

Soil-Cement Mixtures

Soil-cement mixtures were prepared by mixing the desired proportions of potable water, soil and cement. Percentages of cement ranged from 0 to 10% by weight of dry soil. Soil-cement mixtures were first prepared by mixing thoroughly dry weight quantities of crushed soil and cement in a mixing tray to form a uniform paste. Required amount of water determined from moisture-density relationships for soil-cement mixtures was later added to the paste (dry soil-cement mixture) and left for about 3 hrs. before compaction.

RESULTS AND DISCUSSION

The results of the index properties (natural moisture content, specific gravity, Atterberg limits and particle size analysis) and the engineering properties tests (compaction, California bearing ratio (CBR), unconfined compressive strength (UCS) and shear strength) are summarized in Tables 1 and 2.

Natural Moisture Content (NMC)

The natural moisture contents of soil samples L1 – L15 vary from 29.7 – 37.9% (Table 1). The moisture content indicates a high water adsorption capability of the soil material as related to the road failure observed along the area. This influences the shrink-swell potential of soils. The variation in the natural moisture content of the soils may be as a result of the effects of topography, climatic and hydrological conditions of the area where the road is constructed. This in-turn dictates the behavior and performance of engineering properties of the soil of an area. Due to the fact that moisture content is used as an indicator of groundwater level of an area, the high moisture content obtained conforms to the high water influx from the unconfined nature and the near-surface occurrence of the aquifer system in the area.

Specific Gravity (Gs)

The specific gravity of the soil samples ranges from 2.36 – 2.92. It depends on the mineralogical composition of the constituent soil particles. Most clay minerals have specific gravities that fall within 1.6 – 2.9. The results showed that some of the soils contain a proportion of quartz especially soil samples L11 and L12, while others contain sodium and calcium feldspar except L3 and L4soil samples that are montmorillonitic in nature (Das, 2000). The presence of montmorillonite in the soil is detrimental to the stability of engineering structures on the soil.

Table 1: Index Properties of Soils of the Study Area.

Sample Code	NMC (%)	Gs	Consistency Limits				Grain Size Distribution Parameters				
			LL (%)	PL (%)	PI (%)	LS (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Fines (%)
L1	34.6	2.64	62.4	24.7	37.70	5.8	3.4	61.3	18.2	17.1	35.3
L2	37.7	2.66	62.9	24.9	38.00	7.2	2.9	61.5	18.6	17.0	35.6
L3	37.6	2.92	63.0	24.7	38.31	7.7	3.7	67.1	14.5	14.7	29.2
L4	37.4	2.87	62.1	24.6	37.4	6.3	2.8	64.7	16.7	15.8	32.5
L5	37.1	2.60	62.5	24.1	38.40	5.8	3.6	66.2	18.8	11.4	30.2
L6	36.9	2.66	60.0	25.3	34.70	8.2	2.7	66.9	18.4	12.0	30.4
L7	37.9	2.62	61.9	24.6	37.30	7.2	3.6	61.6	20.6	14.2	34.8
L8	29.7	2.61	54.0	20.2	33.80	8.1	3.5	63.8	14.9	17.8	32.7
L9	36.7	2.64	60.9	23.9	38.25	7.7	3.0	65.1	14.1	17.8	31.9
L10	37.0	2.66	63.9	24.0	39.90	6.8	2.8	69.6	11.7	15.9	27.6
L11	36.6	2.42	60.4	23.8	36.60	8.2	3.5	62.5	15.5	18.5	34.0
L12	36.9	2.36	63.5	23.8	39.70	7.2	2.9	63.4	15.8	17.9	33.7
L13	37.0	2.66	63.5	23.8	38.30	7.7	2.8	66.7	17.6	12.9	30.5
L14	37.0	2.61	57.7	24.7	33.00	7.7	3.0	67.7	12.5	16.8	29.3
L15	37.1	2.62	57.5	24.8	32.70	8.2	2.8	66.3	16.5	14.4	30.9

Consistency Limits

The consistency is indirectly capable of revealing the clay mineralogy and shrink/swell potential of a soil (Carter and Bentley, 1991). This is very important in highway engineering for the classification of soil. The results of the Atterberg limits (liquid limit (LL), plastic limits (PL) and the plasticity index (PI) for the natural soil range between 54.0 – 63.9%, 20.2 – 25.3% and 32.7 – 39.9%, respectively (Table 1). Liquid limit between 50% and 70% indicates high plasticity (Whitlow, 1995). This shows that the soils have high plasticity. The soils in their natural state have high liquid limit (> 40%), low plastic limit (< 30%) and high plasticity index (> 20%) indicating high swelling and high compressibility.

The high plasticity suggests the presence of high proportions of clay minerals. This portrays the soils as being susceptible to volume change, capable of swelling in the presence of high moisture content and shrink when dry. This alternating swelling and shrinkage would be responsible for the failures observed along the road of the study area. Plasticity index tends to have influence on the activity of the subgrade. It is higher than 12%, thus unsuitable for use as subgrade and subbase materials for roads and bridges as specified by Federal Ministry of Works and Housing, (FMWH) 2000. The difference in the plasticity index, amount of swelling and compressibility may be due to the presence of montmorillonite clay minerals in the mineralogy of the soils. Hence, modifications of the natural properties of such soils may be necessary for construction purposes.

The linear shrinkage (LS) of the soils ranges from 5.8 – 8.2%. All the soils are ≤ 8% recommended by Madedor (1983) for highway subgrade soil. Based on this consideration, the soils are good subgrade materials and they are expected not to create field compaction problem.

Grain Size Distribution Analysis

The grain size analysis of the soils shows that almost all the soils have percentage passing No. 200 (0.075 mm) of less than 35% (Table 1). They classify in group A-2-7 of the AASHTO classification system, signifying good highway subgrade materials being silty or clayey gravel and sand except the soil at Location L2 in the group A-7-6 indicating poor highway subgrade material being clayey. The Casagrande (1948) chart (Figure 3) classified the soils as inorganic silt of high plasticity and compressibility. The plot implies that the soils are high in plasticity and swelling potential. Based on the Unified soil classification system, the soils fall in the category of MH-OH (inorganic silt/organic clays of medium to high plasticity). Due to this classification, the soils are susceptible to cyclically shrinking and swelling, causing damage to the road as observed and any engineering structure constructed on it. Thus, there is need to improve the geotechnical properties of the soils to be able to withstand/support the high traffic load imposed on it.

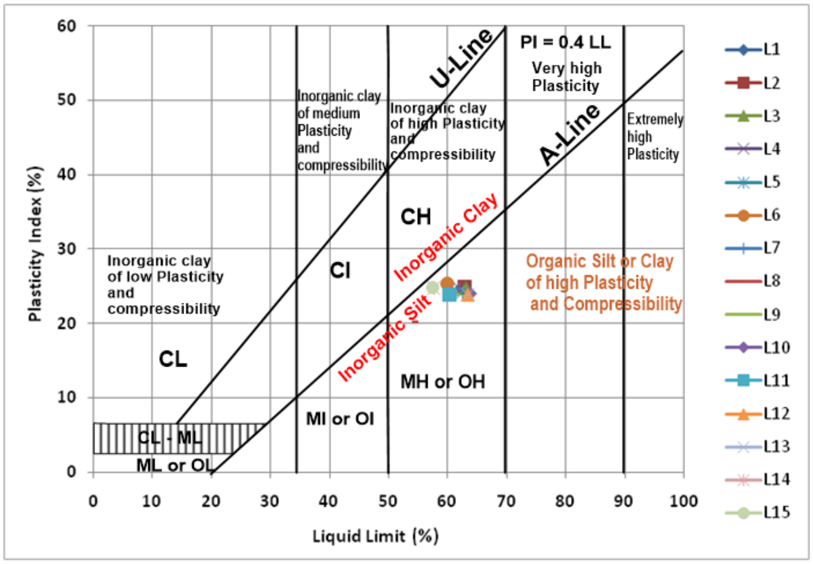

Figure 3: Plasticity Chart for Classification of the Soils (after Casagrande, 1948).

Engineering Property Tests

The basic strength characterizations used in this study are summarized in Table.2. They are Moisture-Density relationship, California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS), shear strength and settlement characteristics.

Strength Characteristics

The resistance of a soil to deformation under load is usually a measure of its strength (Das, 2002; McCarthy, 2007). Subgrade is generally made up of locally available natural soils. The strength and performance of a pavement is dependent on the load bearing capacity of the subgrade soil. Compaction reduces the detrimental effects of water. Lower optimum moisture content is expected to achieve maximum dry density for stability of soil under field conditions. Compacting soils for roads and airfields require attaining a high degree of density during construction to prevent detrimental consolidation from occurring under an embankment's weight or under traffic.

The optimum moisture content (OMC) and the maximum dry density (MDD) of the soils at unstabilized state are presented in Table 2. The OMC ranges from 10.1 – 15.3% while, the MDD ranges from 1845 – 2068 Kg/m^3. The moisture content required to keep the soil suitable as subgrade/fill materials may not be feasible under certain conditions on the field. Low values of MDD suggest that the natural soil is loose as exhibited by its high void ratio and porosity. The greater the soil density, the greater its structural strength and the less its water absorption tendency and improved resistance to settlement. With the compaction characteristics (high MDD with low OMC), the soils of the study area will yield maximum strength if compacted on the dry side of their optimum moisture content on the field. This high density would make the soils not susceptible to dispersal by high water influx and/or erosion.

The unsoaked California Bearing Ratio (CBR) values range from 17 – 28%. The unsoaked CBR value stipulated by the FMWH standards (2000) for subgrade soils is 80% minimum. None of the soil analyzed is within this specification. Also, the soaked CBR (9 – 17%) values represent a poorly drained, flooded environmental situation. Both the unsoaked CBR and soaked CBR fall below the minimum 80% for base course and 30% for subbase course respectively (Table 2). Hence, the soils are unsuitable for road subbase and base courses. The reduction in the soaked CBR compared to the unsoaked CBR is an indication of a significant loss in strength in the presence of water or on increase of water content. The soaked CBR values showed that the soils as subgrade would undergo considerable amount of volume change in the presence of water.

Table 2: Summary of the strength characteristics of soils of the study area

Sample Code	OMC (%)	MDD (kg/m^3)	Unsoaked CBR (%)	Soaked CBR (%)	UCS (kN/m^2)	Cohesion, c (kN/m^2)	Angle of internal friction, ϕ (°)
L1	12.2	1984	19	9	51.6	57.89	15
L2	12.5	1973	27	12	46.6	45.32	13
L3	14.8	1916	20	9	77.4	58.56	12
L4	15.3	1898	19	11	78.4	63.11	16
L5	12.4	1976	25	10	39.4	48.35	11
L6	13.8	1898	17	12	36.5	58.94	14
L7	14.2	1882	26	14	39.4	68.51	16
L8	15.1	1845	28	17	39.2	56.33	13
L9	10.1	2066	20	14	30.7	64.93	15
L10	13.7	1930	28	15	32.1	59.01	12
L11	12.0	2016	28	16	39.2	70.42	14
L12	10.6	2068	20	15	39.1	66.72	15
L13	11.9	1984	23	13	41.9	46.94	12
L14	13.7	1916	19	12	42.1	56.37	11
L15	14.5	1869	18	14	48.2	63.24	13

Consistency of a soil can be expressed in terms of unconfined compressive strength (UCS) of soils which in turn determines the strength of the soil. The UCS value measures the suitability of soil as subgrade/foundation material. It ranges from 31 – 78 kN/m^2 (Table 2). Based on Das, 2000 recommendations, the soils indicate soft to medium stiffness. Soft stiffness soils serve as poor subgrade materials, this is an indication that the soils would be too soft to support traffic load, thus, the failure characteristics witnessed along the highway. Soil stiffness quality of the soils need to be upgraded through stabilization.

Table 2 shows the summary of shear strength test results. Low cohesion (45 – 70 kN/m^2) and frictional angles (11 – 16°) of the soils have a good correlation with its index properties. The low strength will cause failure of pavement under sustained axial loading. The low frictional angle is caused by the presence of swelling clay. Low bearing capacity and low strength characteristics of the soils show their unsuitability for use as either foundation materials for heavy structures or subgrade materials. All the soils of the study area possess poor shear strengths as shown in the values of their cohesion (c) and angle of internal friction (ϕ). Clay fraction content has an influence on the cohesion of residual soils compacted at various moisture contents (Gidigasu, 1976).

All the soils of the study area have low clay content (between 11 and 19%) which is the reason for their low cohesion values. Relative to these low c and ϕ values, the soil possesses water attractive swelling clay which is responsible for the low bearing capacity of the soils. This implies that the soils will have poor slope stability and will therefore be unsuitable as well in the construction of embankments. Low strength characteristics of soils require soil improvement measures for stable structures (Ademila, 2018).

The ranges of values of coefficient of volume compressibility, Mv (0.015 – 0.343 m^2/MN) and coefficient of consolidation, Cv (0.012 – 0.030 $m^2/year$) of the soils obtained from the consolidation tests give an indication of the large volume changes that are associated with the soils. This shows significant lateral variation indicating likelihood of differential settlement for structures founded on the soils.

Influence of Palm Kernel Shell Ash (PKSA) and Cement as Stabilizing Agents on the Soils for Road Construction

Plasticity Characteristics: The addition of PKSA and cement in 2, 4, 6, 8, and 10% to the soil samples caused a change in the liquid limits and plasticity indices of the soil samples. The liquid limit of the soils decreased with increment in PKSA content from 0% to 8% then experience sharp increase for 10% PKSA and plasticity index decreased with increment in PKSA content from 0% to 6%, then experience sharp increase from 8-10%. The liquid limit decreased to the minimum of 50.1, 48.7, 45.9, 41.7 and 50.3% in samples L2, L4, L6, L8, and L12 respectively (Figure 4). The plasticity indices also decreased from 38, 37.40, 34.70, 33.80, and 39.70% in their unstabilized state to 20.7, 20.1, 21.4, 21.8 and 29.8% in samples L2, L4, L6, L8 and L12 respectively (Figure 4).

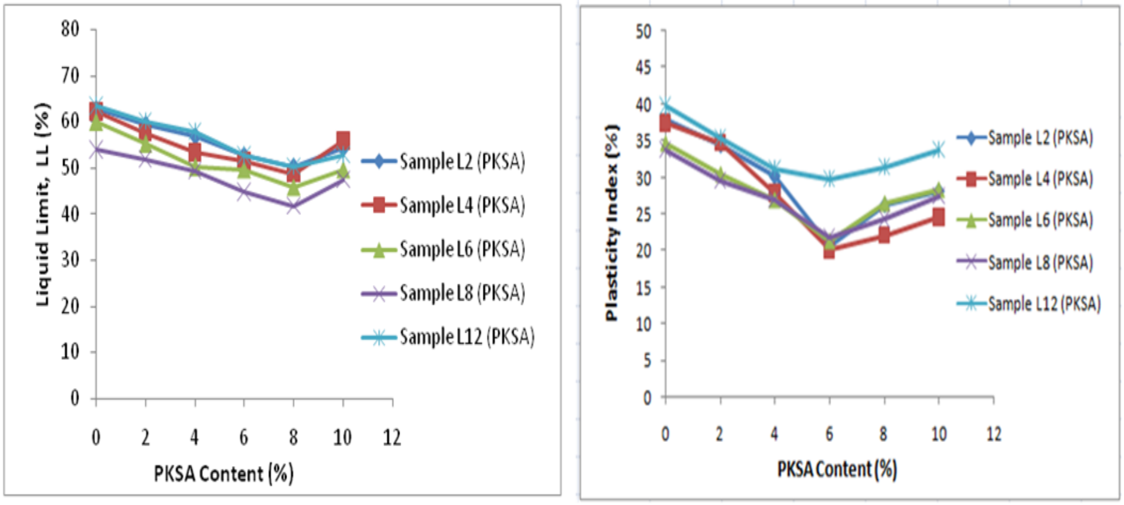

Figure 4: Variation of Liquid Limit and Plasticity Index with PKSA Content for the Five Soil Samples.

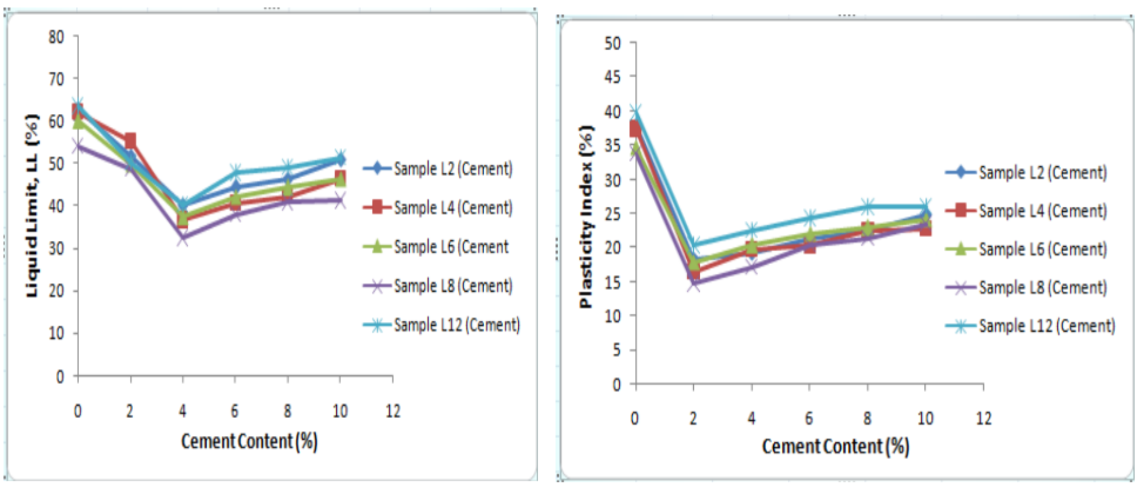

Figure 5: Variation of Liquid Limit and Plasticity Index with Cement Content for the Five Soil Samples.

The liquid limits and plasticity indices of the soils dropped to its optimal value on addition of 4% and 2% by weight of cement respectively (Figure 5). The liquid limit decreased to the minimum of 40.2, 36.7, 37.4, 32.4, and 40.2%, while the plasticity indices also decreased to 20.7, 20.1, 21.4, 21.8, and 29.8% in samples L2, L4, L6, L8, and L12, respectively. The liquid limit of the soils was reduced to its optimal value on addition of 8% (PKSA) and 4% (cement) by weight of the stabilizers. The effects of addition of different percentages of PKSA and cement on the liquid limits of the soils are shown in Figures 4 and 5.

The decrease in liquid limit and plasticity index may be attributed to aggregation and the cementation of particles into larger size clusters to produce denser soil with PKSA and cement mixes. The liquid limit decreases at all soil-PKSA and soil-cement mixes the stabilized soils are therefore suitable as subgrade and subbase materials for road pavement construction.

The reduction in the plasticity index would reduce the potential of the soil to shrink under moisture change. The influence of PKSA and cement on the plasticity indices of the samples are shown in Figures 4 and 5. The plasticity index generally decreased with increase in PKSA and cement contents, which makes stabilized soils much better for use as construction materials. The minimum plasticity index achieved at 6% and 2%

in all the samples for the stabilizers with percentage reduction in the interval of range 24.94% - 46.26% for PKSA and 48.41% - 56.51% for cement stabilized samples. These reductions in plasticity indices are indications of soil improvement with response to a more stable soil with enhanced workability. This reduction in plasticity indices may be attributed to the replacement of the soil fines by PKSA and cement. These values indicate strong influence of PKSA and cement as stabilizers for improving plasticity of migmatite-gneiss derived soils.

Compaction Characteristics

Compacting soils for roads and airfields requires attaining a high degree of density during construction to prevent detrimental consolidation from occurring under an embankment's weight or under traffic. In addition, compaction reduces the detrimental effects of water. The influence of addition of varying quantities of PKSA and cement by weight of soil samples on the compaction parameters of the soils are presented in Figures 6 - 9.

Optimum Moisture Content (OMC)

The effects of the stabilization of the soils with varying percentage by weight of PKSA and cement on the optimum moisture content (OMC) of the compacted soils are displayed in Figures 6 and 7. The stabilization of the soils with PKSA resulted in progressive increase in OMC by as much as 78% - 142% in all the samples from the initial unstabilized soils of 12.5% - 26.4% (L2), 15.3 – 27.2% (L4), 13.8 – 31.2% (L6), 15.1 – 27.4% (L8), and 10.6 – 25.7% stabilized soils with 10% by weight of PKSA (Figure 6).

The progressive increase in the OMC with increase in percentage by weight of PKSA may be due to the increased surface area of particles caused by increased PKSA content in the mix that required more water to lubricate the whole mix to enhance compaction, in addition to the water taken up by PKSA hydration reaction. This increase in OMC with increase in percentage by weight of PKSA may also be attributed to the reaction of K_2O and CaO content of the ash with the naturally occurring CaO in the residual soil initiating cation exchange, flocculation and agglomeration of the soils.

On the contrary, increase in percentage by weight of cement used in stabilization of the soils resulted in decrease in OMC (Figure 7) from the initial values of the unstabilized soils to the minimum of 5.4% (L2), 8.3% (L4), 5.9% (L6), 6.4% (L8), and 4.7% (L12) upon the addition of 8% by weight of cement with percentage reduction in the interval of 45.75 – 57.62%. As observed with OMC, increase in the percentage by weight of cement to 10% resulted in decrease in percentage reduction in the interval of 22.22 – 36.8% (Figure 7). This implies that, it is not necessary to stabilize the soils with more than 8% by weight of cement. The OMC generally reduced correspondingly in all the soil samples with the addition of cement.

Maximum Dry Density (MDD)

The effects of addition of varying quantities of PKSA and cement on MDD of the soils of the study area are illustrated in Figures 8 and 9. Increase in percentage by weight of PKSA content used in stabilization of the soils resulted in reduction of the MDD of the soils. The addition of 2, 4, 6, 8, and 10% PKSA as stabilizer by weight of samples caused reduction in maximum dry density from 1973 Kg/m^3 (L2), 1898 Kg/m^3 (L4 and L6), 1845 Kg/m^3 (L8), and 2068 Kg/m^3 (L12) unstabilized samples to the minimum of 1847 Kg/m^3 (L2), 1826 Kg/m^3 (L4), 1740 Kg/m^3 (L6), 1729 Kg/m^3 (L8), and 1933 (L12) stabilized samples (Figure 8). This is representing 3.79 – 8.32% reduction in density.

Das (2000) opined that a reduction in dry density might occur due to both the particle size, specific gravity of the soil and stabilizer. Also, decreasing of dry density indicates that there is need for low compactive energy than the natural soil to attain its maximum dry density and the cost of compaction will be economical as a result of this (Muntohar, 2000). The benefit of the increase in OMC with increasing quantities of PKSA content and corresponding decrease in MDD of the soils is that, it allows compaction to be easily achieved with wet soil. Therefore, the need for the soils to be dried to lower moisture content before compaction in the field becomes minimal.

On the other hand, the addition of between 2% and 8% by weight of cement led to progressive increase in the MDD. The MDD increased from the initial values (unstabilized samples) to the maximum of 2103 Kg/m^3 (L2), 2294 Kg/m^3 (L4),

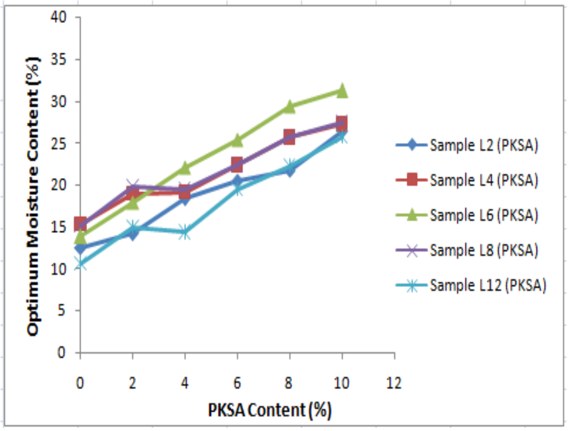

Figure 6: Influence of PKSA Content on Optimum Moisture Content (OMC).

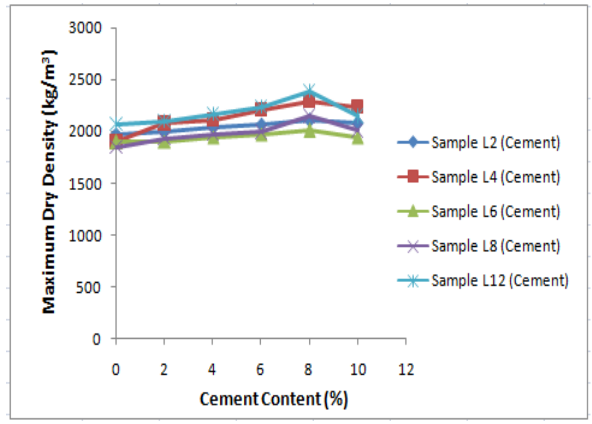

Figure 9: Influence of Cement Content on Maximum Dry Density (MMD).

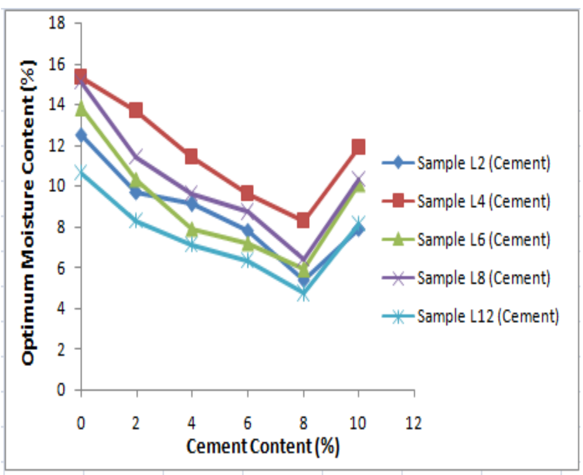

Figure 7: Influence of Cement Content on Optimum Moisture Content (OMC).

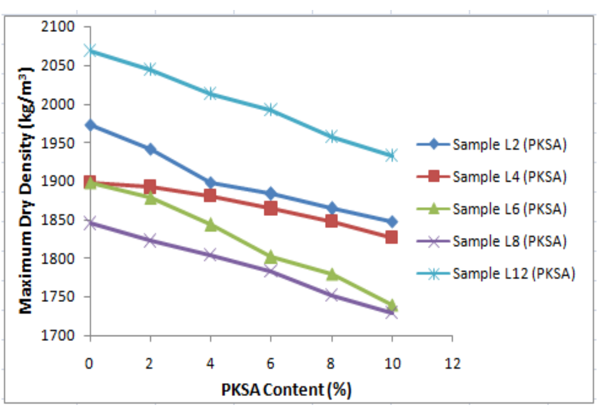

Figure 8: Influence of PKSA Content on Maximum Dry Density (MMD).

2005 Kg/m^3 (L6), 2147 Kg/m^3 (L8), and 2392 Kg/m^3 (L12) on addition of 8% by weight of cement (Figure 9), representing a percentage increase in the interval of 5.64 – 20.86% in all the samples.

Further increase in the percentage of cement more than 8% (10% by weight of cement) resulted in reduction of percentage increase in MDD to an interval of 2.42 – 17.91% in the samples. This shows that increase in percentage by weight of cement content has positive influence on the soil density, and also improve the soils strength. The reduction in the trend of increment in MDD when 10% by weight of cement was employed for stabilization of the soils (Figure 9) suggests 8% by weight of cement as the optimum percentage needed to improve the soils' MDD.

The decrease in OMC may be due to the absorption capacity of cement. The lower the OMC, the better the workability of good soils. The addition of cement increased the dry densities of all the samples, which majorly indicate improvements in the soil properties. An increase in MDD of the soils is a good indication of improvement in soil property, while a reduction in OMC enhances the workability of good soil.

California Bearing Ratio (CBR)

California bearing ratio test is a semi-empirical test for evaluating soil strength (subgrade, subbase and base course materials) for road and

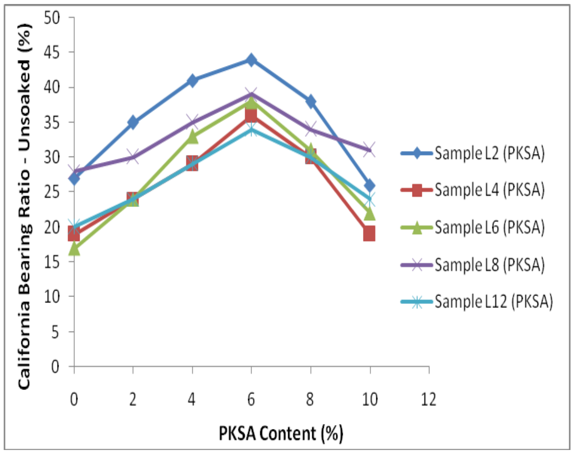

Figure 10: Influence of PKSA Content on California Bearing Ratio (Unsoaked).

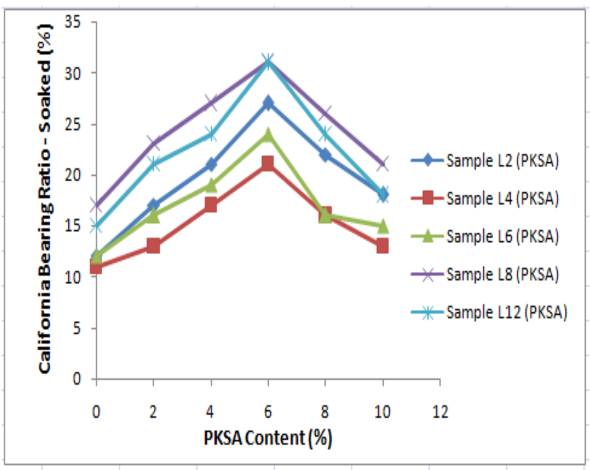

Figure 11: Influence of PKSA Content on California Bearing Ratio (Soaked).

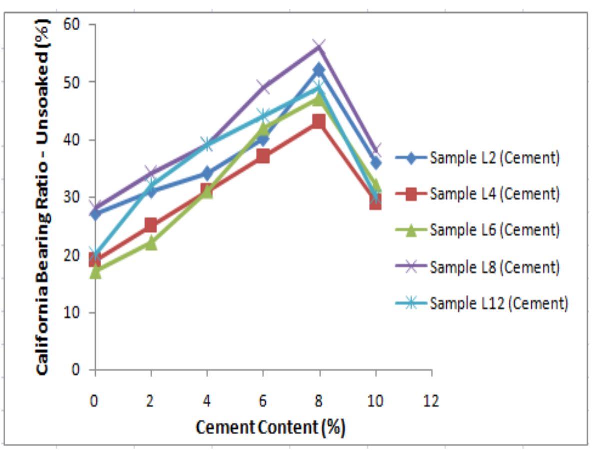

Figure 12: Influence of Cement Content on California Bearing Ratio (Unsoaked).

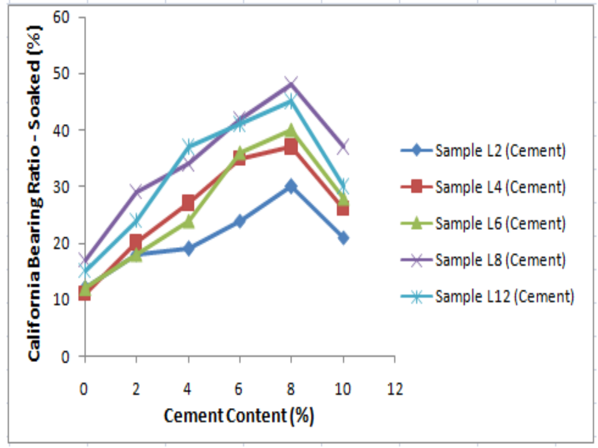

Figure 13: Influence of Cement Content on California Bearing Ratio (Soaked).

airfield pavement design. Soaked CBR test is a simulation of the condition that soils are exposed to insitu upon ingress of water. Figures 10 – 13 present the effects of varying percentages of stabilizers on the unsoaked and soaked CBR of the soil samples.

Stabilization of the soils with PKSA resulted in the increase in both unsoaked and soaked CBR, with the values increasing from 27 – 44% (L2), 19 – 36% (L4), 17 – 38% (L6), 28 – 39% (L8) and 20 – 34% (L12) in unsoaked CBR (Fig. 10) and 12 – 27% (L2), 11 – 21% (L4), 12 – 24 (L6), 17 – 31% (L8), and 15 – 31% (L12) as shown in soaked CBR as shown in Figure 11. The optimal values are obtained upon the addition of 6% PKSA for both unsoaked and soaked CBR. Between 8 and 10% PKSA contents resulted to decrease in the trend of increase of CBR (Figures 10 and 11).

Stabilization of the soils with cement revealed progressive increase in both the unsoaked and soaked CBR with progressive increase in the percentage of cement. Addition of 8% cement optimally improved the CBR of the soils for both the soaked and unsoaked CBR. Furthermore, 10% cement led to reduction in CBR (unsoaked and soaked) for all the soils (Figures 12 and 13). This shows that the load bearing capacities of all the soils increased with the stabilization mix. The minimum CBR requirements for subgrade, subbase and base courses are 10% (soaked), 30% (soaked) and 80% (unsoaked), respectively (Ashworth, 1996).

All the soils are suitable as subgrade/fill materials in their unstabilized and stabilized state, but only soil samples L8 and L12 stabilized with 8% PKSA met the requirements for use as subbase (Figure 11). The addition of between 4 and 10% cement by weight of soil samples make L8 and L12 soils suitable as subbase materials, L4 and L6 stabilized soils are suitable as subbase on addition of 6 – 8% cement contents and L2 soil met the requirement for use as subbase on addition of only 8% cement content by weight of the soil (Figure 13). However, the minimum value of unsoaked CBR for base course is not met by all the soil samples (Figure 12), thus the soils will not be adequate as base material. These are indications that PKSA and cement are effective stabilizers. The results revealed distinct variation in response of the soils to different stabilizers.

Unconfined Compressive Strength (UCS)

Both the CBR and UCS are often used to estimate the bearing capacity of highway subgrade and subbase soils (Gidigasu, 1980). The UCS of the samples increased considerably with the addition of PKSA and cement contents. The UCS increased significantly on addition of 2% to 4% PKSA but decreased gradually thereafter with increasing PKSA content. Addition of 4% PKSA optimally improved the UCS of the soils, strength reductions were observed at other percentages for all the samples. Furthermore, 10% PKSA led to reduction in initial UCS for all the soils (Figure 14).

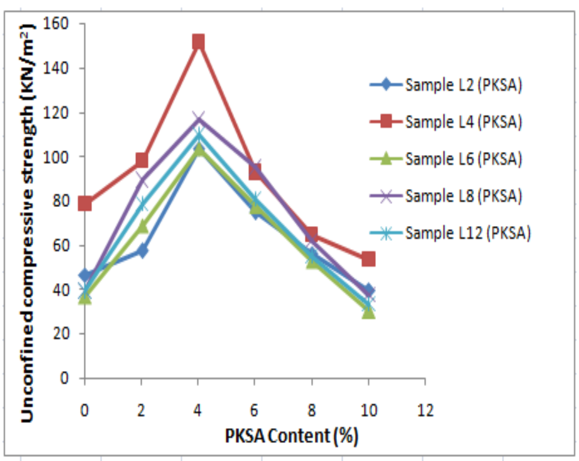

Figure 14: Influence of PKSA Content on Unconfined Compressive Strength.

Similarly, the UCS increased significantly on addition of 2% to 8% cement but decreased gradually thereafter with more than 8% cement content. Addition of 8% cement content optimally improved the UCS of the soils (Figure 15). It is observed that the trend of reduction with the MDD on addition of 10% cement is replicated with the UCS. The UCS increased from 46.6 kN/m^2 to 237.2 kN/m^2, 78.4 kN/m^2 - 250.1 kN/m^2, 36.5 kN/m^2 - 229.2 kN/m^2, 39.2 kN/m^2 - 223.9 kN/m^2, and 39.1 kN/m^2 - 237.6 kN/m^2 upon the addition of 8% cement for all the samples (Figure 15).

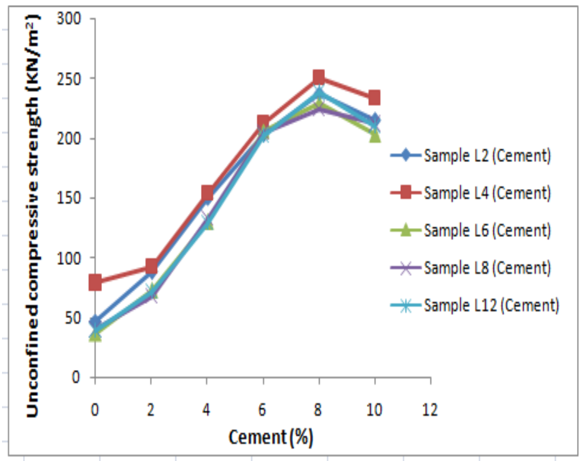

Figure 15: Influence of Cement Content on Unconfined Compressive Strength.

This increase in UCS observed with increase in cement content for all the samples may be as a result of moisture affinity of grains of soil attributable to surface chemical reaction. It is also due to the promotion of cementation and semi rigid framework (Adeyemi and Abolurin, 2000).

These results established the stabilizing potentials of PKSA and cement on residual soils derived from migmatite-gneiss on addition at the optimum level. PKSA and cement improved the UCS of the soils but cement improves the soil properties and enhances the workability of the soils much better than PKSA. Poor subgrade materials are characterized by low stiffness and resistance to deformation which results in pavement failure due to inability to support a high amount of loading (Amer et al., 2014). Low UCS of the soils indicating low strength have been improved by stabilization (PKSA and cement) and categorized as stiff to very stiff to support traffic load (Figures 14 and 15).

Strength indices tests results (UCS and CBR results) were used in model formulation as shown in Figures 16 – 19 with their respective equations (Equations 1 – 4) and R- squared values. This serves as a template in predicting the relationship between the indices in estimation of results with little laboratory work. The models are in form of polynomial equation ($ax^2 - bx + c$) with the values of R^2 indicating the degree of correlation between CBR (unsoaked and soaked) and UCS at various stabilizer contents.

It was observed that 0.158 and 0.121 correlation was obtained for CBR (unsoaked and soaked) and UCS at PKSA content. While, a stronger positive correlation (0.591 and 0.597) was established for CBR (unsoaked and soaked) and UCS at cement content. The results indicate good correlation between CBR and UCS.

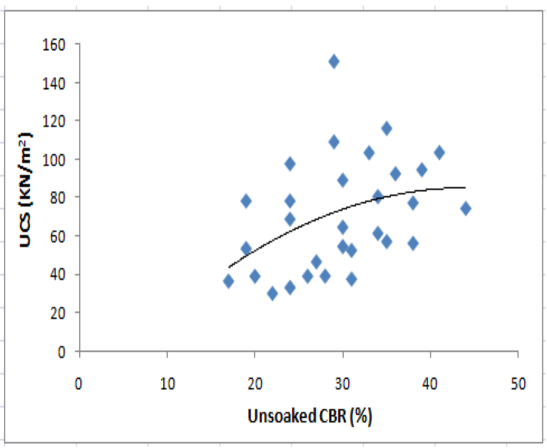

Figure 16: UCS $_{PKSA}$ Vs Unsoaked CBR

$$UCS_{(PKSA)} = 0.056\ CBR_{Un}^2 - 5.008\ CBR_{Un} + 25.37 \ldots (1)$$

$R^2 = 0.158$

Where: $UCS_{(PKSA)}$ = Unconfined Compressive Strength at PKSA content, CBR_{Un} = Unsoaked California Bearing Ratio at PKSA content.

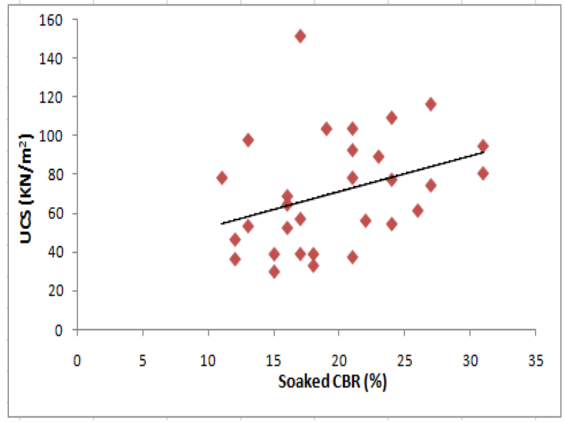

Figure 17: UCS $_{PKSA}$ Vs Soaked CBR.

$$UCS_{(PKSA)} = 0.001\ CBR_S^2 - 1.931\ CBR_S - 33.51 \ldots\ldots\ldots (2)$$

$R^2 = 0.158$

Where: $UCS_{(PKSA)}$ = Unconfined Compressive Strength at PKSA content, CBR_S = Soaked California Bearing Ratio at PKSA content.

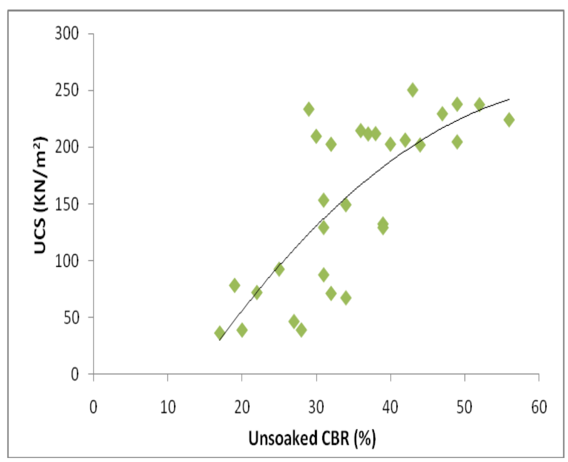

Figure 18: UCS $_{Cement}$ Vs Unsoaked CBR.

$$UCS_{(Cement)} = 0.088\ CBR_{Un}^2 - 11.91\ CBR_{Un} + 147.4 \ldots\ldots\ldots (3)$$

$R^2 = 0.591$

Where: $UCS_{(Cement)}$ = Unconfined Compressive Strength at cement content, CBR_{Un} = Unsoaked California Bearing Ratio at cement content.

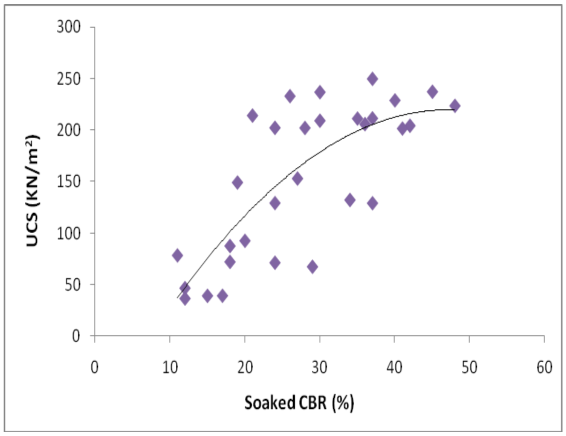

Figure 19: UCS $_{Cement}$ Vs Soaked CBR

UCS $_{(Cement)}$ = 0.139 CBR_S^2 - 13.51 CBR_S + 90.66……… (4)

R^2 = 0.597

Where: UCS $_{(Cement)}$ = Unconfined Compressive Strength at cement content, CBR_S = Soaked California Bearing Ratio at cement content.

CONCLUSION

The moisture content of the migmatite-gneiss derived soils indicates a high water adsorption capability of the soil material as related to the road failure observed along the area. They classify in group A-2-7 of the AASHTO classification system, being silty or clayey gravel with some sand. Based on the Unified soil classification system, the soils are inorganic silt of high plasticity and compressibility (MH-OH). The high plasticity suggests the presence of high proportions of clay minerals. Low bearing capacity and low strength characteristics of the soils show their unsuitability for use as either foundation materials for heavy structures or subgrade, thus, modifications of the natural properties of the soils become necessary for construction purposes.

The addition of PKSA and cement in 2, 4, 6, 8, and 10% to the soil samples caused a change in the liquid limits and plasticity indices of the soil samples. The liquid limits and plasticity indices of the soils dropped to its optimal value on addition of 8% and 6% (PKSA) and 4% and 2% by weight of cement respectively. This established the fact that the activity of the mixture was reduced with marked reduction in swelling nature of the soils. It also indicates a more stable soil with increased workability on addition of PKSA and cement. The stabilization of the soils with PKSA resulted in progressive increase in OMC with corresponding decrease in MDD, addition of 6% PKSA optimally improved the CBR of the soils for both the soaked and unsoaked CBR and 4% PKSA optimally improved the UCS.

Whereas increase in percentage by weight of cement decreased the OMC with increase in MDD. The maximum improvement in the MDD, CBR and UCS including maximum reduction in OMC was observed on the addition of 8% by weight of cement, thus, regarded as the optimum content for improvement of the soils. Addition of more than 8% by weight of cement caused reduction in MDD, CBR and UCS but increase in the OMC. Increase in MDD, CBR and UCS values of all the soils have improved the soils richly to the satisfaction of meeting the minimum requirements that guarantee the soils as subgrade and subbase materials in the construction of flexible road pavement. Therefore, it is highly essential to subject different soils to varying quantities of various stabilizers, to be able to determine the stabilizer(s) and the quantity of such stabilizer(s) that will optimally improve the engineering properties of the soil. Second order polynomial relationships established between CBR (unsoaked and soaked) and UCS with R^2 values indicate good correlation between them.

It is recommended that investigation of PKSA-cement stabilization of soils developed over different parent rocks should be conducted to determine the choice and quantity of stabilizer required to improve the engineering properties of the soils. Adequate drainage should be provided in the area in order to prevent ingress of water below the pavement, which could result in significant loss of strength of the subgrade soils and hence failure of the overlying pavement. Alternative designs are recommended for the existing pavement thicknesses by further understanding of the subgrade properties to prevent continuous road pavement failure.

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