Insights from the Engineering Geological Mapping of Four Basement Rocks Derived Soils

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Abstract. Due to the rapid expansion and associated construction of civil engineering structures on the Federal University of Technology, Akure (FUTA) campus, there arose an urgent need for an engineering geological mapping of the underlying soils (residual soils). Generalized geological mapping revealed four types of basement rocks namely migmatite-gneisses, granites, quartzites and charnockites. Results from the fifty soil samples from twenty-five test pits collected all over the spread of the campus coverage of 6.4 km² revealed that the campus is underlain by soils of granular and clayey composition, generally lateritic, having reddish to brownish colour. Engineering geological tests such as natural moisture content, particle size analysis, consistency limits, California bearing ratio and consolidation were carried out on the soils following standard procedures revealed that the values of natural moisture content do not generally follow a consistent pattern and varied from location to location. The grain size characteristics curve, displayed 84% and 16% subsoils are of well graded and poorly graded type respectively. The soils were grouped into CL (low plasticity), CI (medium plasticity) and CH (high plasticity) from consistency limits results. Compaction characteristics of the subsoils revealed 36% and 64% representative of fair to good and poor to very poor foundation materials respectively. Soils with settlement rates greater than 1mm/year were designated as high settlement subsoils. 72% and 28% of the subsoils fell into hard to stiff and soft categories from the shear strength characteristics respectively, and classified as c- ϕ soils. California Bearing Ratios values range from 10–70, indicating their suitability for pavement construction. Conclusively, areas underlain by migmatite-gneiss and charnockite-derived soils, and granite and quartzite-derived soils possessed low and high strength characteristics respectively which can be attributed to their textural characteristics. The subsoils of the entire campus spread are however capable of bearing very substantial loads.

1. Introduction

Engineering geological mapping has been adjudged as a veritable approach employed in the development and planning of several civil and mining operations and its usages had risen in contemporary time [1]. This approach also helped in the planning of several countries such as United States, Slovakia, Tyne and Wear County England, United Kingdom, Nigeria (Niger Delta Basin, Abuja (Federal Capital City) and Benin City) and Northern Mekelle town, Tigray, Northern and Southern parts of Ethiopia [2]-[13]. However, all engineering geological mapping always result to the production of an engineering geological map with variations in scale. This map always contains the information about the engineering geological features of the area investigated/mapped in respect to the purpose of investigation. The obtained information often played advisory, preventory and planning roles for safe civil engineering works [7]. However, the information on the resulting engineering geological map must include engineering properties (be it physical, mechanical, index and strength) of engineering soils (residual and transported soils) and rocks, water conditions, geomorphological and geodynamical processes [7]. Typical engineering geological maps usually show all these features which were then subjected to classification and assessment by employing different relative weights such as severity, performance or recurrence, and difficult of amelioration [3]. It has been emphasized, that engineering geological maps cannot be

used for selection of specific engineering sites, but are useful for preliminary planning of several civil and mining works [3]. Engineering geological map produced for Slovakia Tyne and Wear County, England was entirely a geological map with additional information on the hydrogeological situation, contemporary geodynamic processes, and thus four main engineering geological regions of the areas were classified [4]. This approach however was employed on a university campus spread (FUTA) which has not been reported in any literature but several authors [13]-[20] have also worked on the foundation studies of specific civil engineering structures in different higher institutions of learning.

Several problems such as differential settlement usually resulting from unfavorable engineering geological conditions have confirmed that engineering geological mapping is a fundamental prerequisite for planning [21]. Many engineering activities, foundations for buildings, roads, bridges, etc., are in most cases limited within the top few metres of the surface of the earth. Therefore, surficial material deposits such as engineering soils (residual soils) are of prime importance in most engineering geological and geotechnical investigations [7, 22].

The current research work took into consideration the present expansion (e.g. increasing population, lecture theatres, office blocks, school buildings and hostels) and futuristic growth of the Federal University of Technology, Akure campus based on its goal and objectives. This was achieved by generalized geological mapping and performing of index and strength tests on the abundant residual soils to appraise the soils in relation to planning and development of the campus. The results are meant to aid proper planning and development of the university campus by characterization of engineering properties of residual soils.

Study area

The study area covers about 6.4 km², lies between Latitudes 7°17'0''N – 7°19'0''N and Longitudes 5°7'0''E – 5°9'0''E, falls under humid tropical climate of the West African Monsoonal type with distinct wet and dry seasons [23].

Annual rainfall reaches mean value of about 1350mm coupled 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% to 90% (January to July) [23].

Vegetation in the study area is however of regrowth rainforest type with evident spread of grasses and scattered trees but had in many parts been modified with human activities such as civil engineering structures such as buildings, roads and land cultivation needed for the growth of the varsity.

The study area is a fast-growing Federal University of Technology which at present had only built up 45% of her land area [23]. Due to the rapid development on the campus following FUTA Campus Master Plan 2013-2022, there has been increasing number of on-going building projects.

Topography, geological setting and drainage conditions of the study area

The topography is undulating with the presence of depressions at the Southern part and fairly level plain at the Northern part of the study area. There are some ridges with their trends in the generally north-south direction. In some places, all the basement rocks except the charnockites occur as inselbergs which stand out against relatively low-lying surroundings (Fig. 1) [23]. The study area is well-drained and the streams draining the area flow southward. The stream directions are largely controlled by the trend of the rocks (Fig. 1). Most of the stream channels shows a meandering pattern while some flow in a straight pattern of about 300m which can be suggested as fracture-controlled drainage system. The drainage is of medium density which is as a result of influence of climate, infiltration capacity and the vegetation cover.

Figure 1. Topographical Map of the Federal University of Technology, Akure (FUTA) (Inset at top is map of Nigeria) [23].

2. Materials and Methods

Reconnaissance survey, field work, and generalized geological mapping of the study area were carried out. A total of twenty-five dug pits from which a total of fifty (50) soil samples comprises of twenty-five (25) disturbed and twenty-five (25) undisturbed soil samples were collected at depth of about 1m in each of the pits.

The laboratory tests include determination of moisture content, grain size distribution, Atterberg limits (liquid and plastic limits), California bearing ratios, triaxial shear strength, consolidation and compaction were all undertaken at the Engineering Laboratory section an arm of the Department of Applied Geology, the Federal University of Technology, Akure (FUTA) Nigeria. The soil samples were tested according to the standard procedures [24]. Microsoft EXCEL, SURFER and GRAPHER softwares were used for statistical analysis and engineering geological map production respectively. The soils were classified according to American Association of State Highways and Officials (AASHTO) and United Soil Classification System (USCS).

The Terzaghi's ultimate soil-bearing capacity equation was applied in the determination of ultimate bearing capacity, allowable bearing pressure and allowable column load for square footing is as follows

$$q_{ult} = 1.3cN_c + qN_q + 0.4\gamma BN_\gamma \quad \text{equation 1}$$

$$q_{all} = q_{ult}/F_s \quad \text{equation 2}$$

where B= diameter of the footing, N_c , N_q and N_γ are Terzaghi's Bearing Capacity Factors, q_{ult} = ultimate bearing capacity, q_{all} = allowable bearing pressure and F_s = factor of safety. The Terzaghi's Bearing Capacity Factors were obtained from the obtained angle of internal friction and cohesion

reading in Terzaghi's Table. Also, c and q are cohesion and angle of internal friction values respectively. Factor of safety used was 2.5, depth of groundwater was average of 4m. With square footing dimensions, width, length and depth to foundation of 2m by 2m by 2m dimensions respectively.

3. Results and Interpretation

Summary of results of different engineering geological tests are presented in Tables 1-5 and Figs. 2-9.

3.1 Generalized Geological Mapping of FUTA

The rock types in the study area and from which the residual soils were developed are migmatite-gneisses, granites, charnockites, and quartzites of which they all conforms to the rocks that are associated with the Basement Complex of Nigeria (Fig. 2). The rocks were however not classified towards any civil engineering purposes.

3.1.1 Granites

The coarse grained granites cover about one-third of the study area to the south (Fig. 2). They are the dominant acid intrusives in the study area intruding migmatite-gneisses. These rocks have also undergone some degree of weathering.

3.1.2 Migmatite gneisses

The migmatite-gneisses cover the major part of the study area (Fig. 2) with deep contacts with the granites in the area. Their trends are usually north-south with feldspars aligned in the same direction with the other minerals. The migmatite-gneisses varied from medium to coarse grained, light grey to pinkish rock. Observed textures in these rocks are marked by mineralogical banding of the felsic and mafic minerals. They have foliation trending 138° and 152° , also the rock is characterized with normal faults in some locations. Structures observed on the rock were shearing and folding.

3.1.3 Quartzites

These rocks only occur as a ridge, which covers smaller part of the study area (Fig. 2). They are massive in nature, coarse grained and with rubbles of quartz.

3.1.4 Charnockites

These are coarse-grained rocks of approximately granitic composition and granoblastic texture. They occur as discrete bodies, boulders and low lying outcrop and found along road cut and they occupied 10% of the study area (Fig. 2).

Figure 2. Generalized Geological map of the study area.

3.2 Description of the Soils

The sub-surface soils in the study area are composed largely of residual soils which are weathering products of the basement rocks. The soils are reddish to brownish in color due to presence of iron oxide, having medium to coarse-grained mineral matters with some clayey materials. They are characterized by mottled and sticky features in some locations. The soils of the study area resemble laterites in nature.

3.2.1 Engineering Geological Index Characters of the FUTA soils

The values of natural moisture content does not follow a general consistent pattern (Table 1) and they varied from location to location which may be due to nearness to the ground surface, prevailing rainfall condition and local drainage conditions [25]. High variation in the moisture contents may cause large volume changes in clayey soils and this can be used to trace the clay content and type in the soil [26]. The results when compared with proposition in [27] of natural moisture content values, twenty-three soils (92%) fall within suitable to marginable engineering materials and two soils (8%) fall within unsuitable engineering materials.

From the grain size characteristics curve plotted for all the soils in the study area, 84% of the subsoils are of well graded type which indicated they consisted of particles that will form a good framework for stress transfer (Fig. 3). They will be relatively stiff and strong, because the smaller particles fill fitly in the pores between the larger particles. They will definitely possess higher shear strengths, densities, reduced compressibilities, stabilities, bearing capacities and low permeabilities. This will also contribute to their suitability for creating road foundations and production of concretes. The remaining 16% subsoils are of poorly graded type which will possess lower shear strengths, densities, high compressibilities, low stabilities, lower bearing capacities and high permeabilities. This was also compared with the results obtained from the consolidation but two of the poorly graded soils confirmed this. The gradation textures also gives insight to the rate of leaching and weathering which can be related to the textural characteristics of the parent rock types. Well graded soils suggest decreasing degree of leaching and weathering and poorly graded soil suggest increasing degree of leaching and weathering.

Table 1. Summary of engineering geological tests results on FUTA soils.

Samples	NMC (%)	Sand (%)	Gravel (%)	Coarse (%)	Fine (%) passing 75 μm	LL (%)	PL (%)	PI (%)	MDD (kg/m^3)	OMC (%)
1	17	23.6	39.4	62	38	42	24.1	17.9	1753	15.21
2	19.5	29.2	20.4	49.6	50.4	49.5	33.3	16.2	1645	20
3	16	34.8	20.8	55.6	44.4	40.8	20.2	20.6	1766	13.95
4	8	39.2	36.5	75.7	24.3	33.2	18.5	14.7	1977	7.71
5	22.5	29.1	6.4	35.5	64.5	54.8	25.1	29.7	1629	19.99
6	11	31.9	36.5	68.5	31.5	60.2	22.7	37.5	1805	14.3
7	13.5	51.6	3	54.6	45.4	29.2	15.5	13.7	1914	10.45
8	17.5	28.6	45.1	73.7	26.3	50.6	26.2	24.4	1871	13.83
9	18.5	23.2	23.2	45.4	54.6	53.8	27.7	26.1	1652	20.15
10	26	11.2	1.9	13.1	86.9	65.8	40.7	25.1	1403	26.58
11	12	32.1	26.5	58.9	41.1	47.4	41.6	5.8	1767	14.34
12	12.5	30.3	38.9	69.2	30.8	37.8	21.5	16.3	1782	15.7
13	16	26.1	42.2	68.3	31.7	51.6	23.5	28.1	1759	15.69
14	13	14.9	57.7	71.6	28.4	54.18	29.6	24.5 8	1760	17.6
15	19.5	30.1	3.8	33.9	66.1	47.4	25.2	22.2	1637	19.63
16	19	27.5	12.3	39.8	60.2	57.8	30.2	27.6	1619	21.32
17	19	35.2	13.8	49	51	47.4	28.2	19.2	1662	18.19
18	18	29.1	20.2	49.3	50.7	58.2	29.6	28.6	1661	19.51
19	16	40.9	2.5	43.4	56.6	51.6	23.4	28.2	1688	18.22
20	15	45.8	0.2	46	54	42	23.8	18.2	1662	18.65
21	14	39.3	16.1	55.4	44.6	47.3	22.3	25	1601	19.13
22	13	30.6	36.8	67.4	32.6	41.9	18.6	23.3	1884	15
23	18	37.1	26.4	63.5	36.5	44.1	27.3	16.8	1647	20.16
24	9.5	30	40.1	70.1	29.9	44.1	12.9	31.2	1951	10
25	15.5	36.6	2.8	39.4	60.6	32.3	17.1	15.2	1742	15.8

Where NMC= Natural Moisture Content, LL = Liquid limit (%), PL= Plastic limit (%), PI= Plasticity Index (%), MDD = Maximum Dry Density (kg/m^3), OMC = Optimum Moisture Content (%).

GRAIN SIZE ANALYSIS

Client:

Date: 6/3/2015

Project:

Location:

Borehole No.

Sample No.

1

Depth, mt.:

Sieve Analysis

Particle Description		Particle Diameter (mm)	Mass Retained (g)	Retained (%)	Percent Passing (%)
	Cobbles	100.000			100.0
		75.000			100.0
GRAVEL	Coarse	63.000			100.0
		37.500			100.0
		19.000			100.0
		14.000			100.0
		9.500			100.0
	Fine	4.750	117.80	24.27	75.7
		2.360	73.60	15.17	60.6
SAND	Coarse	1.180	37.00	7.62	52.9
	Medium	0.850	13.80	2.84	50.1
		0.425	28.80	5.93	44.2
		0.300	12.40	2.56	41.6
	Fine	0.150	21.60	4.45	37.2
		0.075	0.80	0.16	37.0
		0.063	0.00	0.00	37.0
FINES	Clay or Silt	<0.063	179.50	36.99	
		Sum	305.80		
		Initial wat	485.30		

Gravel = 39.4% Gravel = 39.4%
 Coarse Sand = 7.6%
 Medium Sand = 11.3% Sand = 23.6%
 Fine Sand = 4.6%
 Fines = 37.0% Fines = 37.0%

Moisture Content % = 3.0 %

Bulk weight = 500.0 g

Dry weight = 485.3 g

Figure 3. A typical grain size characteristic curve (Sample 1).

Low amount of fines (less than 35%) and high amount of coarse contents (greater than 65%) suggest low amount of micas and feldspars which was also observed on the granite outcrop and quartzite ridge. Soils with high amount of fines (greater than 35%) and low amount of coarse contents (less than 65%) suggest high amount of micas and feldspars which were also observed on

the migmatite-gneisses and charnockites of the study area as shown in the spatial distribution map of the textural contents of the soils in the study area (Fig. 4).

Figure 4. Engineering geological map based on their gradation textural characteristics.

Consistency limits tests such as liquid limits, plastic limits and plasticity indices were employed for classification purposes and predictions of its usages for civil engineering purposes. Applying [28] recommendations of 35%, 50%, 30% and 20% maximum limits for the percentage fines, liquid limit, plastic limit and plasticity index respectively for foundation materials. Fifteen samples fall within acceptable liquid limit values and low potential swell and ten samples fall above acceptable liquid limit values and marginal potential swell. This also indicates sixty percentages (60%) and forty (40%) of the residual soil samples have low potential and marginal potential swells potentials.

Twenty-one soils, (84%) of the samples fall within acceptable plastic limit values and four soils (16%) fall above the acceptable plastic limit values. Therefore, soils with organic content have low plasticity indices corresponding to comparatively high liquid limits [29]. Fifteen samples fall within low potential swell and ten samples fall within marginal potential swell, based on [30] plastic index classification. Those samples with low plasticity index (PI) might have their expansive clays converted to less expansive types [31]. Soils of low plasticity, have larger particles, possess a fewer number of inter-particle contact points and those soils of high plasticity are composed of very small particles, having relatively high surface area per unit weight, possess a large number of particles contact points. High plastic soils (four subsoils) will also be susceptible to high compressibility, swelling on moisture influx, low bearing capacity and leading to low permeability (low drainage) and hydraulic conductivity also possessing high swell potential [26, 32, 33]. The studied soils were then plotted on the Cassagrande plasticity chart, with three, twelve and eleven soil samples falling within low, medium and high plasticity zones, respectively. Plasticity was then correlated with compressibility, based on this fact, soil samples with low, medium and high plasticities will have low, medium and high compressibility respectively. Soils with very high liquid limit tend to possess low bearing capacity while low liquid limit soils tend to possess high bearing capacities.

The studied soil samples were classified into low plasticity (CL), medium plasticity (CI) and high plasticity (CH) soils based on the following three soil characteristics; particle size distribution, liquid limit and plasticity index (Table 2; Fig. 5). Cassagrande's plasticity chart containing the A-line plays a major role in the classification. Three samples fall within low plasticity (CL), twelve

fall within medium plasticity (CI) while ten fall within high plasticity (CH) categories. According to [15], these properties of high plasticity would probably negate few favorable properties, like high MDD and relatively high shear strength parameters and ultimately render it unsuitable for use for most construction purposes especially where moisture influx cannot be effectively controlled.

Table 2. Soil Classifications of FUTA Soils.

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Location	Liquid limit, LL (%)	Plastic limit, PL (%)	Plasticity index, PI (%)	% of Soil Passing 2.36mm sieve	% of Soil Passing 425µm sieve	% of Soil Passing 75µm sieve	Group index, GI	AASHTO classification	USCS Classification code
1	42.0	24.1	17.9	60.6	44.2	37.0	2.2	A-2-7	CI
2	49.5	33.3	16.2	79.6	57.5	50.4	6.0	A-7-5	CI
3	40.8	20.2	20.6	79.2	57.7	44.4	5.0	A-7-5	CI
4	33.2	18.5	14.7	63.5	36.1	24.3	0.0	A-2-6	CL
5	54.8	25.1	29.7	93.6	75.5	64.5	17.8	A-7-6	CH
6	60.2	22.7	37.5	63.5	38.0	31.6	3.5	A-2-7	CH
7	29.2	15.5	13.7	97.0	68.4	45.5	2.7	A-2-6	CL
8	50.6	26.2	24.4	54.9	31.0	26.3	0.0	A-2-7	CH
9	53.8	27.7	26.1	76.8	60.8	53.6	11.2	A-7-6	CH
10	65.8	40.7	25.1	98.1	91.0	86.9	20.0	A-7-5	CH
11	47.4	41.6	5.8	73.5	51.5	41.4	0.4	A-7-6	CI
12	37.8	21.5	16.3	61.1	39.9	30.8	0.2	A-2-6	CI
13	51.6	23.5	28.1	57.8	37.7	31.7	2.2	A-2-7	CH
14	54.2	29.6	24.6	42.3	30.4	27.4	0.0	A-2-7	CH
15	47.4	25.2	22.2	96.2	78.6	66.1	13.6	A-7-6	CI
16	57.8	30.2	27.6	87.7	67.9	60.2	15.2	A-7-5	CH
17	47.4	28.2	19.2	86.2	61.9	50.9	7.1	A-7-6	CI
18	58.2	29.6	28.6	79.8	57.7	50.7	11.2	A-7-6	CH
19	51.6	23.4	28.2	97.5	74.4	56.6	13.1	A-7-6	CH
20	42.0	23.8	18.2	99.8	75.8	54.0	7.2	A-7-6	CI
21	47.3	22.3	25.0	83.9	57.6	44.6	6.7	A-7-6	CI
22	41.9	18.6	23.3	63.2	44.1	32.6	1.8	A-2-7	CI
23	44.1	27.3	16.8	73.6	48.0	36.6	1.8	A-2-7	CI
24	44.1	12.9	31.2	59.9	42.2	29.8	2.0	A-2-7	CI
25	32.3	17.1	15.2	97.2	78.2	60.6	6.5	A-7-6	CI

Where AASHTO= American Association of State Highways and Officials, USCS= United Soil and Classification System.

Figure 5. Plot of tested samples on the USCS classification chart.

The result of AASHTO classification shows that ten of the tested soils classify as A-7-6 soils, nine classify as A-2-7 soils, while three classify as A-7-5 soils, and the remaining three classify as A-2-6 soils. This classification denotes two type of soils namely granular soils (A-2-6 and A-2-7) and clayey soils (A-7-5 and A-7-6). The granular soils are considered suitable for road construction materials while the clayey soils are considered unsuitable for road construction materials. It follows that the suitability of the soils as road construction materials is low, and this worsens southward. They can only be successfully used for base and sub-base courses after stabilization.

3.2.2 Engineering Geological Strength Characters of the FUTA soils

Theoretically and practically from compaction test, the best soils are those that have high maximum dry density values at relatively low optimum moisture content [34]. Based on [35] proposition, two out of the tested soils fall within the good foundation materials category, seven soils fall within the fair foundation materials category, fourteen soils fall within poor foundation materials category and two soils fall within the very poor foundation materials category (Table 1; Figs. 6 and 7). Low maximum dry density value of subsoils is a prediction to the future occurrence of construction failures in the study area.

Figure 6. A typical compaction curve (Sample 1).

Figure 7. Engineering geological map based on their compaction characteristics.

It was also observed that all the soils with coarse contents greater than 65% are characterized with averagely higher MDD values which fall within fair to good engineering materials according to [35] classification. Evidence of this was traced to the low degree of weathering of the parent rocks. Those soils with fine contents greater than 35% were characterized with lower MDD values which fall within poor to very poor engineering materials according to [35] classification. They are believed to have resulted from the higher degree of weathering of the parent rocks.

The one-dimensional consolidation test was adopted for this study in the determination and calculation of the magnitude and rate of settlement of the soils in the study area. The load-settlement data obtained from the full cycle of loading and unloading are used to draw a pressure-voids ratio curve from which the coefficient of volume compression (m_v) is derived. This was used to calculate the magnitude of consolidation settlement under any given loading.

The rate of settlement ranges from 0.596 to 1.614 mm/yr, the pre-consolidation pressure obtained using the Cassagrande method (void ratio-pressure plot) ranges from 0.0236 to 0.0589 and the coefficient of volume compressibility (m_v) ranges from 0.190 to 0.458 (Table 3).

Soils with settlement rates greater than 1mm/year portend high settlement and these high values are not related to shallow water table because the water table is at deep depth and can therefore not contribute to any settlement of the subsoil. The cause of the high settlement values are possibly related to the deep weathering of locally enriched zones of feldspars in the original parent rocks [20]. The settlement rates in most places in the study area are believed to be influenced by genetic similarities between them as this almost followed the route of compaction results with the underlying geology. It has been established that the values of coefficient of consolidation varies with the presence of pore spaces in the soils. Soils with lower coefficient of consolidation values are suggestive of soils of clayey type and low permeability while those with higher coefficient of consolidation values are suggestive of granular (sandy clayey) soils characteristics of high permeability.

Applying USCS classification, seven samples fall within soft category (where angle of internal friction is less than 20°), seventeen fall within the hard category (where angle of internal friction is between 20 - 35°) and only one sample falls within the stiff category (where angle of

internal friction is greater than 35°). Soils that fall within the hard and stiff categories suggest that they will serve as subgrade materials for road construction. These soils would have considerable strength, could withstand shear stresses, and be of high bearing capacity during the engineering life of civil engineering projects on the university campus. These types of soils suggest the absence of minerals that can bring about drastic change in their properties. Those soils that fall within soft category cannot serve as subgrade materials. This will be of unsuitable construction materials with lesser strength, and are not likely to withstand shear stresses. They will also be of low bearing capacities during the engineering life of civil engineering projects on the university campus.

Table 3. Summary of the results from Consolidation, Shear strength and California Bearing Ratios Tests.

Samples	σ_p	m_v	S (mm/yr)	Internal Friction Angle	Cohesion (o)	Shear Strength (Kpa)	CBR (%)
1	0.024	0.458	0.6	22.1	10.1	38.4915	44%
2	0.043	0.443	1.05	32.9	10.6	123.468	10%
3	0.042	0.52	1.19	14.9	30.5	29.6212	22%
4	0.05	0.272	0.68	26.2	24.4	81.3164	70%
5	0.047	0.404	0.97	16.8	26.6	35.0394	18%
6	0.041	0.433	0.73	18.8	99.3	103.534	25%
7	0.051	0.409	1.11	18.2	19.2	36.1677	26%
8	0.058	0.19	0.59	16.7	79.7	73.4509	65%
9	0.053	0.356	1.21	29.9	77.9	199.552	14%
10	0.059	0.314	1.61	32.1	10	88.5895	21%
11	0.055	0.253	0.95	29.6	43.2	144.455	31%
12	0.051	0.357	0.96	25.9	35	94.9891	38%
13	0.045	0.443	1.03	31.9	46.4	162.022	52%
14	0.053	0.264	0.81	22.3	120.2	159.282	37%
15	0.05	0.407	1.31	12.5	22.2	19.672	20%
16	0.054	0.307	1.16	15.9	32.6	36.3279	29%
17	0.051	0.255	0.93	18.2	19.2	36.1677	24%
18	0.042	0.24	0.66	29.9	14.9	89.2678	43%
19	0.053	0.249	0.99	10.6	46.1	23.5944	30%
20	0.026	0.39	0.64	21.4	24	43.8666	14%
21	0.051	0.341	1.08	22.4	31.5	114.803	27%
22	0.047	0.132	0.45	36.4	5.7	169.502	22%
23	0.056	0.222	0.95	25	62.4	119.777	44%
24	0.058	0.174	0.96	31.5	66.7	202.495	25%
25	0.026	0.41	0.65	20.5	9.3	34.0901	22%

σ_p = pre-consolidation pressure, m_v = coefficient of volume compressibility, S= Settlement (%)

Spatial distribution maps were drawn for the soils based on the obtained properties and these was used to classify the soils into three, namely low, medium and high shear strengths characters (Fig. 8). The shear strength characters displayed similarities in the presence of the parent rocks with migmatite-gneiss and charnockites contributing to low to medium shear strengths characters while granites and quartzites contributing to high shear strength characters.

Figure 8. Engineering geological map based on their undrained shear strength characteristics.

California Bearing Ratios which has been adjudged as an indication of soils strength for the proper evaluation of highway, road and footpath subgrade and subbase materials [36]-[38]. CBR has also been correlated with pavement performance and applied in the establishment of design curves for pavement thickness [26, 39, 40]. [41], however recommends a California Bearing Ratio of greater than 10% for subgrade materials. The subsoils usages based on the California Bearing Ratio result, could be subdivided into three foundation purposes namely Base, Sub-grade and Sub-base usages. Two soils in the study area have the capacity to serve as base materials, fifteen soils have the capacity to be employed as subgrade materials and the remaining eight soils have the capacity to be used as sub-base materials (Table 4).

Table 4. Summary of unsoaked California Bearing Ratio results.

Location	unsoaked CBR	General Rating	Use
1	44%	Good	sub-base
2	10%	Good	subgrade
3	22%	Excellent	subgrade
4	70%	Good	base
5	18%	Good	subgrade
6	25%	Excellent	subgrade
7	26%	Excellent	subgrade
8	65%	Good	base
9	14%	Good	subgrade
10	21%	Excellent	subgrade
11	31%	Good	sub-base
12	38%	Good	sub-base
13	52%	Good	sub-base
14	37%	Good	sub-base

15	20%	Excellent	subgrade
16	29%	Excellent	subgrade
17	24%	Excellent	subgrade
18	43%	Good	sub-base
19	30%	Good	sub-base
20	14%	Good	subgrade
21	27%	Excellent	subgrade
22	22%	Excellent	subgrade
23	44%	Good	sub-base
24	25%	Excellent	subgrade
25	22%	Excellent	subgrade

All the studied soils, on the basis of their CBR values will not pose any failure of highway pavements, roads and footpaths (walkways) in the study area (Fig. 9). Seven subsoil samples located close to the drainage systems in the campus were discovered to still possess high coarse contents and low settlements rates except soil sample 7 with high settlements, it can be then deduced that drainage does not directly influence the settlement rates of soils but the underlying geology influenced the engineering properties of soils in the study area.

Figure 9. Engineering geological map based on their highway characters.

3.2.3 Undisturbed Tests

Moisture content values ranged from 7.09 to 17.85% and bulk density values varied from 16.41 to 23.94KN/m³ (Table 5). From Terzaghi bearing capacity formula, the ultimate bearing capacity ranges from 464 to 5214KPa, allowable bearing pressures ranges from 185 to 2085KPa and allowable column loads range from 553 to 8153KN. The variation in the bearing capacities and allowable bearing pressures is traceable to the different kinds of soils in FUTA as observed from the AASHTO classification, depth of soil collection, loading form and type [42]. These also confirm that the soils ranged from fairly competent to competent in bearing very substantial loads.

Table 5. Summary of results from undisturbed soil tests.

Samples	A	B	C	D	E	F,	G	H	I	J
1	7.09	1.81	1.69	0.54	0.35	7.09	17.74	682	273	903
2	15.49	1.81	1.57	0.66	0.40	15.49	17.79	2255	902	3419
3	13.19	1.91	1.69	0.52	0.34	13.19	18.74	704	282	937
4	7.60	1.87	1.74	0.52	0.34	7.60	18.34	1567	627	2318
5	17.05	1.80	1.54	0.68	0.41	17.05	17.62	725	290	971
6	11.17	1.90	1.71	0.52	0.34	11.17	18.63	2410	964	3667
7	10.42	2.24	2.03	0.27	0.21	10.42	21.94	723	289	968
8	7.83	2.29	2.12	0.24	0.19	7.83	22.43	1767	707	2638
9	16.01	1.98	1.71	0.51	0.34	16.01	19.43	4902	1961	7655
10	14.39	1.89	1.65	0.52	0.34	14.39	18.51	2068	827	3121
11	11.71	1.71	1.53	0.68	0.40	11.71	16.73	2989	1195	4593
12	10.63	1.85	1.68	0.59	0.37	10.63	18.19	1888	755	2813
13	13.81	1.85	1.62	0.58	0.37	13.81	18.12	4054	1622	6298
14	9.70	2.44	2.23	0.22	0.18	9.70	23.94	3807	1523	5902
15	13.13	1.81	1.60	0.60	0.38	13.13	17.72	464	185	553
16	15.12	1.93	1.67	0.57	0.36	15.12	18.89	797	319	1087
17	17.85	1.90	1.61	0.62	0.38	17.85	18.65	673	269	888
18	10.48	1.87	1.69	0.52	0.34	10.48	18.35	1819	728	2721
19	17.54	1.85	1.58	0.70	0.41	17.54	18.16	716	286	956
20	14.49	1.75	1.52	0.67	0.40	14.49	17.12	976	390	1373
21	16.04	1.78	1.54	0.68	0.41	16.04	17.49	1276	511	1853
22	10.43	2.00	1.81	0.43	0.30	10.43	19.60	3393	1357	5241
23	14.06	1.67	1.47	0.83	0.45	14.06	16.41	2577	1031	3934
24	8.91	2.14	1.96	0.35	0.26	8.91	20.95	5214	2085	8153
25	11.78	2.03	1.82	0.42	0.30	11.78	19.93	609	244	786

Where A= moisture content (%), B=bulk density (KN/m³), C=dry density (KN/m³) D=void ratio, E=porosity, F=water content(%), G= bulk unit weight, H=ultimate bearing capacity (q_u) (KPa), I=allowable bearing pressure (q_{all}) (KPa) and J= allowable column load (P) (KN).

Conclusion

Residual soils derived from granitic and quartzitic rocks possess the highest strength while those derived from migmatite-gneissic and charnockitic rocks possess lesser strength for foundation and construction purposes. Results of compaction, triaxial shear strength, and consolidation and California Bearing Ratio tests also confirmed that subsoils are generally made of cohesive-frictional particles and have appreciable amount of cohesive (clay) binders to resist substantial shear force. The plasticity of the soils also qualifies them to be suitable for clay bricks and certain ceramics since larger percentage of the soils fall above the A-line on the Cassagrande plasticity chart, indicating that they are essentially inorganic clays.

Spatial distribution maps of the soils drawn for particle size distribution, plasticity, compaction, California bearing ratio, and consolidation data then revealed the influences of genetic similarities on the properties of the residual soils. This gave a generalized grouping of the soils. The degree of weathering is also believed to influence the amount of coarse and fine contents. High degree of weathering is responsible for the high fine contents resulting from enrichment of feldspar which weather into clay. Low degree of weathering is perceived to have influenced the high amount of coarse contents which reflect the depletion in enrichment of feldspar and enrichment of resistant quartz minerals in granites and quartzite ridges. Residual soils derived from migmatite-gneissic and

charnockitic rocks possess higher amount of fines which can be attributed to high degree of weathering. This also resulted in the low strengths observed from their low MDD values, internal frictional angles and high settlement rates. On the other hand, subsoils derived from granites and quartzites possess high amount of coarse particles which can be attributed to low degree of weathering. This also resulted in the high strengths observed from their high MDD values, shear strengths, internal frictional angles, CBR and low settlement rates. Seven of the eight samples taken close to the drainage systems in the study area were discovered to possess high coarse contents and low settlements rates. It can then be deduced that drainage alone does not directly influence the settlement rates of soils, rather the interaction between the underlying geology and the drainage. These also confirm that the soils ranged from fairly competent to competent in bearing very substantial loads at shallow depth.

Recommendation

The few soils that show high plasticity (expansive) in the study area can be stabilized mechanically and chemically to increase its strength and enhance its uses as foundation and construction materials.

Conflict of Interest Statement

The authors declare that there is no conflict of interest in this manuscript.

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