The Use of Seismic Refraction and Geotechnical Parameters to Conduct Site Investigation – A Case Study

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Abstract— *Due to the disastrous effects of building collapse in recent years as a result of improper site investigation, the site for a multi-purpose storey building at New Atuabo in Tarkwa, Ghana was investigated using seismic refraction and some geotechnical parameters. The results for the seismic refraction revealed that there are four strata. The site has a top soil of 1.5 m thickness and second layer made of 8 m thick weathered material. The third and fourth layers being saturated material and bedrock respectively have vertically extensive depths. The seismic refraction also showed an average depth to water table of 9.5 m. The geotechnical tests conducted were Dynamic Cone Penetration Test (DCPT), Sieve Analysis and Atterberg's Limit Test. DCPT was conducted to estimate the bearing capacity of the soil. The safe bearing capacity was 294 kN/m² at a depth of 2.2 m. Sieve analysis results showed that the soil is well graded. And finally, Atterberg's Limit Test was conducted to know the plastic nature of the soil. This yielded an average plasticity index of 7.75 %, which means the soil is silty with low plasticity. The plasticity index was used to estimate the swelling potential of the soil and was found to have a low swelling potential.*

Keywords— *site investigation, seismic refraction, bearing capacity, plasticity index.*

I. INTRODUCTION

A multi-purpose storey building is to be put up at New Atuabo in Tarkwa, Ghana. With the information of a building collapse in the past due to improper or lack of site investigation, the owner contacted some people from the Department of Geological Engineering of the University of Mines and Technology to conduct the site investigation in order to ascertain the safety of the auditorium.

Buildings need firm and competent foundations to be able to last. Some soils are problematic and adversely affect the foundations of structures thereby compromising the stability of the structures. These soil problems have resulted in excessive settlement, tilting and collapse of many buildings not only in Ghana but also around the world (Katzenbach *et al.,* 2005). A study conducted in Ghana shows that between the year 2000 and 2016, there were reported cases of eight buildings that collapsed in Greater Accra Region, five in Ashanti Region and one in BrongAhafo Region with

twenty-six deaths, two missing people and several people sustaining various degrees of injuries. The report also captured the collapse of an uncompleted five-storey hotel building in Tarkwa, Western Region on January 31, 2010 which killed 3 people (Asante and Sasu, 2018).

Geophysics and geotechnical investigations are very useful in site investigations. Seismic refraction method is one of the most commonly used geophysical methods for site investigation and has been employed by many engineers and geoscientists to investigate the subsurface conditions of construction sites such as overburden and litho-stratigraphy, depth of water table and the discontinuities of the subsurface among others (Rucker 2000; Rucker and Ferguson, 2006). Geotechnical investigations are also conducted to describe the nature, subsoil property, soil bearing capacity, soil index property and settlement capacity of the soil and to predict and solve potential foundation problems including ascertaining the suitability

of the soil at the construction site among others (Arora, 2004).

II. LOCATION AND GEOLOGY OF STUDY AREA

New Atuabo is a suburb of Tarkwa, the capital of Tarkwa Nsuaem Municipality, in the Western Region of Ghana. It is accessible by road about 322 km from Accra. Tarkwa is hosted on the unconformable contact between younger Tarkwaian rocks to the west and Birimian rocks to the east. The Birimian rocks consist of penecontemperaneous low-grade sedimentary and volcano-clastic rocks (Eisenlohr and Hirdes, 1992). The sediments have been metamorphosed to lowgrade green schist facies and are commensurate with braided stream environment (Kesse, 1985). The Tarkwaian Group comprises a sequence of coarse, clastic, fluviatile meta-sedimentary rocks consisting of the Kawere conglomerates.

Banket Series (host of gold mineralisation), Tarkwa Phyllite and Huni Sandstone. About 20 % of the total Tarkwaian within the Tarkwa area is made up of intrusive igneous rocks, which form conformable to slightly transgressive sills with small number of dykes. The Tarkwa area is faulted and jointed with the most prominent joints trending in *WNW* to *ESE* direction (Hirdes and Nunoo, 1994). New Atuabo area is predominantly underlain by Kawere Conglomerate and the Birimian.
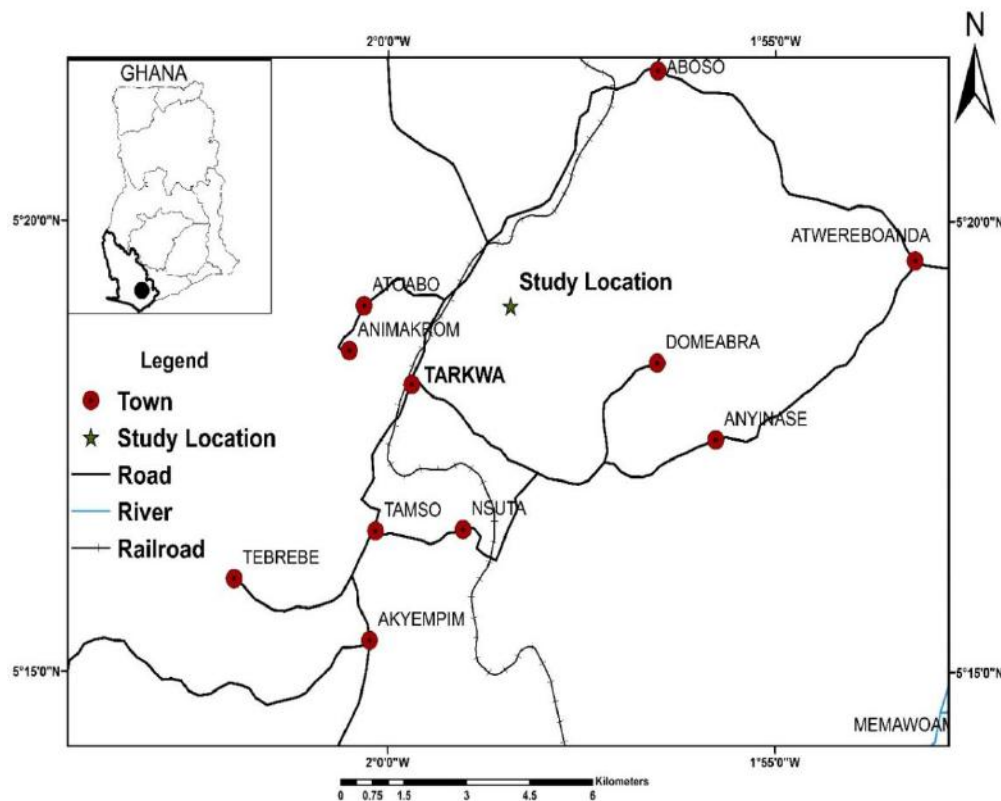

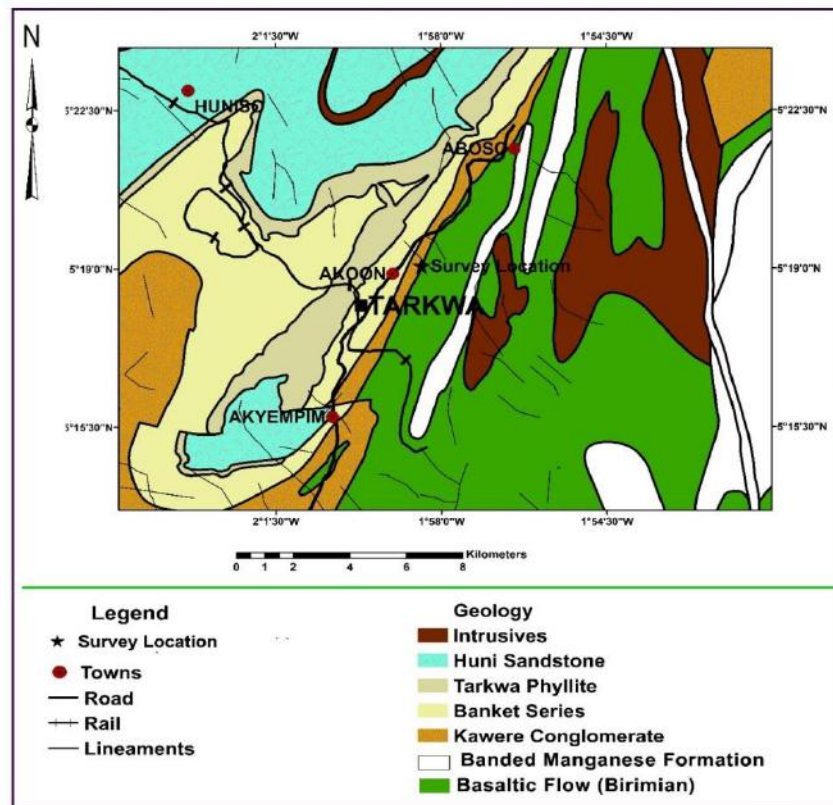*Fig. 1 Map of Tarkwa showing Study Location*

Fig. 2 Geological Map of Study Location

Table 1 Stratigraphic Succession of the Tarkwaian System

Series	Thickness (m)	Composite Lithology
Huni Sandstone (and DompimPhyllite)	1370	Sandstones, grits and quartzite with bands of phyllites.
TarkwaPhyllites	120 – 400	Hunisandstone transitional beds and green chloriticand sericiticphyllitesand schists.
Banket Series	120 – 160	Tarkwaphyllitetransitional beds and sandstones, quartzites, grits, brecciasand conglomerates.
Kerewe Group	250 – 700	Quartzites, grits and phyllitesand conglomerates

III. PROCEDURES

3.1 Field Works

Seismic Refraction Survey

The seismograph equipment was used to measure seismic waves. Geometric ES-3000 seismograph was used for the seismic data collection. The set up was made up of a seismograph, 60 m spread cable, 12 geophones, a sledge hammer and a metal striker plate. Two seismic refraction profiles were acquired across the study area. Stack of three (3) shots was used at various shot locations on a profile to minimize background noise effect and to increase signal to noise ratio. For each profile, the geophone interval was 5 meters and had seven shot points. A sampling rate of 62.50 µs with recording length of 0.25 s was used. Moreover, a low cut filter of 15 Hz was used to filter noise frequency from traffic and a notch filter of 60 Hz was used to filter frequency noise from power lines.

The geophones were firmly inserted into the ground and kept as vertical as possible to achieve best connection of the seismic signal with the geophones. Lower frequency geophones in particular tend to have loose sensitivity if they are not vertical. To achieve maximum signal connection,

loose materials were cleared before placing the striker plate on the ground. The sledgehammer was prevented from bouncing on the striker plate when stricken, to avoid false trigger from occurring.

The first step in the data processing is to pick the first arrivals. This was done with Pickwin, a software designed by Geometrics to be used to pick first arrivals from seismograms. The first breaks were picked automatically and adjusted manually, to achieve high accuracy in the travel times.

The next step was to assign layers to the travel times, which were picked with the Pickwin software. This was done with the aid of Plotrefa, which is a seismic refraction analysis software produced by Geometrics. This arrival time picks are used to plot the travel time curves from where the velocity layers can be estimated from the reciprocal of the slopes obtain from the plot. After the layer assignment, an initial velocity model is estimated using time-term inversion. In this case, a two or three velocity layer model is represented by the results obtained from the simple interpretation of the travel time plot from the seismic refraction data. Tomographic inversion is then generated from the initial velocity model after some number of iterations are completed. After each iteration, ray tracing is initiated to produce a calculated travel time curve. The difference between the calculated and the observed travel times is shown as the RMS error. The rule of thumb is that, the smaller the RMS error, the higher the accuracy of the data. The iterations for the tomographic inversions are stopped when the RMS error reduces to a minimum value, where further iterations results in no change in the RMS error.

Dynamic Cone Penetration Test

Dynamic Cone Penetration (DCP) test was used to determine the bearing capacity of the in-situ ground. The test was conducted in accordance with *BS 1377: Part 9:1990.* The DCP apparatus consists of a 16 mm diameter steel rod with a 60^0 conical tip. The rod is topped with an anvil. The rod was connected to a second steel rod. This rod was used as a guide to allow an 8 kg hammer which was repeatedly raised and dropped from a height of 575 mm. The connection between the two rods consisted of anvil which allowed for quick connections between the rods and for efficient energy transfer from the falling weight to the penetrating rod. After the test apparatus was assembled, the DCP was placed at the test location and the initial penetration of the rod was recorded to provide a zeroing scale. While holding the rod vertically, the hammer was raised to the top of the rod 575 mm above the anvil and dropped. The penetration of the rod was measured and recorded after each drop. The test was terminated when the desired depth was reached.
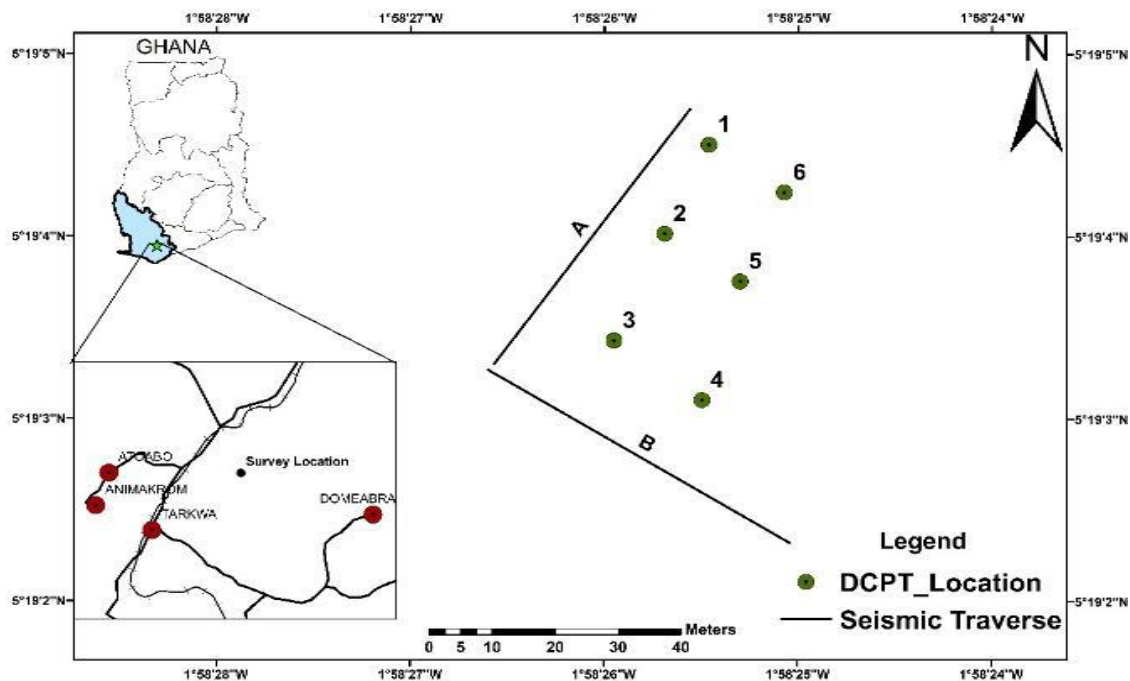

Fig. 3 Locations for Seismic Traverse and DCPT Boreholes

3.2   Laboratory Works

The laboratory works; sieve analysis and Atterberg Limit test were conducted in accordance with *BS: 1377: Part 2: 1990.* These tests give information about the soil index properties.

Sieve Analysis

The percentages of the various sizes of particles of the soil samples were obtained by dry sieve analysis. The sample was first air dried at a temperature less than 50 ^{0}C, turned over from time to time with a metal scoop to avoid local drying out. The size was reduced by the rifling method. 1000 g of the sample was then weighed. Sieving was performed by arranging sieves in descending order of aperture size with a receiver at the base. The sieve sizes used were in accordance with the BS: 410: 1990. The dried sample was placed on the top sieve, covered with a lid and the whole set of sieves was mechanically shaken for 15 min. The quantity of material retained on each sieve was then weighed.

Atterberg Limit Test

Liquid Limit

The liquid limit apparatus was checked to make sure it was clean. The height through which the Cassagrande cup falls was checked to make sure it was 100 mm. Empty moisture content containers were numbered and weighed. Over 200 g of air dried soil was sieved through a 425 μm sieve to remove the coarser particles. The finer particles were placed onto the glass plate. A quantity of distilled water was added to mix. The fine soil was spread on the glass plate and was thoroughly mixed with the distilled water using the palette knife until it became a thick paste. The sample was kept together in the middle of the glass plate to minimize drying due to exposure to air.

With the Cassagrande cup resting on the base, the palette knife was used to fetch and fill the remixed soil onto the cup. The sample was compressed to exclude any trapped air and levelled to the base to a minimum depth of about 10 mm. A depression was made using the grooving tool along the diameter of the cup that passes through the centre of the hinge. The handle of the machine was turned in an anticlockwise manner at a convenient speed. Counting was made on the number of blows as the machine is turned until the groove closed. The process was repeated for several quantity of the mixed soil and each time it was done, a portion of the material was taken into a labelled moisture content container. The container was weighed with its content, was oven dried for a period of 24 hours and was re-weighed to determine the moisture content.

The material remaining in the cup was returned onto the glass plate and was remixed with the rest of the sample together with a little more water to obtain a uniform mixture. The above procedures were repeated to obtain a lower count of blows. The experiment was conducted for at least 4 different moisture contents so as to have the number of blows that were fairly evenly distributed to be between 50 and 10. The moisture content of the samples obtained from each blows count was calculated. A plot of average moisture content against the average number of blows on the semi-log graph paper was made. The best straight line through these points was drawn and was read off the water content corresponding to 25 blows to the nearest 0.1 %. The result was quoted to the nearest whole number as the Liquid limit.

Plastic Limit

About 50 g of soil sample was prepared for a plastic limit test. The sample was mixed with enough water to form a homogeneous dry paste, just plastic enough to be rolled into a ball of about 15 mm diameter. A ball of soil was rolled between the hand and the glass plate until a thread 3 mm diameter was formed. The thread was manipulated into a ball was rolled repeatedly until the 3 mm diameter thread started to crumble. The thread was placed into a moisture content container, weighed, oven dried and the moisture content was determined. The process was repeated 5 times.

IV.   RESULTS AND DISCUSSIONS

4.1   Field Works

Seismic Refraction

As shown in Fig. 3, the seismic refraction survey was conducted along two profiles, thus Profile A and Profile B. Below in Fig. 4 and Fig. 5 are the seismic refraction tomographic models for both profiles where colour bands are used to represent the P-wave velocity passing through each soil or rock layer at subsurface.
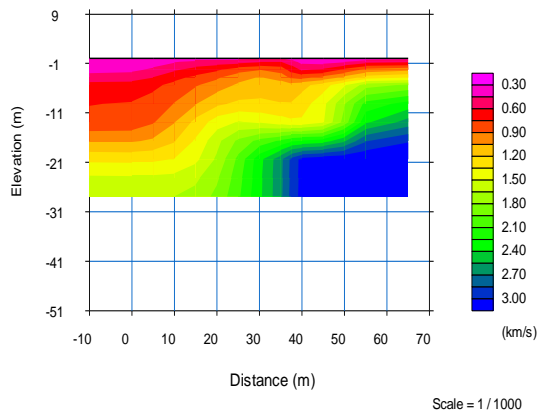
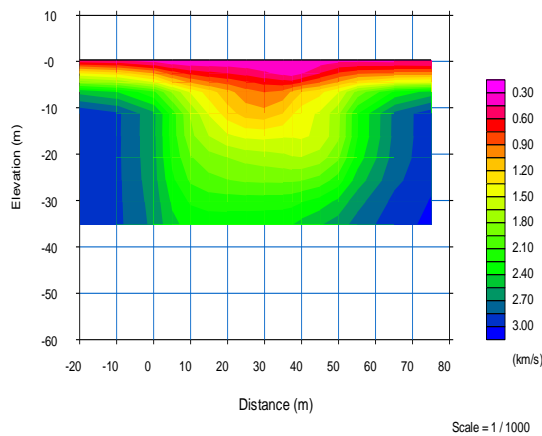*Fig. 4 A 2-D Subsurface Seismic Refraction Tomography Model for Profile A*

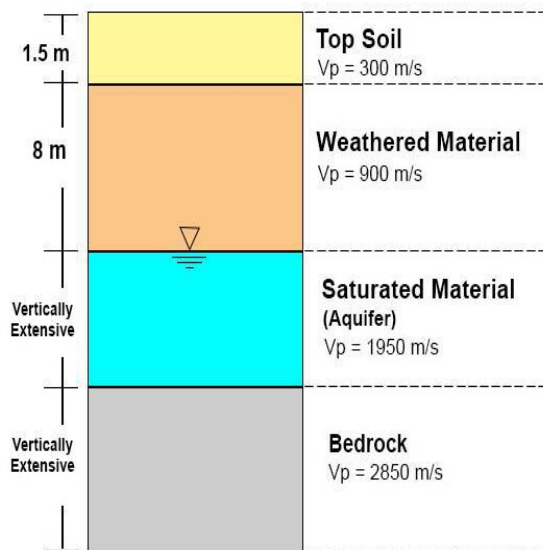*Fig. 5 A 2-D Subsurface Seismic Refraction Tomography Model for Profile B*

Fig. 6 Stratigraphy of the Study Area and Location of the Water Table

Discussion

Based on the velocities measured from the Seismic Refraction Tomography (SRT), the subsurface layers can be grouped into four.

From the SRT analysis, it can be inferred that the subsurface is made up of four layers. The first layer is a top soil with an average thickness of 1.5 m andan average velocity of 300 m/s. The second layer is a weathered material with thickness ranging from 5 m to 11 m and an average velocity of 900 m/s. The third layer is a saturated material with an average velocity of 1950 m/s. The fourth layer is the bedrock, having an average velocity of 2850 m/s. The analysis shows that the aquifer is unconfined and is located in the third layer, which is a saturated material with seismic velocities ranging from 1500 m/s to 2400 m/s. The depth to the water table is averagely 9.5 m. The SRT results conform to the known geology of the study area.

Bearing Capacity

After the DCPT was conducted, the Ultimate Bearing Capacity as well as the Allowable Bearing Capacity values were plotted against their corresponding depths. Below is Fig. 7 showing the graph obtained. from the DCPT. The tabular representation of the DCPT results are in Appendix A.
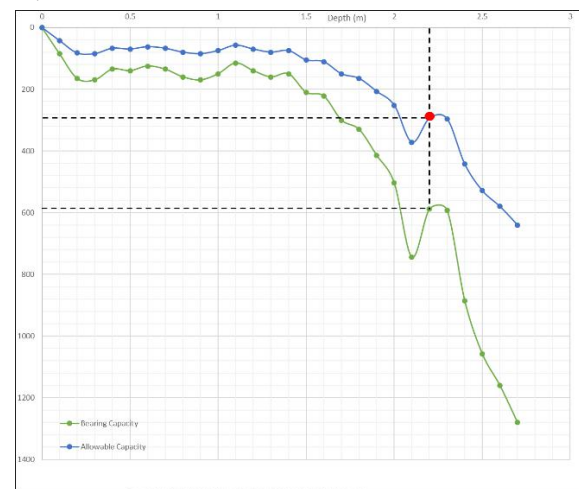

Fig. 7 Graph showing Ultimate and Allowable Bearing Capacities against Depth

Discussion

Based on the graph, a depth of 2.2 m would be convenient for foundation depth. This depth is located in the second layer. At this depth, the ultimate bearing capacity is 588 kN/m^2 and the allowable bearing capacity is 294 kN/m^2. This means that the maximum pressure the foundation can

withstand without undergoing excessive settlement is 588 kN/m^2 and the maximum pressure the foundation soil is being subjected to is 294 kN/m^2.

Laboratory Works

Sieve Analysis

A Particle size distribution graphs were plotted for Tests 1 to 5 in Fig. 8 after the sieve analysis test was conducted. The graphs show the percentages of particular sizes of materials passing through each sieve size.
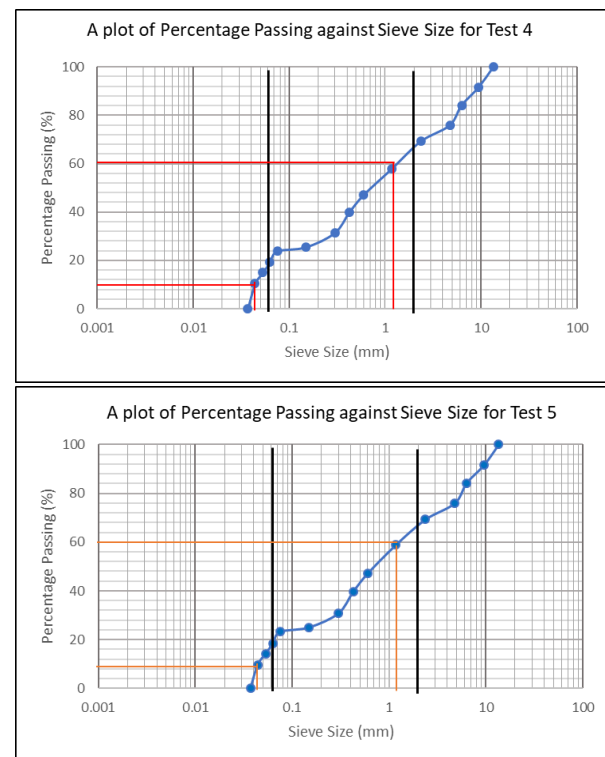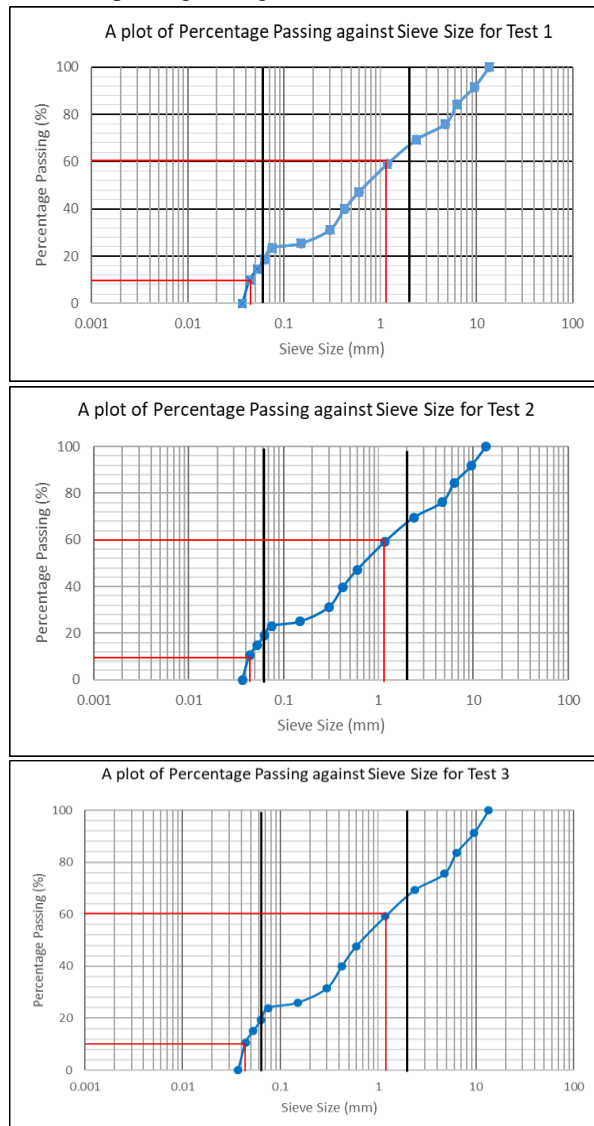

Fig. 8 Particle Size Distribution Graphs for Tests 1 to 5

Table 2 Percentages of soil types at the site

Soil Type	Test 1	Test 2	Test 3	Test 4	Test 5	Average
Silt	19.50	19.00	19.75	20.00	20.50	19.75
Sand	48.50	49.50	50.13	51.00	51.50	50.13
Gravel	32.00	31.50	30.12	29.00	28.00	30.12

Discussion

From Table 2, 19.75% of the soil is made of silt, 50.13% is made of sand and 30.12% is made of gravel.

From Fig. 8, using average values, the Uniformity Coefficient, C_u is computed as

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{...........................Eqn.1}$$

$$C_u = \frac{1.200}{0.044}$$

$$C_u = 26.82$$

The Uniformity Coefficient is greater than 5, therefore the soil is well graded.

Atterberg Limit Test

After the Liquid Limit test was conducted, a graph of Average Moisture Content was plotted against the Number of blows. The graph in Fig. 9 shows the Flow Curve for Tests 1 to 5 of the Liquid Limit test.
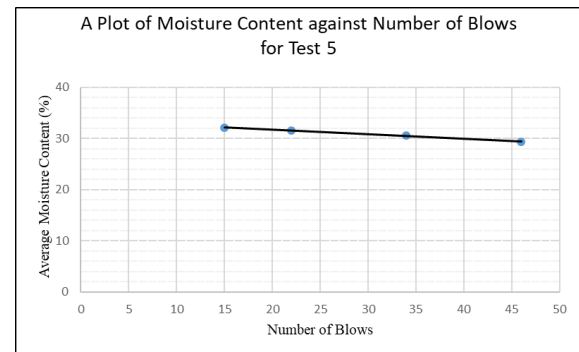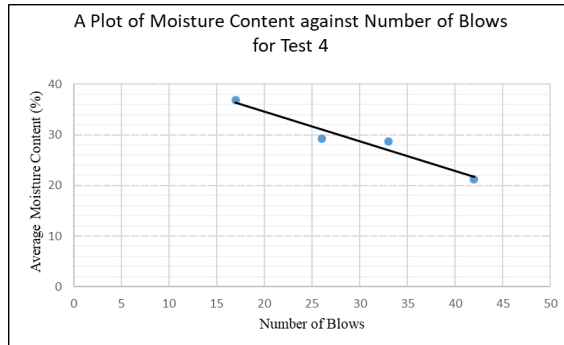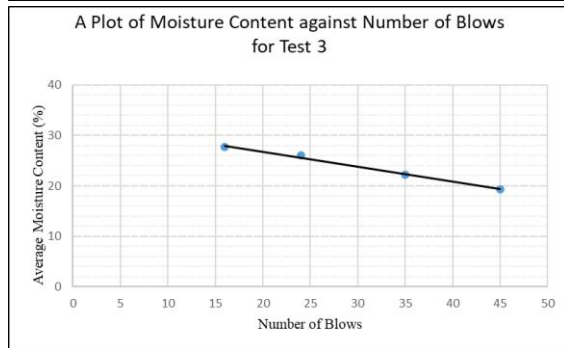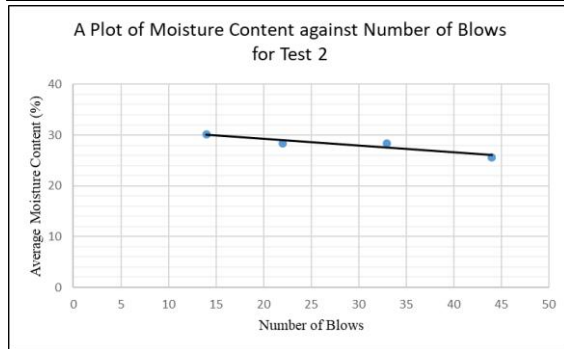

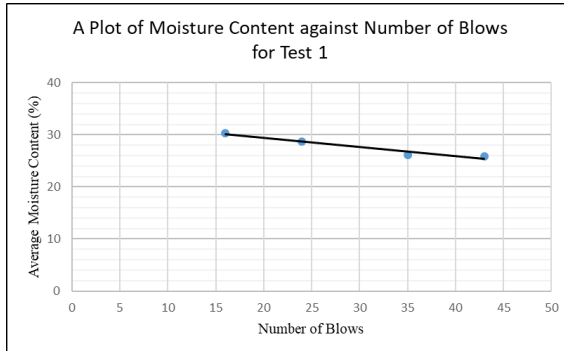*Fig. 9 Flow Curves for Tests 1 to 5*

Discussion

Table 3 Liquid Limit, Plastic Limit and Plasticity Index values obtained from Tests

	Test 1	Test 2	Test 3	Test 4	Test 5
Liquid Limit (%)	27.77	28.15	23.83	28.99	30.90
Plastic Limit (%)	19.84	20.59	16.02	21.26	23.18
Plasticity Index (%)	7.93	7.56	7.81	7.73	7.72
Average P.I. (%)	7.75				

From Table 3, the Liquid Limits range from 23.83 % to 30.90 %, Plastic Limits range from 16.02 % to 23.18 % and the Plasticity Index values range from 7.56 % to 7.93 %. This yielded an Average Plasticity Index of 7.75 %. The Plasticity Index and Liquid Limit values were plotted on the Cassagrande Chart shown in Fig. 10 below. Based on the plotting on the chart, it can be said that the soil at the site would be classified as silt with low plasticity.
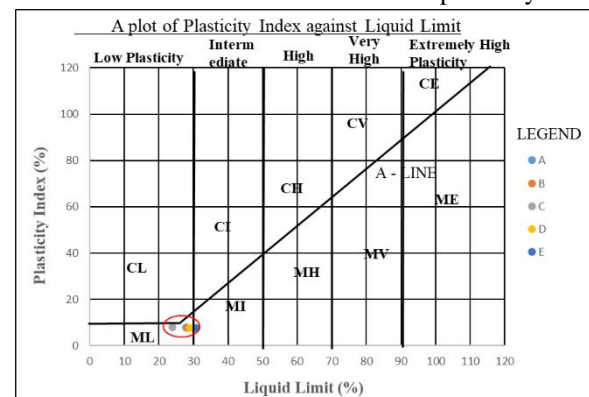

Fig. 10 Plot of Plasticity Index and Liquid Limit on Cassagrande Chart

Swelling Potential

Perk, Hansen and Thorburn (1974) related the plasticity index to the swelling potential of soils in a simple relation shown in Table 4.

The Average Plasticity Index for the Atterberg test was 7.75 %. This value falls within the 0 – 15% Plasticity Index range. Therefore, the soil has a Low Swelling Potential.

Table 4 Relationship between Plasticity Index and Swelling Potential

Plasticity Index (%)	Swelling Potential
0 -15	Low
10 – 35	Medium
20 – 35	High
35 and Above	Very High

V. CONCLUSIONS

- The site has a top soil of 1.5 m thickness and second layer made of 8 m thick weathered material. The third and fourth layers being saturated material and bedrock respectively have vertically extensive depths.
- The depth to the water table is averagely 9.5 m.
- There will be a safe bearing capacity of 294 kN/m² at a depth of 2.2 m.
- The uniformity coefficient of 26.82 shows that the top soil is well graded.
- The plasticity index shows that the soil is Silty with Low plasticity.
- The soil has a low swelling potential.
- All the above results showed that the building would be safe to be constructed on the designated site.
- The detailed subsurface conditions observed shows that the combination of geophysical and geotechnical methods in site investigations yields much information, hence safer.

RECOMMENDATIONS

- A strip footing foundation type should be used.
- The foundation should be made to reach a depth of 2.2 m.
- To enhance the integrity of the foundation of this project, reinforced beams should be used.

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