Engineering Heritage Journal (GWK)

DOI: http://doi.org/10.26480/gwk.02.2021.58.64

ZIBELINE INTERNATIONAL™ PUBLISHING

SSN: 2521-0904 (Print)
ISSN: 2521-0440 (Online)
CODEN: EHJNA9

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RESEARCH ARTICLE

ENGINEERING PROPERTIES AND STRENGTH EVALUATION OF SUBSOIL IN EDE NORTH, SOUTHWESTERN NIGERIA: ITS COMPETENCE FOR FOUNDATION PURPOSES

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ARTICLE DETAILS

Article History:

Received 16 November 2021
Accepted 22 December 2021
Available online 27 December 2021

ABSTRACT

This research is aimed at examining the strength properties of subsoil at Ede North, Southwestern Nigeria so as to determine its competence as foundation material. A total of 45 soil samples: 30 disturbed samples and 15 undisturbed samples were taken for different tests and analysis. These samples were subjected to laboratory tests of grain size analysis, atterberg limits for the disturbed samples while density, triaxial compression test, permeability, unconfined compression test and odometer consolidation test for the undisturbed samples. The liquid limit of the soil samples at Pit A, Pit B, Pit C, Pit D and Pit E range from 34.57% to 46.20%, 42.43%-48.02%, 40.20%-50.14%, 35.21%-46.04% and 43.04%-47.62% respectively. The plasticity indexes of the soil samples at all pit points range from 16.90%-22.70%. The specific gravity of the subsoil ranges from 2.55 to 2.65. This shows that these sampled soils are either sand or silty sand. The coarse contents of the sampled soil ranges from 33.7% to 61.2% while the fine contents ranges from 38.8% to 66.3%. Samples in pit A fall within the A-7-6 and A-6, samples in pit B and E falls within A-7-6, samples in pit C falls within A-7-6 and A-7-5 while most samples in pit D falls within A-7-6 and A-6. This implies that the soil samples are rated between fair to poor sub-grade materials. They general fall under clayey soils. The coefficient of permeability for the soils ranged from 6.45×10^{-8}cm to 1.4×10^{-9}cm which classified them as practically impermeable soils. Again, the values of the shear strength parameters are; the angle of internal friction ranged from 11.9^0 to 37.5^0, the cohesion ranges from 4.7 kPa to 84.9kPa.

KEYWORDS

Atterberg limits, Specific gravity, Strength properties, Subsoils, Foundation purpose

1. INTRODUCTION

In recent years, the incidence of structural damage accompanied by collateral losses recorded across the nation has increased tremendously. The engineering community has in response proposed some reasons why buildings, bridges and other engineering structures fail unexpectedly which include among others; poor quality of building materials, inadequate supervision during construction, and non-compliance of construction to engineering specifications. Less frequently mentioned is the subsurface conditions of the ground on which the structure are sited. The design of a structure which is safe, durable and with low maintenance costs depends upon an accurate understanding of the nature of ground on which such structure is located (Oyedele and Olayinka, 2012). Adequate site characterization has therefore become highly imperative in order to prevent loss of lives and valuables that always accompany such failure (Akintorinwa and Adelusi, 2009). Pre – foundation investigation provides information required by engineers in the design of engineering structures. The most challenging part of these investigations is to collect only those data needed with the least amount of money and in the least amount of time (Oyedele and Olayinka, 2012). All engineering structures are expectedly founded on residual soils but not all residual soils are useable for construction purposes. Foundation studies usually provide subsurface information that normally assists civil engineers in the planning, designing and the construction of civil engineering structures.

Foundations are structural elements that connect buildings, bridges and other structures to the ground (Coduto, 1990). Most new buildings require a soil investigation for the design of foundations and other soil related aspects of the construction. A soil investigation is also usually needed to satisfy the requirements of the City or Country building department. A soil investigation typically involves drilling test, borings or excavating test pits to evaluate the underlying soil, bedrock and groundwater conditions. Soil samples are obtained on the site and tested in the soil laboratory for their engineering properties such as moisture content, dry density, expansiveness, shear strength, and compressibility. The objectives of foundation investigations are to determine the stratigraphy and nature of subsurface materials and their expected behaviour under structure loadings and to permit savings in design and construction costs. The investigation is expected to reveal adverse subsurface conditions that could lead to construction difficulties, excessive maintenance, or possible failure of the structure.

The scope of investigations depends on the nature and complexity of the subsurface materials and the size, requirements for and cost of the structure (Joint department of the Army and Air Force, 1983). Improper performance of a building foundation system often causes distress in the

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building superstructure. The foundation system consists of not only the structure placed on (or in the) ground but also the ground itself. To be effective, the scope of the investigation must include a definitive analysis of both the soil, the foundation structure, the loading placed on the foundation, and events that changed the nature of the foundation system. For soil exploration, a modern and expedient approach is offered by cone penetration testing (CPT) which involves pushing an instrumented electronic penetrometer into the soil and recording multiple measurements continuously with depth (Schmertmann, 1978; Campanella and Robertson, 1988; Briaud and Miran, 1992). The cone penetrometer test is becoming increasing more popular as an in-situ test for site investigation and geotechnical design.

As a logging tool, this technique is unequalled with respect to the delineation of stratigraphy and the continuous rapid measurement of parameters like bearing capacity and friction. The addition of pore pressure measurements during cone penetrometer testing has added a new dimension to the interpretation of geotechnical parameters. The cone penetrometer test provides indices which can be correlated to soil behaviour and the interpretation of cone penetration data is made with empirical correlations to obtain required geotechnical parameters (Robertson and Campanella, 1983a). In its simplest application, the cone penetrometer offers a quick, expedient, and economical way to profile the subsurface soil layering at a particular site. No drilling, soil samples, or spoils are generated; therefore, CPT is less disruptive from an environmental standpoint.

The continuous nature of CPT readings permits clear delineations of various soil strata, their depths, thicknesses, and extent, perhaps better than conventional rotary drilling operations that use a standard drive sampler at five (5) feet vertical intervals. Therefore, if it is expected that the subsurface conditions contain critical layers or soft zones that need detection and identification, CPT can locate and highlight these particular features. In the case of piles that must bear in established lower foundation formation soils, CPT is ideal for locating the pile tip elevations for installation operations (Mayne, 2007a). This research will investigate the strength characteristics of the residual soils derived from Ede North Local Government Area of Osun State so as to establish their behavior and suitability as foundation and construction materials for any civil engineering structures.

2. Location and Accessibility

The study area is situated within Ede North Local Government Area of Osun State. It is situated on an open piece of land along Owode - Ede Road, off Osogbo - Gbongan highway. It lies within longitudes 04° 27' 43" and 04° 27' 45"E and latitudes 07° 43' 27" and 07° 43' 30"N (Figure 1). The study area covers 50m by 50m. It is sparsely vegetated though, but a virgin land. Vegetation was mainly grasses; however, a Mango tree was also on the site at the time of investigation, and topography is slightly sloppy. The climate in Osun State is tropical and is characterized by distinct wet and dry seasons. The dry season occurs from around November to March, while the wet season is from April to October with a peak period between July and August characterized by torrential rain fall. The yearly rainfall ranges from 100 to 187 cm. The daily minimum temperature ranges from 22^0C in late December to 25^0C in February, marking the intervening cold and dry harmattan period. The hottest month is March which has a daily maximum temperature of around 36^0C. The mean annual relative humidity is over 80%.

3. Geology of The Study Area

Osun state is an inherent part of the western wing of the Nigerian basement complex. Osun State is underlain by Pre-Cambrian-Cambrian (age) rocks: which comprises of migmatite, gneisses, schist and quartzite into which has been a positioning of granitic and, to a lesser extent, more basic materials (Rahaman, 1981). The prevalent rock types in this area are the gneisses, schists, granites, granodiorites, diorites and pegmatites (Figure 2).

4. Sampling and Testing Methods

Five (5) CPTs were performed at the site and another five trial (5) pits were dug with each located about 1m beside each of the five CPT point (figure 3). The pits which are about 1.2m by 2.0m were dug up to 3m depth. There are two main types of soil sample recovered from each trial pit (disturbed and undisturbed samples). The disturbed samples were taken at interval of 0.5m from each pit to the depth of 3m; hence, six (6) samples were taken from each pit. Undisturbed samples were collected at interval of 1m from each pit to the same depth: making a total of three (3) samples from each pit and fifteen (15) in all. Disturbed samples were use to determine grain size, Atterberg limits and compaction characteristics of soil. On the other hand, the undisturbed samples were used to determine the density, triaxial compression test, permeability, unconfined compression test and odometer consolidation test. A total of forty-five samples were taken in all (i.e. thirty (30) disturbed samples and fifteen (15) undisturbed samples). Theses samples were subjected to laboratory tests at the engineering soil laboratory, Civil Engineering Department of Obafemi Awolowo University, Ile-Ife. All of these tests were carried out in accordance with (British Standard, 1990; British Standard Institute, 1990).

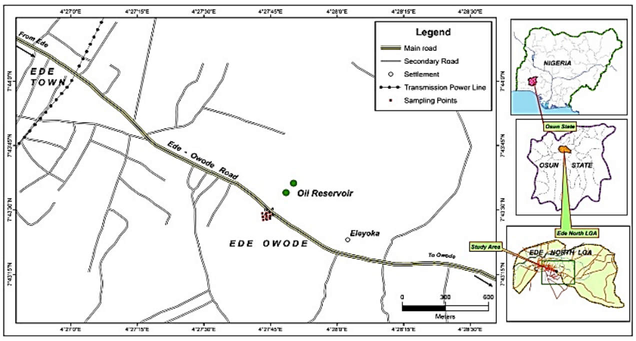

Figure 1: Location and accessibility map of the study area, inset map of Nigeria, map of Osun State, map of Ede North local Government Area showing the study area.

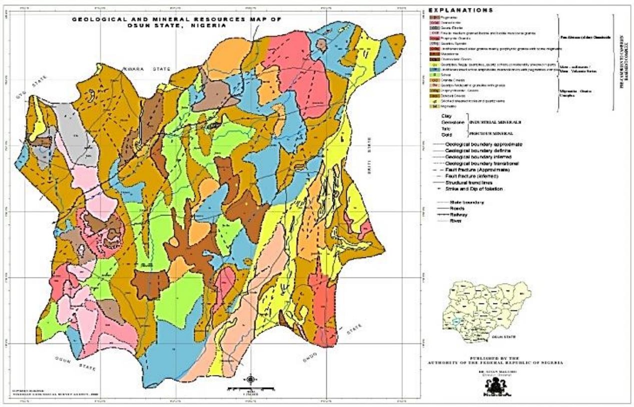

Figure 2: The geological map of Osun State, Nigeria showing the mineral resources

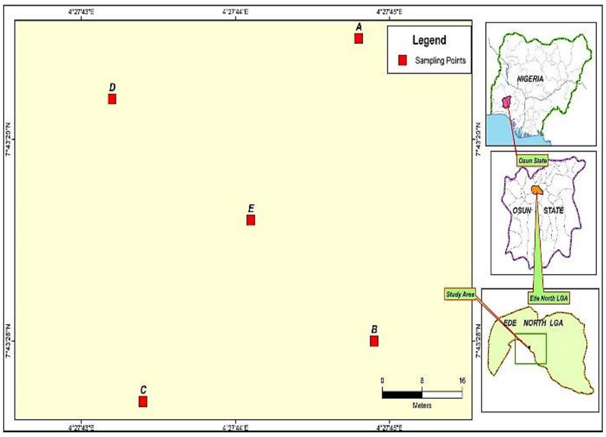

Figure 3: Layout map of the study area showing the CPT and trial pits investigated points

5. Results and Discussion

The values of natural moisture content range from 13% to 19% and this is presented in Table 1. According to proposition on natural moisture content values that 5%-15% are suitable/favorable engineering materials, 16%-19% as marginal favorable engineering materials and 20%-35% as unfavorable engineering materials (Underwood, 1679). Applying the test results with the proposition of natural moisture content values indicates that samples fall within suitable/favourable and marginal favourable engineering materials (Underwood, 1679). Consistency limits relate soils to the relative ease to which the soil can be deformed based on the interaction with water. The liquid limit of the soil samples at Pit A, Pit B, Pit C, Pit D and Pit E range between 34.57% and 46.20%, 42.43%-48.02%, 40.20%-50.14%, 35.21%-46.04% and 43.04%-47.62% respectively (table 1).

This shows that the liquid limits of the soil samples of pit A and D are lower than those of the liquid limits of soil samples in pits B, C and E. Generally, the values for all the pits fall within 34% and 50%. Again, the plasticity indexes of the soil samples at all pit points range from 16.90%-22.70%. The plasticity values are generally lower than 25%, the maximum value recommended for sub-grade tropical Africa soils (Simon et al., 1973). Also, all the soil samples at the five pit points fall under medium plasticity according to classification tables (Burmister, 1949; Sowers, 1979). According to subgrade/fill material should have liquid limit ≤50% and plasticity index ≤30% while for sub-base, liquid limit should be ≤ 30% and plasticity index ≤ 12% (Federal Ministry of Works and Housing, 1997). All the soils meet the requirement for use as subgrade/fill materials. The higher the liquid limit of a soil, the poorer the material for foundation purposes.

The Casagrande chart classification places virtually all the soil samples in the medium plasticity/ compressibility region and hence the soils would be expected to exhibit medium swelling potential (Ola, 1983). Figure 4A – 4E are the graphs of plasticity index against liquid limit for the soils from the five pits in the study area. From the graphs, almost all the soils fall within the medium plasticity region and they also fall below A-line. All the soil samples are of medium plasticity and this indicates that the soil will not swell much when it is in contact with water. The coarse contents of the sampled soil ranges from 33.7% to 61.2% and the fine contents ranges from 38.8% to 66.3% (table 1). The variation of fine and coarse contents in values with depth is presented in Table 1. The specification by requires subgrade soils to possess less than 35% amount of fines (Federal Ministry of Works and Housing, 2010). This implies that all the values fall below the of 35% maximum permissible of fine content use for a good foundation materials (Federal Ministry of Works and Housing, 2010).

These soils will fail under repeated shrinkage and swelling conditions; thereby, making all of the sampled soils unsuitable as foundation materials. Pit 3 shows a decrease in the values of fine content with depth; this is as a result of decrease in weathering activities with depth. On the other hand, the other four pits gave fluctuating (decreasing and increasing) values with depth. The decrease and increase of fine contents with depth is possibly because the studied area is affected by a high level of weathering and topography of the area (some of these areas are covered with thick vegetations while others are barred). Figures 5a – 5g show the grading envelope of the studied soils in Pit A, B, C, D, and E respectively. The soils are well graded sandy, silty, clay with some gravel. Using classification, Soils that are classified under groups A-1, A-2, and A-3 are granular materials of which 35% or less of the particles pass through the No. 200 sieve (AASHTO, 1993). Soils that have more than 35% pass through the No. 200 sieve are classified under groups A-4, A-5, A-6, and A-7.

These soils are mostly silt and clay-type materials. Samples in pit A fall within the A-7-6 and A-6, samples in pit B and E falls within A-7-6, samples in pit C falls within A-7-6 and A-7-5 while most samples in pit D falls within A-7-6 and A-6 (table 1). This implies that the soil samples are rated between fair to poor sub-grade materials. Therefore, they fall under clayey soils. A group researchers recommended amount of fines of at least 20% for landfill seals i.e. for soil that can be good for base of landfill (Daniel, 1993). Therefore, the studied soils meet this standard specification and can be used for base of landfill. The average uniformity coefficient of soil samples at Pit A are 50, 929,166, while that of the pit B ranges from 52-915, pit C ranges from 381-1843, pit D ranges from 170-922 and pit E ranges from 63-177(table 2). These values are greater than one, which is indicative of a well graded soil and hence a good highway sub-base and sub-grade materials. Some of the residual lateritic soil samples have no uniformity coefficient because of the absence of D_{10}.

Specific gravity of soil particles is a very good primary index property to characterize lateritic soils as it reflects their iron and sesquioxide content and the extent of lateritization (Tuncer and Lohnes, 1977). The specific gravity of the subsoil range from 2.55 to 2.65 (table 1). The specific gravity of the test soils did not change appreciably with depth but there is a little variation which may have been as a result of low degree of weathering at those points as compared to others. Some researchers gave values of 2.50 to 3.60 for specific gravity of lateritic soils Thus, the specific gravity value of the derived soil samples fall within the stipulated range (Alexander and Cady, 1962). Correlation of the results with of 2.60 to 2.70 for sand and silty sand, it shows that these sampled soils are either sand or silty sand (Bowles, 2012).

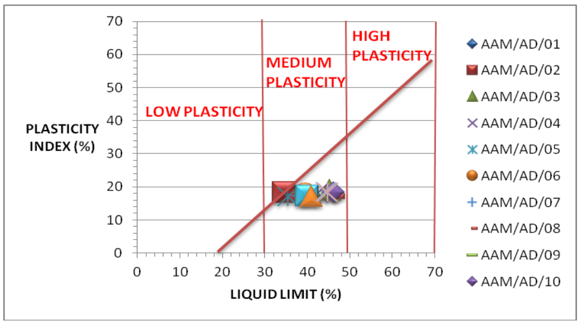

Figure 4A: Casagrande chart for pit A

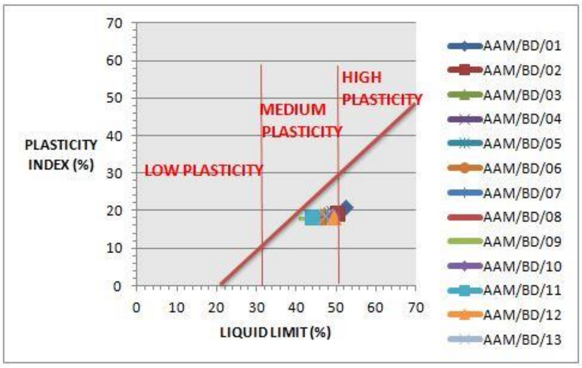

Figure 4B: Casagrande chart for pit B

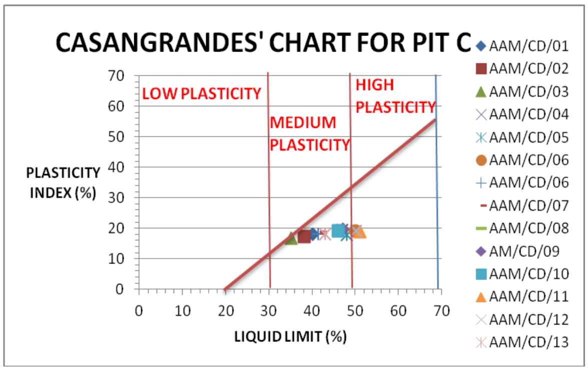

Figure 4C: Casagrande chart for pit C

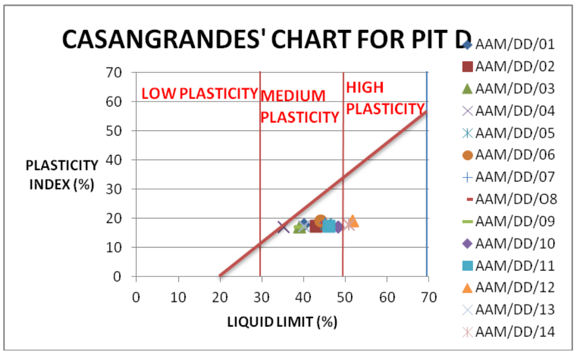

Figure 4D: Casagrande chart for pit D

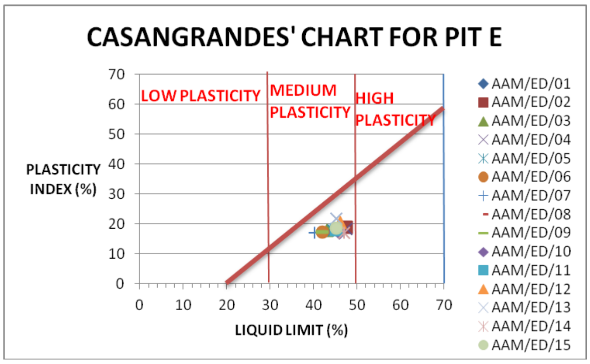

Figure 4E: Casagrande chart for pit E

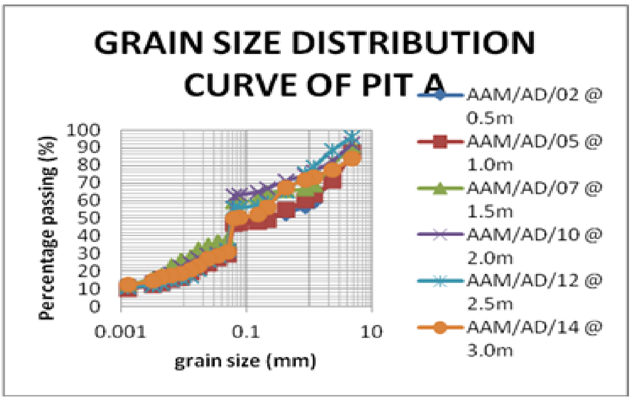

Figure 5A: Grain size distribution curve for Pit A samples

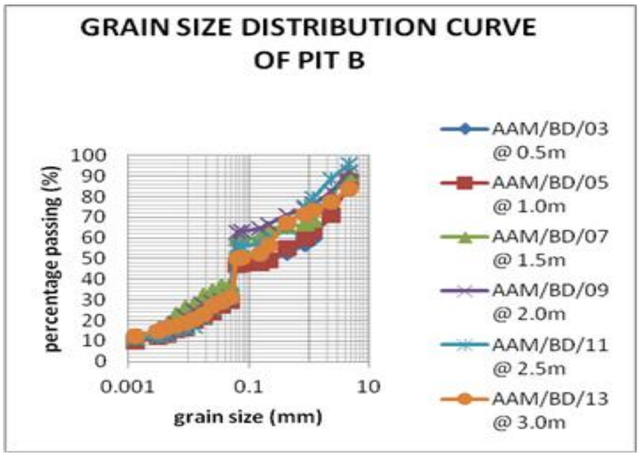

Figure 5B: Grain size distribution curve for Pit B samples

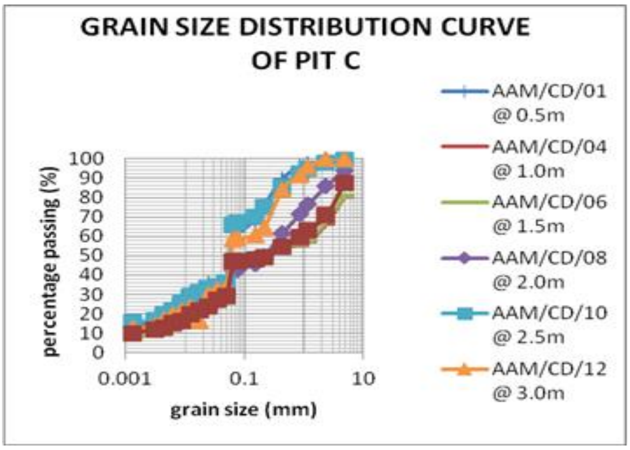

Figure 5C: Grain size distribution curve for Pit C samples

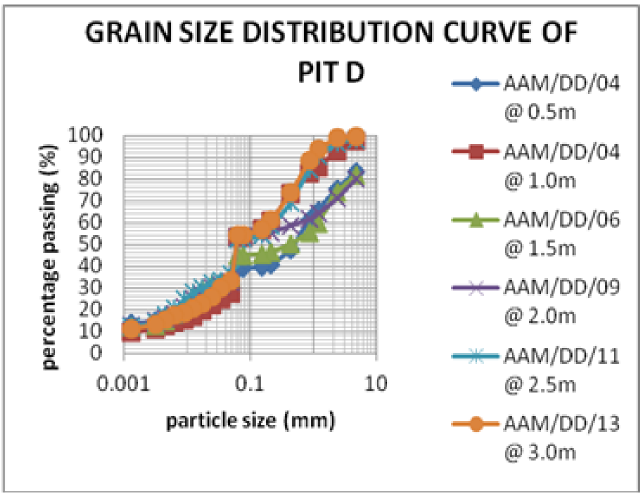

Figure 5D: Grain size distribution curve for Pit D samples

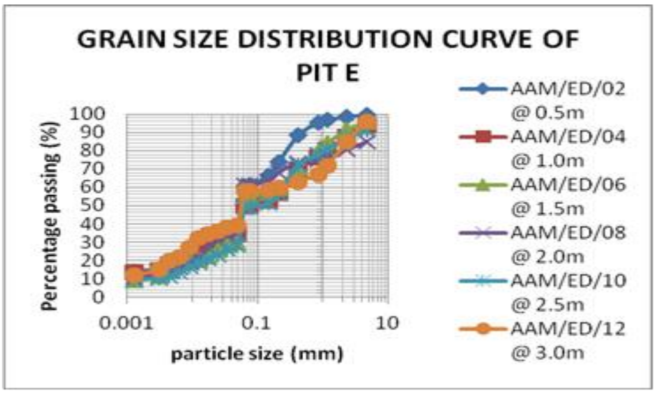

Figure 5E: Grain size distribution curve for Pit E samples

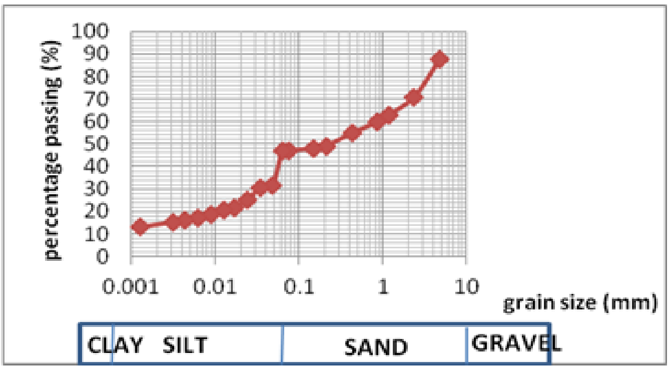

Figure 5F: Grain size distribution curve for Pit A @ 1.0m

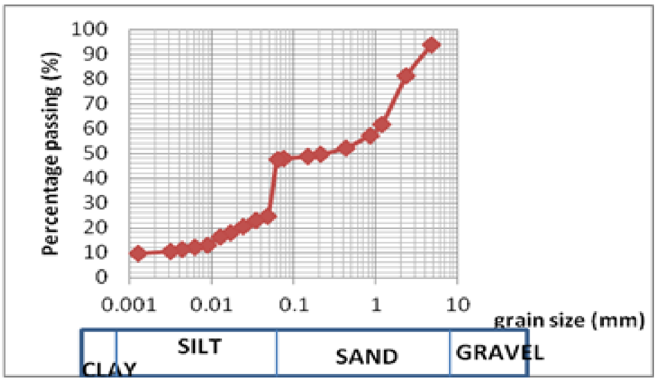

Figure 5G: Grain size distribution curve for Pit A @ 2.5m

NMC natural moisture content, SG specific gravity, LL liquid limit, PL plasticity limit, PI plasticity index, LS linear shrinkage, GI group index, AASHTO American Association of State Highway Transportation Office.

The penetrometer test values vary between 41 KN/m2 and 95 KN/m2 at 0.25m depth and became higher as the values now vary between 68 KN/m2 to 230 at 0.5m depth (table 2). The increase in the values became very appreciable and continue with depth while between 0.75m depth and 1.00m depth, the values rise and became more appreciable as this vary between 203 KN/m2 and 405 KN/m2. At depth beyond 1.00m, the values keep increasing at all investigation points as the values now vary between 270 KN/m2 and 451 KN/m2.. The CPT graphs show a general increase in the cone resistance readings with depth. The cone resistance readings range from 25Kg/Cm2 – 150Kg/Cm2 at depth below 1 meter and from 130 Kg/Cm2 - 167 Kg/Cm2 at depth beyond 1 meter (table 2).

This is because the minimum allowable bearing pressure at 1m depth is 270KN/m2 which is moderately high enough for some buildings and hence shallow foundation is recommended for building to be erected on this site. Again, the depth of a good foundation also depends on the working load of the intended structure as compared with the estimated allowable bearing pressure at each depth. The coefficient of permeability for the soils ranged from 6.45 × 10-8cm to 1.4 × 10-9cm (table 3). According to U. S. Bureau of Reclamation, soils are classified as (i) Impervious: k (coefficient of permeability) less than 10-6 cm/sec, (ii) Semi pervious: k between 10-6 to 10-4cm/sec (iii) Pervious: k greater than 10-4 cm/sec. this implies that all the soils belong to semi pervious and have low degree of permeability. All of the soils within the five locations with depth are classified as practically impermeable soils i.e. When these soils come in contact with water for a short period of time, the soils will not allow the easy passage of water and will not retain water.

This shows that the soils are good as foundation materials. Soil samples taken from 1m, 2m, and 3m respectively were compacted and subjected to triaxial shear strength tests. The values of the shear strength parameters are; the angle of internal friction ranges from 11.90 to 37.50, the cohesion ranges from 4.7 kPa to 84.9kPa (table 4a & b). Pit A soil has the highest cohesion value (84.9Kpa) while the angle of internal friction is 14.5. The pit D at 1.0m has the lowest cohesion value (4.7kpa) while the angle of internal friction is 37.5. According to a study, the results obtained from the triaxial shear strength test can be used to classify the soils based on angle of internal friction (ASTM, 1993). Soil having angle of internal friction less than 20o are classified as soft, between 20-35o are classified as hard and above or greater than 35o are classified as stiff. Applying eight (8) samples fall within soft category, five (5) samples fall within the hard category and two (2) sample falls within the stiff category (ASTM, 1993).

It shows that more of the obtained values fall within the soft category. On the average per location, all the studied soils possess good shear strengths as evident in the values of their angle of internal friction (ϕ) and cohesion (c). A group researcher reported that the clay fraction content has an influence on the cohesion of residual soils compacted at various moisture contents (Gidigasu, 1976). With these c and ϕ values, the soil will have high bearing capacity and will be good foundation materials for heavy structures. They will also have fairly high slope stabilities and will therefore be useful in the construction of embankments. Pit A to D show that the soils have high cohesion (c) and angle of internal friction (ϕ) values with depths while pit E values decreased with depth.

Sample code	NMC	LL (%)	PL (%)	PI (%)	Fines (%)	Coarse (%)	SG	Activity	Clay Type	GI	AASHTO	USCS
AAM/AD/0.5	14.4	39.0	16.2	22.7	64.52	34.48	2.60	1.23	ILLITE	12.1	A-6	CI
AAM/AD/1.0	15.6	35.5	18.42	17.1	60.41	39.59	2.65	1.26	ILLITE	7.8	A-6	CI
AAM/AD/1.5	13.6	38.4	21.2	17.2	47.00	53.00	2.60	1.13	ILLITE	4.6	A-6	CI
AAM/AD/2.0	18.6	46.2	27.95	18.3	59.40	40.60	2.60	1.09	ILLITE	9.4	A-7-6	CI
AAM/AD/2.5	15.4	41.0	24.03	17.0	47.72	52.28	2.60	1.66	ILLITE	4.9	A-7-6	CI
AAM/AD/3.0	15.6	44.8	26.4	18.4	46.11	53.89	2.65	1.54	ILLITE	5.1	A-7-6	CI
AAM/BD/0.5	14.6	47.0	28.09	18.9	48.25	51.75	2.60	1.40	ILLITE	6.1	A-7-6	CI
AAM/BD/1.0	14.0	48.0	29.32	18.7	47.00	53.00	2.65	1.57	ILLITE	5.6	A-7-6	CI
AAM/BD/1.5	13.8	47.2	29	18.2	59.40	40.60	2.60	1.16	ILLITE	11.0	A-7-6	CI
AAM/BD/2.0	14.6	42.4	24.41	18.0	62.72	37.28	2.60	1.32	ILLITE	9.8	A-7-6	CI
AAM/BD/2.5	15.0	44.0	24.77	18.2	55.75	44.25	2.60	1.52	ILLITE	8.0	A-7-6	CI
AAM/BD/3.0	14.6	47.3	28.73	18.6	50.11	49.89	2.65	1.28	ILLITE	6.7	A-7-6	CI
AAM/CD/0.5	14.0	40.2	22.06	18.1	61.78	38.22	2.60	1.19	ILLITE	9.5	A-7-6	CI
AAM/CD/1.0	15.2	47.0	27.64	19.4	44.17	55.83	2.65	1.43	ILLITE	5.0	A-7-6	CI
AAM/CD/1.5	14.8	50.0	30.95	19.1	44.04	55.96	2.65	2.02	ILLITE	5.0	A-7-5	CH
AAM/CD/2.0	14.0	42.0	23.39	18.6	42.73	57.27	2.60	1.25	ILLITE	4.1	A-7-6	CI
AAM/CD/2.5	14.0	46.1	26.97	19.1	66.27	33.73	2.60	1.15	ILLITE	12.1	A-7-6	CI
AAM/CD/3.0	14.4	50.1	31.1	19.0	58.75	41.25	2.60	1.35	ILLITE	10.0	A-7-5	CH
AAM/DD/0.5	13.0	39.0	22.06	17.0	38.76	61.24	2.65	1.16	ILLITE	2.4	A-2-6	CI
AAM/DD/1.0	15.2	35.2	18.23	17.0	53.00	47.00	2.60	1.53	ILLITE	5.9	A-6	CI
AAM/DD/1.5	14.4	44.2	25.16	19.0	44.77	55.23	2.65	1.47	ILLITE	4.9	A-7-6	CI
AAM/DD/2.0	16.2	39.3	22.36	16.9	53.49	46.51	2.60	1.13	ILLITE	6.3	A-6	CI
AAM/DD/2.5	15.4	46.0	28.84	17.2	52.37	47.63	2.60	1.18	ILLITE	6.7	A-7-6	CI
AAM/DD/3.0	15.2	40.1	22.82	17.3	54.30	45.70	2.60	1.28	ILLITE	6.8	A-7-6	CI
AAM/ED/0.5	19.0	47.0	28.87	18.1	60.53	39.47	2.55	1.24	ILLITE	9.9	A-7-6	CI
AAM/ED/1.0	18.2	47.5	29.23	18.3	49.80	50.20	2.60	1.25	ILLITE	6.5	A-7-6	CI
AAM/ED/1.5	17.6	43.0	25.81	17.2	55.37	44.63	2.60	1.56	ILLITE	7.4	A-7-6	CI
AAM/ED/2.0	18.4	46.0	28.98	17.0	60.99	39.01	2.60	1.31	ILLITE	9.3	A-7-6	CI
AAM/ED/2.5	17.8	46.1	25.66	20.5	49.21	50.79	2.60	1.97	ILLITE	6.9	A-7-6	CI
AAM/ED/3.0	17.8	47.0	30.07	16.9	57.92	42.08	2.60	1.11	ILLITE	8.4	A-7-5	CI

Table 2: Sum-up of penetration test data for the study area

	CONE PENETRATION RESISTANCE VALUES qc (Kg/Cm²) AND ESTIMATED ALLOWABLE BEARING PRESSURE (KN/m²)									
Depth (m)	CPT 01	EST. ABP (KN/m²)	CPT 02	EST. ABP (KN/m²)	CPT 03	EST. ABP (KN/m²)	CPT 04	EST. ABP (KN/m²)	CPT 05	EST. ABP (KN/m²)
0.25	25	67.5	17	46	35	95	15	41	20	54
0.50	85	230	65	176	30	81	50	135	25	68
0.75	98	265	75	203	75	203	100	270	85	230
1.00	125	338	150	405	120	324	125	338	100	270
1.25	148	400	155	419	142	383	130	351	130	351
1.50					150	405	132	356	148	400
1.75									167	451

Table 3: Estimated permeability (K) = C (D_{50})^2 values from grain-size

Sample codes	D_{50} (mm)	Permeability K(m)
AAM/AU/1.0	0.062	1.4×10^{-11}
AAM/AU/2.0	0.064	1.5×10^{-11}
AAM/AU/3.0	0.150	8.03×10^{-11}
AAM/BU/1.0	0.210	1.57×10^{-10}
AAM/BU/2.0	0.063	1.42×10^{-11}
AAM/BU/3.0	0.075	2.0×10^{-11}
AAM/CU/1.0	0.425	6.45×10^{-10}
AAM/CU/2.0	0.213	1.62×10^{-10}
AAM/CU/3.0	0.064	1.50×10^{-11}
AAM/DU/1.0	0.063	1.42×10^{-11}
AAM/DU/2.0	0.063	1.42×10^{-11}
AAM/DU/3.0	0.064	1.50×10^{-11}
AAM/EU/1.0	0.075	2.0×10^{-11}
AAM/EU/2.0	0.065	1.51×10^{-11}
AAM/EU/3.0	0.065	1.51×10^{-11}

Table 4a: Sum-up of average Triaxial shear strength parameters

Sample code	Cohesion (kpa)	Internal friction Ø	Category
AAM/AU/1.0	45.0	18.9	Soft
AAM/AU/2.0	21.3	33.9	Hard
AAM/AU/3.0	84.9	14.5	Soft
AAM/BU/1.0	58.4	16.3	Soft
AAM/BU/2.0	76.6	11.9	Soft
AAM/BU/3.0	53.5	23.5	Hard
AAM/CU/1.0	70.3	17.4	Soft
AAM/CU/2.0	54.1	24.0	Hard
AAM/CU/3.0	23.0	36.3	Stiff
AAM/DU/1.0	4.7	37.5	Stiff
AAM/DU/2.0	50.4	21.4	Hard
AAM/DU/3.0	63.6	19.6	Soft
AAM/EU/1.0	45.8	22.7	Hard
AAM/EU/2.0	77.2	13.5	Soft
AAM/EU/3.0	75.7	12.7	Soft

Table 4b: Sum-up of average Triaxial shear strength parameters for the pits

S/N	Cohesion (kpa)	Internal friction Ø	Category
Pit A	50.4	22.4	Hard
Pit B	62.8	17.2	Soft
Pit C	49.1	25.9	Hard
Pit D	36.2	26.1	Hard
Pit E	66.2	16.3	Soft

6. Conclusion

In this study, the laboratory test conducted on 45 soils sampled revealed that:

i. The atterberg limit values show that all the soils meet the requirement for use as subgrade/fill materials.

ii. On grain size analysis, none of the sampled soils met the specification of the FMWH (1972) of 35% maximum permissible of fine content use for good foundation materials. These soils will fail under repeated shrinkage and swelling conditions.

iii. The soil samples fall between A-6 and A-7-6 in the AASHTO classification and are rated between fair and poor sub-grade materials. Therefore, they fall under clayey soils.

iv. The values of the shear strength parameters are; the angle of internal friction ranges between 11.9^0 and 37.5^0, the cohesion ranges between 4.7 kPa and 84.9kPa and the undrained shear strength varied from 19.6 to 202.4kPa. Applying the USCS classification, eight (8) samples fall within soft category, five (5) samples fall within the hard category and two (2) sample falls within the stiff category. It shows that more of the obtained values fall within the soft category.

In summary, the results of this study clearly indicated that soils in the study area are good for shallow foundation material. This is because the minimum allowable bearing pressure at 1m depth is 270KN/m^2 which is moderately high enough for some buildings and hence shallow foundation is recommended for building to be erected on this site. Again, the depth of a good foundation also depends on the working load of the intended structure as compared with the estimated allowable bearing pressure at each depth.

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