

Sustainable Improvement of Residual Lateritic Soils Using Alum

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Abstract

Cement stabilization has gained popularity in the tropics despite its known side effects. In Nigeria, a developing nation for instance, high cost and negative environmental impacts accompanying the use of cement constitute major drawbacks raising concerns. These shortcomings often make more sustainable alternatives necessary in a depressed economy. The current study explores the effects of alum on swelling potentials and strength of fine-grained Nigerian lateritic soils with a view to ascertaining the viability of the additive as cost-effective, eco-friendly and sustainable substitute for improving engineering properties of the soils as highway embankment or pavement materials. Particle-size analysis and Atterberg limits tests conducted according to British Standards (BS 1377) and ASTM-D4318 respectively, aided in AASHTO classification of the un-treated soils as A-6 (clayey soil), A-4 (silty soil) and A-7-5 (clayey soil). These are poor to fair subgrade materials requiring stabilization. Similarly, high amounts of fines in the natural soils suggest susceptibility to recurring shrink and swell during alternating dry and wet seasons typical of tropical regions. However, alum-treated soils show reduction in plasticity with strong negative correlations. This implies reduction in swelling potentials and improvement in strength with alum treatment. Also, California bearing ratio (CBR) and shear strength, which followed the British Standards indicate strong positive correlations with alum content for the granite- and charnockite-derived soils. This indicates improved strength. Generally, the results suggest that alum has the potential to improve plastic fine-grained lateritic soils in line with the Federal Ministry of Works and Housing general specification for roads and bridges, with combined engineering, economic and environmental benefits.

Keywords: Additive, cement, embankment, highway, subgrade, soil stabilization, swelling soils

DOI: 10.7176/JSTR/7-10-06

1. Introduction

Lateritic soils are common residual tropical weathering products abundantly developed over various rock types across wide geological terrains in Nigeria. Their wide distribution keeps them in high demand in the construction industry. However, reports show that they exhibit varying degrees of objectionable engineering properties which have necessitated modification of engineering properties of Nigerian soils to suit various construction and foundation uses. The deficiencies in engineering properties include excessive amounts of fines and the consequent high natural moisture content (Onuoha *et al.*, 2014; Arumala and Akpokodje, 2016). This is against the backdrop of one of the major criteria for stability and suitability of lateritic soils for use as construction materials, that is, proper proportion of constituent clay-sized proportions (and clayey materials) which act as binder (Ackroyd, 1960). Besides, the sensitivity of these soils to moisture and resultant shrinking or swelling behaviour depends on the percentage of constituent clay-size fractions. Hence, some residual tropical soils were adjudged unsuitable as base course and subgrade materials for possessing high proportions of fines in parts of the region (Adeyemi and Oyeyemi, 2000; Onuoha *et al.*, 2014; Arumala and Akpokodje, 2016). Failure of many engineering projects such as highway pavements, dams and buildings have been attributed to several geotechnical factors in different parts of the country (Akpokodje, 1986; Ademila, 2017; Adenika *et al.*, 2018; Owoseni and Atigro, 2019). Therefore, the role of proper soil investigation

in engineering use of soils cannot be overemphasized. Where necessary, soils are stabilized to achieve some desirable engineering performance and optimal usefulness. Various researchers have attempted to stabilize engineering soils using mechanical methods (Yilmaz, 2015), chemical techniques (Arulrajah *et al.*, 2017; Hassan *et al.*, 2017; Latifi and Meehan, 2017; Rashid *et al.*, 2017; Latifi *et al.*, 2018), biological means (Lee *et al.*, 2017; Dehghan *et al.*, 2018; Hataf *et al.*, 2018; Shankar *et al.*, 2019) and electrical approaches (Tjandra and Wulandari, 2007; Hojati, 2017) for optimal use in various geotechnical construction works. Mechanical methods such as compaction produce increased soil strength, reduced compressibility, minimized volume change, lowered permeability and reduction in susceptibility to frost action. Chemical soil strengthening techniques sometimes take the form of mixing chemical additives with or injecting chemical substances into the soil. This approach of using chemical additives is still rated as the most common method of soil stabilization (Hataf *et al.*, 2018).

From literature, the commonly used traditional chemical additives for soil strengthening include Portland cement, lime and bituminous materials (Joel and Agbede, 2010; Patel and Mishra, 2014; Mousavi and Sing, 2015; Pelliser *et al.*, 2015; Hojati and Radlinska, 2017; Debnath and Dey, 2018). Cement has consistently gained global acceptance as chemical additive since its introduction as soil strengthener in the 1960s (Makusa, 2012; Chang *et al.*, 2015; Al-Bared and Marto, 2017; Rashid *et al.*, 2018). However, its negative environmental impacts which include emission of carbon-dioxide (CO₂), nitrous oxide (NO_x) and particulate air (Chang *et al.*, 2015; Hataf *et al.*, 2018) are key shortcomings. These limitations often necessitate sustainable alternatives, especially in depressed economy like in Nigeria. In the last few decades, the feasibility of other maintainable alternatives to traditional chemical additives like cement has been an area of active research globally. Such researches are exploring ways of maximizing the use of available but objectionable soil resources for construction purposes by studying the viability of some common but cost-effective, eco-friendly and sustainable chemical additives as substitutes to the depleting and expensive traditional stabilizers (Rashid *et al.*, 2017; Tabarsa *et al.*, 2018; Yaghoubi *et al.*, 2018).

More recently, the chemical additives which researchers have investigated for soil improvement include recycled tiles and tyres for concrete (Al-Bared *et al.*, 2018; Al-Bared *et al.*, 2018b), xanthan and guar gums for fine-grained collapsible soils (Chang *et al.*, 2015; Dehghan *et al.*, 2018), liquid polymer for tropical peat (Latifi *et al.*, 2019) and, waste materials (Mujah *et al.*, 2015; Ilies *et al.*, 2017). Besides, agricultural wastes have also been evaluated for possible use as engineering soil stabilizers in some parts of the world (Chittaranjan, 2011). In Nigeria, the agricultural wastes which have been investigated for efficient, eco-friendly, cost-effective and sustainable improvement of tropical soils, groundnut shell ash (Oriola and Moses, 2010), coconut shell and husk ash (Amu *et al.*, 2011), and, rice-husk ash (Owoseni and Atigro, 2014). In all these, the engineering performance of tropical soils was reportedly improved without any of the traditional additives. In the same vein, the current study examines the viability of alum as an alternative to the conventional stabilizers for sustainable tropical soil improvement for construction purposes in Nigeria. The work elucidates the effects of alum on the swelling potentials and strength characteristics of some lateritic soils as highway embankment and flexible pavement construction materials in Akure metropolis, southwestern Nigeria, where road failures have been persistently observed (Fig. 1).

1.1. The study Area

The study area, as shown in Fig. 2 (Federal Surveys Nigeria, 1966) lies within Latitudes 7° 12' N and 7° 19' N and Longitudes 5° 08' E and 5° 16' E. The area has humid tropical climate of the West African monsoonal type, characterized by distinct wet and dry seasons. Annual rainfall sometimes rises to a mean value of about 1350mm with high temperatures reaching a peak of about 32° C around February and a threshold of about 21° C around August. Relative humidity ranges from about 70% around January to about 90% in July. Both factors of rainfall and elevated temperature enhance the disintegration of rocks through chemical processes of weathering. The climate is considered the most important primary factor of lateritization; thus, Maignien (1966) suggested that contemporary lateritic soils have developed at mean annual temperatures of about 25° C. Lateritic soils are generally believed to always correspond to climates in which the wet period is warm. Typically, the vegetation of the study area is the tropical rain forest with thick undergrowth but had in many parts been modified by human activities such as urbanization, construction, land cultivation and deforestation (Maignien, 1966).



Figure 1. Failed sections of pavements in Akure, adjacent to sampling points (i) S1 (ii) S2 (iii) S3

1.2. Geological Setting

The area under investigation lies within the Precambrian Basement Complex (PBC) of Nigeria. Researchers have variedly classified the assemblages of the PBC into four lithological groups, viz: minor intrusives, Older granites, schist belts and migmatite-gneiss-quartzite complex (Oyinloye and Odeyemi, 2001; Anifowose and Borode, 2007). The most prominent of these lithologic units in the study area are the minor intrusive rocks, the Older granites and the migmatite-gneiss-quartzite complex (Fig. 3) (Nigeria Geological Survey Agency, 2006). Granite, gneisses and charnockites (from Oba-Ile area, Oke-Aro and Ire-Akari areas respectively) constitute the local geology in the study area. The rock assemblages vary texturally from medium to coarse (porphyritic) grains. Generally, the fissure zones trend NS while the foliation trends are NNW-SSE and NNE-SSW (Olorunfemi *et al.*, 1999).

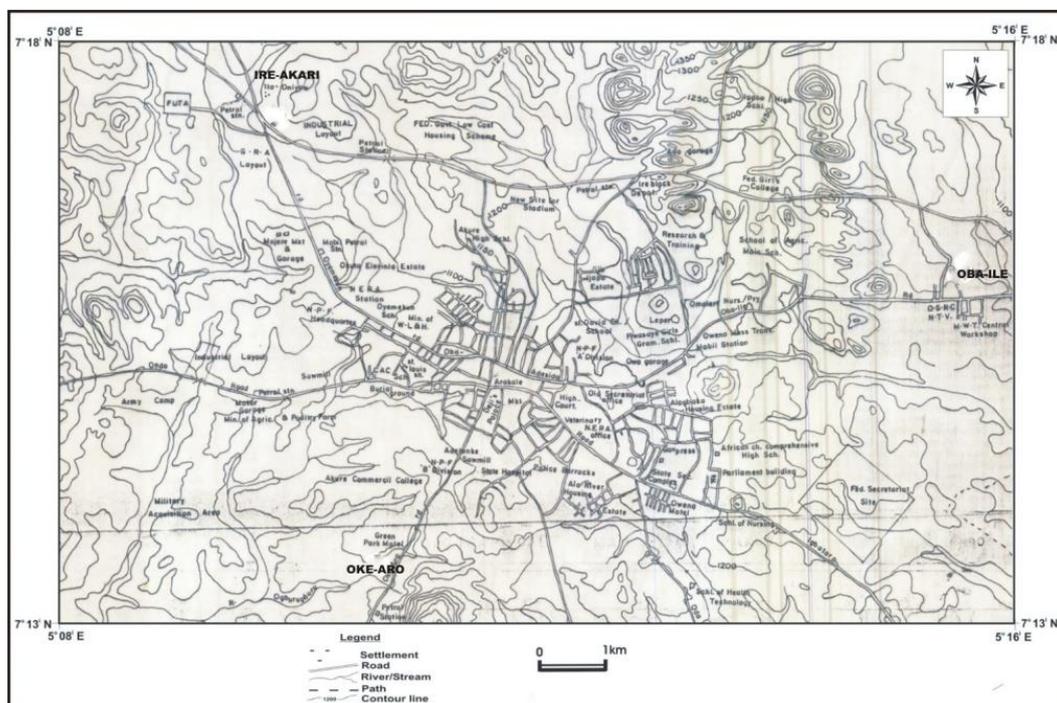


Figure 2. Topographic map of Akure environs (Federal Surveys Nigeria, 1966)

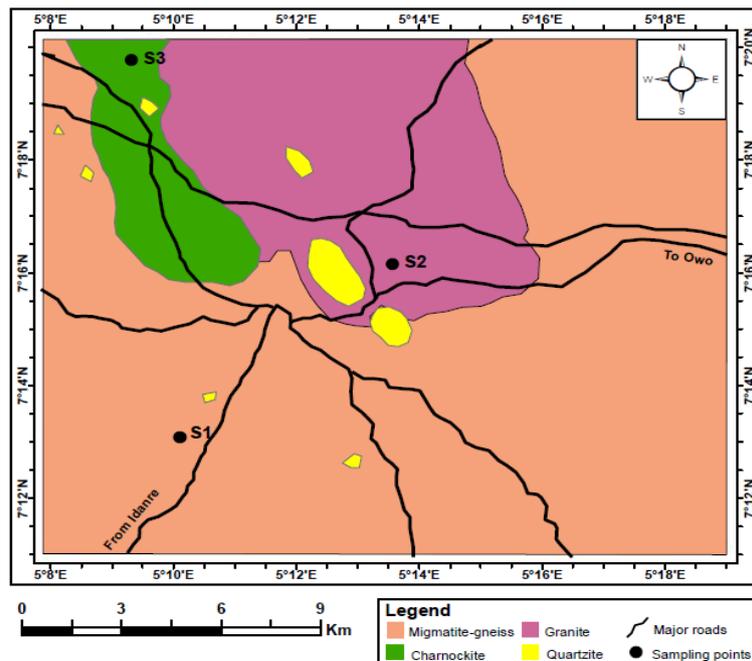


Figure 3. Geological map of Akure (Modified after Nigeria Geological Survey Agency, 2006)

2. Materials and methods

2.1. Residual lateritic soil

Three disturbed representative bulk samples of granite-, charnockite-, and migmatite-gneiss-derived lateritic soils were collected at depth of 1.0 m below the ground surface, and were air-dried under laboratory conditions for 7 days.

2.2. Dissolve alum

The alum used for treating the soils was sourced locally from a market in Akure metropolis. In chemical terms, alum is a group of double salts that contain aluminum sulphate coupled with potassium or ammonium sulphate. It is a water-soluble mineral with density of about 1.76 and is commonly used in water treatment due partly to its solubility in water. It used to be a raw material in the textile industry as a mordant or fixing agent for holding natural dyes to fabrics. It has also been applied to improve the physical properties of potting media and to act as liming material (Bugbee and Frink, 2006). It has been shown to be effective in improving soil aggregate stability and decreasing aggregate rupture stress (Eltaif and Gharaibeh, 2008).

2.3. Sample Preparation

The potash alum used as additive was pulverized after which wet mixing was done. Specified amounts (5, 10, and 15 g/l) of the pulverized alum were first dissolved in distilled water to give hydro-solution. Thereafter, the dissolved alum solutions were mixed in turn with soil samples at room temperature. The mixing was done by hand to ensure proper soil-alum-water admixture. This procedure was repeated for all the soil samples for the different dosages of the additive.

2.4. Test Procedure

For the un-treated soils, air-dried samples were used for the consistency limits tests. To further investigate the plasticity and strength characteristics of the soil-water-alum admixture, three basic engineering tests were conducted, viz: consistency limits, CBR and Triaxial unconsolidated-undrained tests. The consistency limits tests followed the ASTM-D4318 specifications while the CBR and triaxial tests were conducted in accordance with the British standard, BS 1377 (1990) specifications. For the triaxial unconsolidated-undrained test, confining pressures of 100, 200 and 300 kPa were applied.

2.5. Data Analysis

The geotechnical data obtained from the laboratory analyses were subjected to univariate and bivariate statistical treatments. Results of statistical data analysis were presented as scatter plots and graphical regression lines. The effects of the stabilizer on the geotechnical properties of the residual soils were further understood and quantitative relationships between plasticity index, CBR and shear strength, and varying amounts of alum were established.

3. Results and Discussion

3.1. Some geotechnical properties of un-stabilized soils

The results of the laboratory geotechnical analyses on the un-stabilized soil samples are summarized in Tables 1 and 2.

3.1.1. Soil classification

The results of particle-size distribution analysis of the un-treated soils (Table 1) showed that migmatite-gneiss-derived soil (S1) has the least amounts of fines (56.30%) and highest coarse fractions (43.70%). The charnockite-derived soil (S3) exhibited the highest amounts of fines and the least coarse fractions (96.10% and 3.90% respectively). For the granite-derived soil (S2), the values are relatively intermediate (75.80% fine and 24.20% coarse fractions respectively). These wide variations in particle distributions reflect the mineralogy and textural characteristics of the parent rocks. The high amounts of fines suggest that the three soils are susceptible to recurrent shrink and swell during alternating dry and wet seasons characteristic of the tropics (Ademila, 2017). None of the un-treated soils meet the Nigerian specifications of $\leq 15\%$ fines for base course materials or $\leq 35\%$ fines for embankment and general filling purposes. Based on the particle-size distributions, liquid limit and plasticity index characteristics, the un-treated migmatite-gneiss-, granite- and charnockite-derived soil samples fall into groups A-6 (clayey soil), A-4 (silty soil) and A-7-5 (clayey soil) of the AASHTO classification system. This implies that the soils are fair to poor subgrade materials which would require special treatment to meet the requirements for subgrade materials in highway construction. Hence, the need for stabilizing the soils.

3.1.2. Natural moisture content and specific gravity

As expected, the clayey charnockite-derived soil with the highest amounts of clay-sized fractions, exhibited the highest natural moisture content value (22.2 %) which exceeds the 5 – 15% specified for engineering construction in Nigeria (Federal Ministry of Works and Housing, 2000). This suggests high water absorption tendency for the soil. Next ranked in natural moisture content (14.19%) is the clayey migmatite gneiss-derived soil with medium clay-sized fractions. The silty granite-derived soil with the lowest proportion of clay-sized particles indicated the least moisture content (5.09 %). The moisture contents of the migmatite-gneiss- and granite-derived soils fall within the Nigerian specification. These wide-ranging moisture content values reflect differences in clay mineralogy and textural characteristics of the parent rocks. Also, the specific gravity of the soils varied from 2.64 to 2.70. Since the degree of soil maturity is directly related to specific gravity, the migmatite-gneiss-derived soil with the highest specific gravity value (2.70) is the most matured and lateritized while the charnockite-derived soil with the lowest value (2.64) may have undergone the least degree of lateritization.

Table 1. Results for particle size distribution and specific gravity of the un-stabilized soils

| Sample no. | Parent Rock | Specific gravity | % Gravel | % Sand | % Fine |
|------------|---------------------|------------------|----------|--------|---------------|
| | | | | | % silt + clay |
| S1 | Migmatite Gneiss | 2.70 | 9.08 | 34.65 | 56.27 |
| S2 | Granite | 2.65 | 0.57 | 23.65 | 75.78 |
| S3 | Charnockite | 2.64 | 0.00 | 3.91 | 96.09 |

3.1.3. Plasticity characteristics

The liquid limit (LL) and plasticity index (PI) values for the soil samples varied from 20.40 to 48.78 and 5.40 to 20.87 respectively (Table 2). Only the silty granite-derived soil (with LL = 20.40 and PI = 5.40) meet the Nigerian specification of $LL \leq 30$ and $PI \leq 13$ for base course materials. Hence, soil stabilization is necessary for the clayey migmatite-gneiss- and charnockite-derived soils. From the liquid limit values, the un-treated granite-derived soil (S1 with LL = 20.40) is non-plastic while the migmatite- and charnockite-derived soils (S2 and S3 with LL = 35.85 and 48.78 respectively) possess intermediate plasticity (Table 3) (Bell, 2007). The plasticity index values for the un-treated soils varied from 5.40 to 20.87 %. According to Ola (1981) who classified swelling potential based on plasticity index values (Table 4), the granite-derived soil (S1) exhibited low swelling potential while the migmatite-gneiss- and charnockite-derived soils (S2 and S3 respectively) possessed medium swelling potentials.

3.1.4. Strength characteristics

3.1.4.1 California Bearing Ratio (CBR)

The results of California bearing ratio (CBR) test often indicate suitability or otherwise of engineering soils in road construction as base course, subbase and subgrade materials (Ademila, 2017). The unsoaked CBR (30%, 66%, 14%) for S1, S2 and S3 respectively (Table 2) all fall below the required 80% minimum value recommended for highway subgrade and subbase soils by the Federal ministry of works and housing (FMWH, 2000) (Table 8). According to FMWH specification, both S1 and S2 are suitable as base course materials while S3 falls short of the required minimum value of 25%.

3.1.4.2 Unconfined Compressive Strength (UCS)

The results of unconsolidated undrained test give clue on the consistency of the clayey soils in terms of unconfined compressive strengths, thus indicating the performance of the soils as engineering foundation materials (Ademila, 2017). The Unconfined Compressive Strength (UCS) values obtained for migmatite-gneiss-, granite-, and charnockite-derived soils are 69.82 kPa, 23.95 kPa and 101.71 kPa respectively (Table 2). According to Das (2000), these values indicate medium, very soft and stiff consistency classes respectively (Table 5). These are indications of poor subgrade materials liable to pavement failure and requiring some sorts of improvements for better engineering performance.

3.1.4.3 Shear strength

For the triaxial unconsolidated-undrained test, confining pressures of 100, 200 and 300 kPa were applied to determine the stress-strain characteristics and shear strengths of soils. The test results (Table 2) show low cohesion (39.50 kPa – 64.20 kPa) and angles of internal friction (14.80° – 24.10°).

Table 2. Summary of the geotechnical properties of un-stabilized soils

| Sample No. | Parent Rock | M_c (%) | LL (%) | PL (%) | PI (%) | k (m/s) | CBR (%) | UCS (kPa) | Compaction | | Triaxial | |
|------------|------------------|-----------|--------|--------|--------|-----------|---------|-----------|------------|--------------------------|-------------|--------------|
| | | | | | | | | | OMC (%) | MDD (Kg/m ³) | C_u (kPa) | ϕ_u (°) |
| S1 | Migmatite Gneiss | 14.19 | 35.85 | 20.83 | 15.02 | 0.36 | 30 | 69.82 | 16.40 | 1745 | 43.80 | 24.10 |
| S2 | Granite | 5.09 | 20.40 | 15.00 | 5.40 | 0.29 | 66 | 23.95 | 9.60 | 1958 | 39.50 | 14.80 |
| S3 | Charnockite | 22.20 | 48.78 | 27.91 | 20.87 | 0.15 | 14 | 101.71 | 26.50 | 1559 | 64.20 | 16.30 |

M_c natural moisture content; LL liquid limit; PL plastic limit; PI plasticity index; k hydraulic conductivity; τ shear strength; OMC optimum moisture content; MDD maximum dry density; CBR California bearing ratio; UCS unconfined compressive strength.

Table 3. Relating liquid limit to plasticity of soils (Bell, 2007)

| Liquid limit (%) | Plasticity | Results |
|------------------|--------------|------------------------|
| < 35 | Low | S2 (20.40) |
| 35 – 50 | Intermediate | S1 (35.85), S3 (48.78) |
| 50 – 70 | High | Nil |
| 70 - 90 | Very high | Nil |
| > 90 | Extra high | Nil |

Table 4 Relating plasticity index to swelling potential of soils (Ola, 1981]

| Plasticity Index | Swelling potentials | Results |
|------------------|---------------------|------------------------|
| 0 – 15 | Low | S2 (5.40) |
| 15 – 25 | Medium | S1 (15.02), S3 (20.87) |
| 25 – 35 | High | Nil |
| > 35 | Very high | Nil |

Table 5 Relating Unconfined Compressive Strength to consistency of soils (Das, 2000)

| Unconfined Compressive Strength (kPa) | Consistency | Results |
|---------------------------------------|-------------|-------------|
| 0 – 25 | Very soft | S2 (23.95) |
| 25 – 50 | Soft | Nil |
| 50 – 100 | Medium | S1 (69.82) |
| 100 – 200 | Stiff | S3 (101.71) |
| 200 – 400 | Very stiff | Nil |
| > 400 | Hard clay | Nil |

3.2. Effects of alum on stabilized soils

The results of the laboratory geotechnical analyses on the alum-stabilized soil samples are summarized in Tables 6. The geotechnical data are further subjected to statistical analysis and the descriptive statistics presented in Table 7. The standard deviation (SD) shows the measure of variability for each single data set. The degree of dispersion among the variables measured in different units is better understood from the coefficient of variation (CV). The results show that unconfined compressive strength (UCS) and liquid limit (LL) have the highest and least CV (105.45 and 33.21 %) respectively. These indicate that the data points for UCS are spread out over the largest range of values while the data points for LL tend to be the closest to the mean.

3.2.1. Plasticity and swelling characteristics

The results of Atterberg limit tests (Table 6) conducted on alum-stabilized soil samples showed that addition of 5% alum produces a drastic reduction in plasticity of the three soils while the reducing effect was progressively more or less gradual on addition of 10% and 15% alum content in turn. For the migmatite-gneiss-derived soil, the Liquid limits (LL) reduced drastically from 35.85% (un-stabilized state) to 24.89%, then steadily to 24.25% and finally to 20.59% with 5,10 and 15 g/l alum contents respectively. The plasticity index (PI) dropped appreciably from 15.02 (for the un-stabilized state) to 11.08 (26.23% reduction), then steadily to 9.65 (35.75% reduction) and further to 8.69 (42.14% reduction) when treated with 5,10 and 15 g/l alum content respectively. Here, there are indications that the geotechnical performance with respect to plasticity, was improved upon from medium to low plasticity with the addition of alum. Also, for the granite-derived soil, the PI reduced drastically from 5.40 for the natural soil to progressively increasing non-plastic state with increased alum content (0%

through 10% to 15%).

Table 6. Summary of consistency limits, CBR and UCS tests for alum-stabilized soils

| Sample no. | Alum content (g/l) | LL (%) | PL (%) | PI (%) | CBR (%) | Shear Strength (kPa) |
|------------|--------------------|--------|--------|--------|---------|----------------------|
| S1 | 0 | 35.85 | 20.83 | 15.02 | 30.00 | 34.91 |
| | 5 | 24.89 | 13.81 | 11.08 | 26.00 | 13.97 |
| | 10 | 24.25 | 14.60 | 9.65 | 23.00 | 12.89 |
| | 15 | 20.59 | 11.90 | 8.69 | 22.00 | 9.24 |
| S2 | 0 | 20.40 | 15.00 | 5.40 | 66.00 | 11.98 |
| | 5 | 14.80 | 29.30 | NP* | 71.00 | 13.81 |
| | 10 | 18.70 | 28.70 | NP* | 73.00 | 13.97 |
| | 15 | 29.50 | 30.30 | NP* | 76.00 | 14.85 |
| S3 | 0 | 48.78 | 27.91 | 20.87 | 14.00 | 50.86 |
| | 5 | 29.81 | 13.95 | 15.86 | 19.00 | 93.17 |
| | 10 | 30.35 | 16.80 | 13.55 | 17.00 | 46.73 |
| | 15 | 27.69 | 14.90 | 12.79 | 26.00 | 136.50 |

*NP non-plastic CBR California bearing ratio; UCS unconfined compressive strength; kPa kilopascal

Table 7. Descriptive statistics of geotechnical data for alum-stabilized soils

| Variable | Units | n ^a | Min. | Max. | Mean | SD | CV (%) |
|----------|-------|----------------|-------|--------|-------|-------|--------|
| LL | % | 12 | 14.80 | 48.78 | 27.13 | 9.01 | 33.21 |
| PL | % | 12 | 11.90 | 30.30 | 19.83 | 7.15 | 36.06 |
| PI | % | 12 | 0.00 | 20.87 | 12.55 | 4.53 | 36.10 |
| CBR | % | 12 | 14.00 | 76.00 | 38.58 | 24.77 | 64.20 |
| UCS | kPa | 12 | 18.47 | 273.10 | 75.47 | 79.58 | 105.45 |

LL liquid limit, PL plastic limit, PI plasticity index, CBR California bearing ratio, UCS unconfined compressive strength, n^a number of samples, Min minimum, Max maximum, SD standard deviation, CV coefficient variation.

Moreover, the LL for the charnockite-derived soil dropped considerably from 48.78 (for un-stabilized state) to 29.81 (38.89% drop), picked a little up to 30.35 (37.78% drop) and fell again to 27.69 (43.23% fall) when stabilized with 5,10 and 15 g/l alum content respectively. PI reduced from 20.87 (for un-stabilized state) to 15.86 (24.01 % reduction), 13.55 (35.07 % reduction) and 12.79 (38.72 % reduction) with 5, 10 and 15 g/l alum content respectively (Figure 4a). Here also, the soil improved from medium plasticity to low plasticity. Furthermore, the study reveals very strong negative correlations of -0.95, -0.77 and -0.92 between alum content and PI for gneiss-, granite-, and charnockite-derived soils respectively (Figures 4b). The results of consistency limits indicate that the dissolved alum content of 5–15% improved the plasticity characteristics of the investigated soils.

In all, the addition of alum improved the engineering geological properties of the soil samples, especially by significantly reducing their plasticity indices, an indication of reduction in swelling potential which could translate to an increase in strength characteristics.

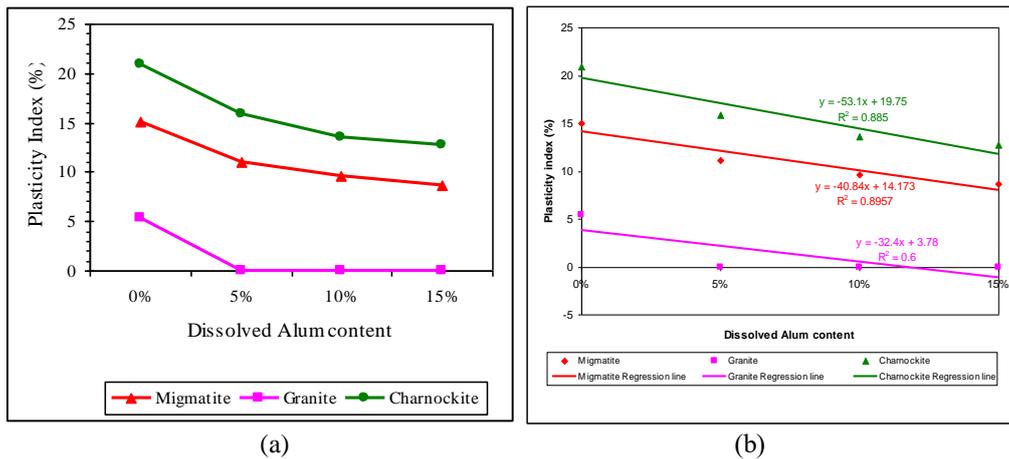


Figure 4. Effects of varied proportions of alum on different soil samples (a) Relative reduction in Plasticity Indices (b) Regression lines of Plasticity indices with Alum contents

3.2.2. Strength Characteristics

3.2.2.1 California Bearing Ratio (CBR)

Generally, the results of CBR tests (Table 6) indicate that addition of dissolved alum has positive effects (CBR increase) on the granite- and charnockite-derived soils while the effect on migmatite-gneiss-derived soils is rather adverse, resulting in CBR reduction (Figure 5a). For the granite-derived soil, the CBR rose drastically by 5% (from 66% for un-stabilized soil to 71%), then dropped gradually by 2% (to 73%) and finally rose again to 76% with dissolved alum content of 5, 10 and 15 g/l respectively. Moreover, for the charnockite-derived soil, the CBR increased from 14% for un-stabilized soils to 19%, 17% and 26% with dissolved alum content of 5, 10 and 15 g/l respectively. However, for the migmatite-gneiss-derived soil, the CBR value reduced from 30% for un-stabilized soil to 26%, 23% and 22% with dissolved alum content of 5, 10 and 15 g/l respectively. This study reveals very strong positive correlations (0.98 and 0.86) between alum content and CBR for the granite- and charnockite-derived soils respectively. However, there is a negative correlation (-0.97) between alum content and CBR for the gneiss-derived soil (Figure 5b). From the foregoing, dissolved alum (about 5–15%) may be adjudged suitable for both the granite- and charnockite-derived soils while other stabilizing agents may be used for the migmatite-gneiss-derived soil. The results also indicated that load bearing capacity increased when dissolved alum was added to both granite- and charnockite-derived soil samples. This load bearing capacity (or ratio) would increase the longevity of road, improve the soil aggregate stability and decrease aggregate rupture stress.

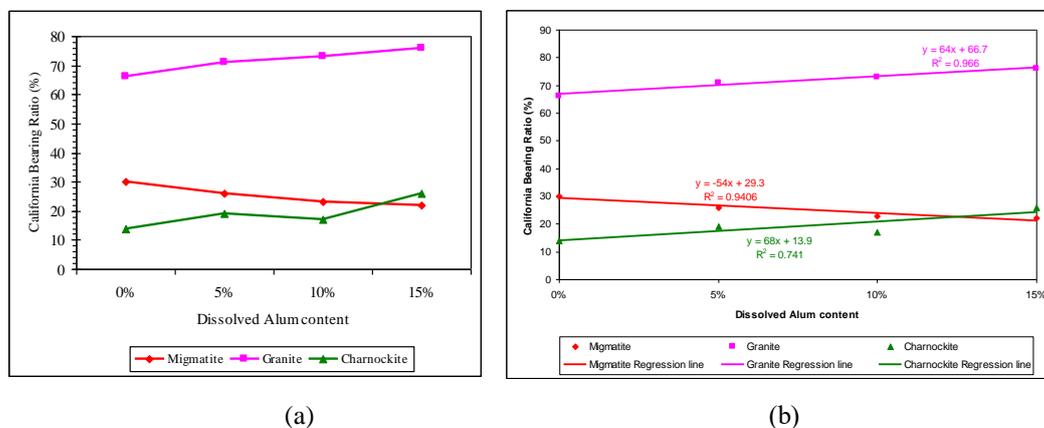


Figure 4. Effects of varied proportions of alum on different soil samples (a) Relative increment in CBR (b) Regression lines of CBR with Alum contents

3.2.2.2 Shear Strength

Table 4 shows the variation of Shear strength with dissolved alum content for the soils investigated. It is obvious from Fig. 5a that increasing dissolved lime has pronounced positive effect (increase in shear strength) on the charnockite-derived soil, and moderately positive effect on the granite-derived soil, while the effect is adversely negative (reduction in shear strength) on the migmatite-gneiss-derived soil. For the granite-derived soil the shear strength rose from 11.98 Kpa for unstabilized granite-derived soils to 13.81, 13.97, and 14.85 Kpa with dissolved alum content of 5, 10 and 15 g/l, implying 15.28%, 16.61%, and 23.96% strength improvement respectively. Also, for the charnockite-derived soil, the shear strength improved drastically from 50.86 Kpa (unstabilized state) to 93.17Kpa, dropped to 46.73Kpa, and sky-rocketed again to 136.50Kpa with dissolved alum content of 5, 10 and 15 g/l respectively. However, for the gneiss-derived soil, the shear strength reduced from 34.91Kpa to 13.97Kpa, then fell further to 12.89Kpa and then to 9.24Kpa with dissolved alum content of 5, 10 and 15g/l respectively. Figure 5b shows the improvement in shear strength of the granite- and charnockite-derived soils with positive correlations of 0.94 and 0.65 respectively between alum content and shear strength. In contrast however, the shear strength was worsened with a negative correlation of -0.87 between alum content and shear strength for the gneiss-derived soil. Summarily, the addition of alum has improved the shear strength characteristics of both the granite- and charnockite-derived soils appreciably.

4. Conclusion

This study investigated the viability of alum as potential alternative soil additive to cement for improved soil engineering performance with benefits of eco-friendliness, cost-effectiveness and sustainability. Laboratory investigations which include Atterberg limits, California bearing ratio and triaxial tests were conducted for residual lateritic soils on different crystalline basement rocks in southwestern Nigeria. The effects of alum on the swelling potentials and strength properties of the soils were investigated for use highway embankment construction materials in Nigeria. The following conclusions can be drawn based on the study:

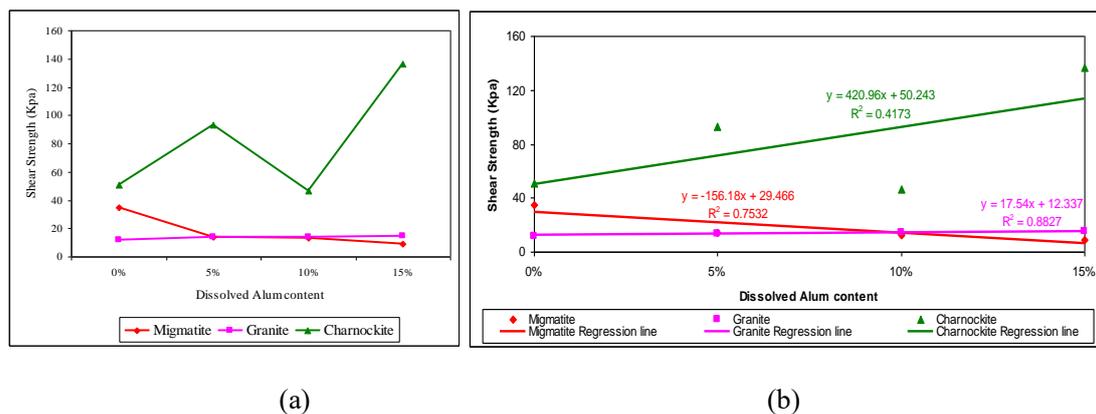


Figure 5. Effects of varied proportions of alum on different soil samples (a) Relative increment in Shear strength (b) Regression lines of Shear strength with Alum contents

1. The addition of alum reduced the plasticity of the gneiss- and charnockite-derived soils by 26.23 – 42.14% and 24.01 – 38.72% respectively. There were strong negative correlations (-0.95, -0.77 and -0.92 respectively) between alum contents and plasticity indices. Significant reduction in plasticity indices implied reduction in swelling potential and hence an increase in strength properties.
2. The California bearing ratio (CBR) of alum treated granite- and charnockite-derived soils increased by 5 - 10% and 5 – 12% respectively. Strong positive correlations (0.98 and 0.86 respectively) were also established. The observed improvement in load bearing capacity indicates higher strength which would enhance longevity of road and stability of soil aggregates under vehicular load.

3. The shear strengths of treated granite- and charnockite-derived soils improved by 15.28 – 23.96% and 83.19 – 168.38% respectively. Strong positive correlations were obtained between alum content and shear strengths of treated soils (0.94 and 0.65 respectively).

Generally, the results suggest that alum has the potential to improve plastic fine-grained lateritic soils in line with the Federal Ministry of Works and Housing general specification for roads and bridges (Table 8), with combined engineering, economic and environmental benefits. Hence, alum improved the swelling and strength properties of the residual soils. Hence, alum can be adjudged a viable substitute for stabilizing the granite- and charnockite-derived soils in Akure for use in highway embankment and pavement construction with combined engineering, economic and environmental benefits.

Table 8 Geotechnical properties of un-stabilized and alum-stabilized soils compared with Nigerian specifications (FMWH, 2000)

| Property | Nigerian Specification | Un-stabilized soils | S1 | | | | | S2 | | | | | S3 | | | | |
|---|------------------------|---------------------|--------------------|-------|-------|-------|--------------|--------------------|-------|-------|--------------|-------|--------------------|-------|--|--|--|
| | | | Alum content (g/l) | | | | | Alum content (g/l) | | | | | Alum content (g/l) | | | | |
| | | | 0 | 5 | 10 | 15 | 0 | 5 | 10 | 15 | 0 | 5 | 10 | 15 | | | |
| <i>Embankment and general fill material</i> | | | | | | | | | | | | | | | | | |
| Liquid limit | < 40 | 20.40 – 48.78 | 35.85 | 24.89 | 24.25 | 20.59 | 20.40 | 14.80 | 18.70 | 29.50 | 48.78 | 29.81 | 30.35 | 27.69 | | | |
| Plasticity Index | < 20 | 5.40 – 20.87 | 15.02 | 11.08 | 9.65 | 8.69 | 5.40 | NP | NP | NP | 20.87 | 15.86 | 13.55 | 12.79 | | | |
| <i>Sub-base course</i> | | | | | | | | | | | | | | | | | |
| Liquid limit | < 35 | 20.40 – 48.78 | 35.85 | 24.89 | 24.25 | 20.59 | 20.40 | 14.80 | 18.70 | 29.50 | 48.78 | 29.81 | 30.35 | 27.69 | | | |
| Plasticity Index | < 16 | 5.40 – 20.87 | 15.02 | 11.08 | 9.65 | 8.69 | 5.40 | NP | NP | NP | 20.87 | 15.86 | 13.55 | 12.79 | | | |
| Un-soaked CBR | ≥ 25 | 14.00 – 66.00 | 30.00 | 26.00 | 23.00 | 22.00 | 66.00 | 71.00 | 73.00 | 76.00 | 14.00 | 19.00 | 17.00 | 26.00 | | | |
| <i>Base course</i> | | | | | | | | | | | | | | | | | |
| Liquid limit | ≤ 35 | 20.40 – 48.78 | 35.85 | 24.89 | 24.25 | 20.59 | 20.40 | 14.80 | 18.70 | 29.50 | 48.78 | 29.81 | 30.35 | 27.69 | | | |
| Plasticity Index | ≤ 35 | 5.40 – 20.87 | 15.02 | 11.08 | 9.65 | 8.69 | 5.40 | NP | NP | NP | 20.87 | 15.86 | 13.55 | 12.79 | | | |
| Un-soaked CBR | ≥ 80 | 14.00 – 66.00 | 30.00 | 26.00 | 23.00 | 22.00 | 66.00 | 71.00 | 73.00 | 76.00 | 14.00 | 19.00 | 17.00 | 26.00 | | | |
| <i>Subgrade</i> | | | | | | | | | | | | | | | | | |
| Liquid limit | ≤ 40 | 20.40 – 48.78 | 35.85 | 24.89 | 24.25 | 20.59 | 20.40 | 14.80 | 18.70 | 29.50 | 48.78 | 29.81 | 30.35 | 27.69 | | | |
| Plasticity Index | ≤ 20 | 5.40 – 20.87 | 15.02 | 11.08 | 9.65 | 8.69 | 5.40 | NP | NP | NP | 20.87 | 15.86 | 13.55 | 12.79 | | | |
| Un-soaked CBR | ≥ 80 | 14.00 – 66.00 | 30.00 | 26.00 | 23.00 | 22.00 | 66.00 | 71.00 | 73.00 | 76.00 | 14.00 | 19.00 | 17.00 | 26.00 | | | |

S1: migmatite-gneiss-derived soil; S2: granite-derived soil; S3: charnockite-derived soil; CBR: California bearing ratio; FMWH: Federal ministry of works and housing.

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