Geotechnical Investigation Report

Clients: ANSAH AGYEMANG BARIMAH
&
RITA AGYEMANG BARIMAH

Location:

ADWEMADOR

Subject:

GEOTECHNICAL REPORT FOR A PROPOSED 3NO 5-STOREY CLASSROOM BLOCK AND 5-STOREY ADMINISTRATION BLOCK

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EXECUTIVE SUMMARY

The geotechnical investigation was carried out to determine the suitability of a site for the construction of a Proposed School Project. The site is located at Adwemador in the Ningo Prampram District Assembly of the Greater Accra Region of Ghana. The site can be located on google map (5.782874, 0.021333). The land is approximately 1.477 acres or 0.598 hectares. Our clients run a Non-Governmental Organisation (NGO), which is committed in giving formal education to children of school-going age in deprived communities.

This geotechnical investigation report is in partial fulfillment for the acquisition of Building Permit from the District Assembly, and also to aid the structural engineer in his design calculation.

Five (5No.) dynamic cone penetrometer test (DCPT) was conducted at the proposed site. Four number investigative trial pits were done to expose the ground for further studies. The excavation exposed at the proposed project site revealed that, the soil at the site mainly consist of ancient igneous rocks, ancient metamorphosed sediments, relatively younger sediments, Dahomayan gneiss and schists.

The soil profile as revealed by the trial pit is shown in the Appendix C. Logs of the DCP tests penetrated the various soil strata and it encountered refusal at a depth of 3.0m to 4.2m. Based on the observations of the geologic features of the reference site area, the estimated depth to weathered bedrock could be within 7m to 8m. From the field and laboratory testing results, high allowable bearing pressures of about 490 KN/m^2 may be mobilized at the existing foundation depth of about 2.1m for the Proposed 5-Storey School Project.

The foundation for the proposed buildings is recommended to sit within the silty sandy gravel/ very dense sandstone layer at a least depth of 2.0m and beyond. Isolated pad spread footings, are recommended with allowable bearing pressures of 490 KN/m^2 based on the amount of settlement that can be tolerated.

Though this site is not seismic prone zone, formation at this site conforms to seismic class B for EC 8 and equivalent to seismic class D of the ASCE 7 classification. Agreed deep rock horizontal peak ground acceleration (PGA) of 0.2g may be considered for the design

of the proposed construction of the Proposed 5-Storey School Project. These values are defined for an annual exceedance probability of 10% in 50 years.

The recommendations given in this report are based on the expert opinion of the engineers and take technical feasibility, construction expediency, and cost into account.

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1.0 INTRODUCTION

Geotechnical investigations are necessary when a new structure is to be put up or when additions and alterations are to be made to existing structures. Since soils are often inhomogeneous and show variability in structure and composition, a sub-soil investigation followed by engineering test works will enable the engineer evaluate the load bearing capacity of the foundation, estimate the probable settlement of the structure and provide information for the selection of the type and depth of the foundation suitable for the structure. In addition, the investigation will establish any potential foundation problems such as expansive or collapsible soils etc.

For this reason, the clients **Ansah Agyemang Barimah and Rita Agyemang Barimah**, contracted us to conduct a geotechnical investigation on their land. They intend to construct a 5-Storey School Building at Adwemador.

The purpose of the investigations was to establish the subsoil conditions and the bearing capacity of the foundation soil at the site for the purposes of subsequently designing the foundation for the above-mentioned project. The investigation was undertaken on the 26th March, 2022. The extent of the investigation included:

- Reviewing available geological data within the vicinity of the proposed development.
- Conducting field and laboratory test for the soil.
- Performing the necessary engineering analysis and preparation of geotechnical report for the client.

The subsurface investigation was carried out in a manner consistent with principles generally accepted in the geotechnical profession and it was in accordance with British Standard Code of Practice BS.5930, 1999 - Site Investigation for Civil Engineering Projects.

1.1 Objectives & Scope of the Investigation

The main purpose of this soil investigation was to assess the soil load bearing capacity and deformation characteristics, which may impose restrictions on the design and construction of the proposed structure. In addition, other objectives of the investigation included:

- The performance of field and laboratory testing.
- Observations of likely foundation constraints, analyzing the significance and impact of the constraints and making recommendations for possible variations if any, for the design of the foundation type to support the proposed Building Project.
- The level of ground water table and the chemical constituents (if any).
- The existing soil compositions around the existing foundation.
- The Seismicity location of the area.

Among other things, this geotechnical investigation also seeks to establish the thickness and the characteristics of overburden soils present at the proposed site and the influence of ground water (if any) on substructure and superstructure construction.

2.0 SITE LOCATION

The project is located at Adwemador, which is a suburb within the Ningo Prampram District Assembly (NiPDA). The Ningo Prampram District is one of the twenty nine (29) districts in the Greater Accra Region. Originally it was formerly part of the then-larger Dangme West District in 1988, which was created from the former Dangme West District Council, until the southern part of the district was split off to create Ningo Prampram District on 28th June 2012. The district capital of prampram. The district is located in the eastern part of Ghana (proximity to Accra) in the Greater Accra Region. It has a total land size of 622.2 square kilometres.

2.1 Project location

The site is within Adwemador. The road network to the site is untarred. There are no drains around the site. Electricity is available but there is no water connection to individual properties. Most of the dwellers rely on private water suppliers (water tankers) for water supply. Others have also constructed borehole. It is imperative to state the area is fast developing therefore the district assembly in collaboration with the utility companies should drive their focus to the area.

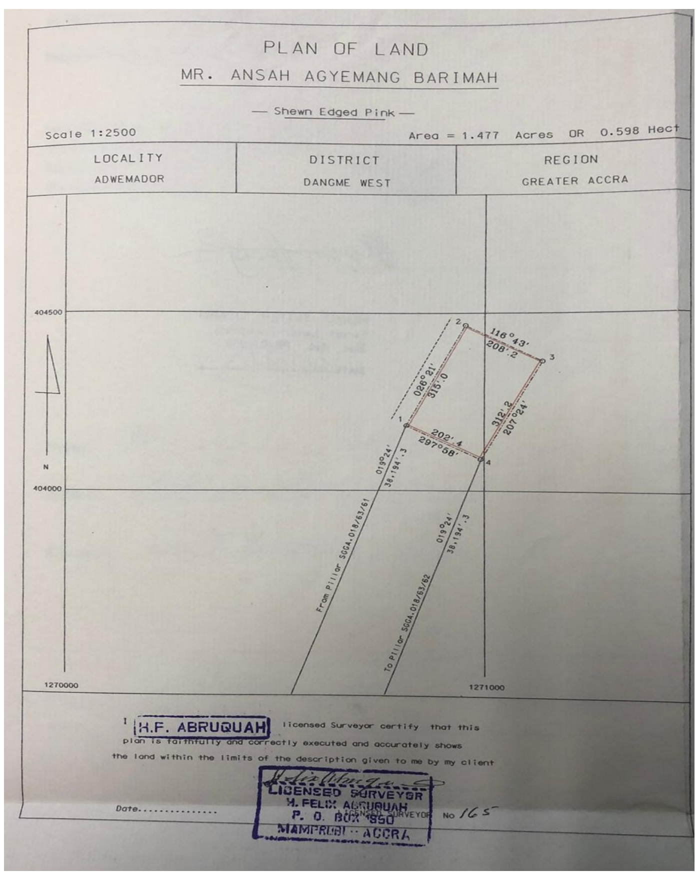

Fig 1: Site Plan

2.2 Climatic Conditions.

Owing to its location in the Dahomeyan Gap, where the coast runs parallel to the prevailing moist monsoonal winds, Accra features a tropical savanna climate (Köppen climate classification) that borders on a hot semi-arid climate . The average annual rainfall is about 730 mm, which falls primarily during Ghana's two rainy seasons. The chief rainy season begins in April and ends in mid-July, whilst a weaker second rainy season occurs in October. Rain usually falls in short intensive storms and causes local flooding in which drainage channels are obstructed. Very little variation in temperature occurs throughout the year. The mean monthly temperature ranges from 25.9 °C (78.6 °F) in August (the coolest) to 29.6 °C (85.3 °F) in March (the hottest), with an annual average of 27.6 °C (81.7 °F). The "cooler" months tend to be more humid than the warmer months. As a result, during the warmer months and particularly during the windy harmattan season, the city experiences a breezy "dry heat" that feels less warm than the "cooler" but more humid rainy season. As a coastal city, Accra is vulnerable to the impacts of climate change and sea level rise, with population growth putting increasing pressure on the coastal areas. Drainage infrastructure is particularly at risk, which has profound implications for people's livelihoods, especially in informal settlements. Inadequate planning regulation and law enforcement, as well as perceived corruption in government processes, lack of communication across government departments and lack of concern or government co-ordination with respect to building codes are major impediments to progressing the development of Accra's drainage infrastructure, according to the Climate & Development Knowledge Network. As Accra is close to the equator, the daylight hours are practically uniform during the year.

Relative humidity is generally high, varying from 65% in the midafternoon to 95% at night. The predominant wind direction in Accra is from the WSW to NNE sectors. Wind speeds normally range between 8 and 16 km/h. High wind gusts occur with thunderstorms, which generally pass in squalls along the coast. The maximum wind speed record in Accra is 107.4 km/h (58 knots). Strong winds associated with thunderstorm activity often cause damage to property by removing roofing material. Several areas of Accra experience microclimatic effects. Low-profile drainage basins with a north-south orientation are not as well ventilated as those oriented east-west. Air is often trapped in pockets over the city, and an insulation effect can give rise to a local increase in air temperature of several degrees. This occurs most notably in the Accra Newtown sports complex areas.

2.3 Vegetation

The vegetation is mainly coastal savannah characterized by short savannah grass and interspersed with shrubs and short trees. The vegetation is highly influenced by the climatic condition which results in a long period of dry season. Along the coast, there are stretches of coconut trees and patches of coconut groves which combine to give the area a classic look.

2.4 Topography

The site is gently rolling; drain should therefore be managed effectively with construction of lined drains to collect surface runoff away from the site.

2.5 Geology

The geology of the region can be divided into three distinct i.e. Accraian, Togo and Dahomeyan series known as Accra formation. The Accraian series belong to the Devonian age are sedimentary deposits, and consists of upper interbedded sandstone and Shale, Middle clay shale and lower sandstone. The Togo series belong to the Upper Middle Precambrian age and consists of Quartzite, Shale and Phyllites. The Dahomeyan system belong to the middle Precambrian age and consists mainly of acid and basic horn blends Gneiss, Quartz mica schist, Muscovite-biotite gneiss and Biotite gneiss.

The project site is underlain by rocks belonging to the Dahomeyan series of early to Middle Precambrian age

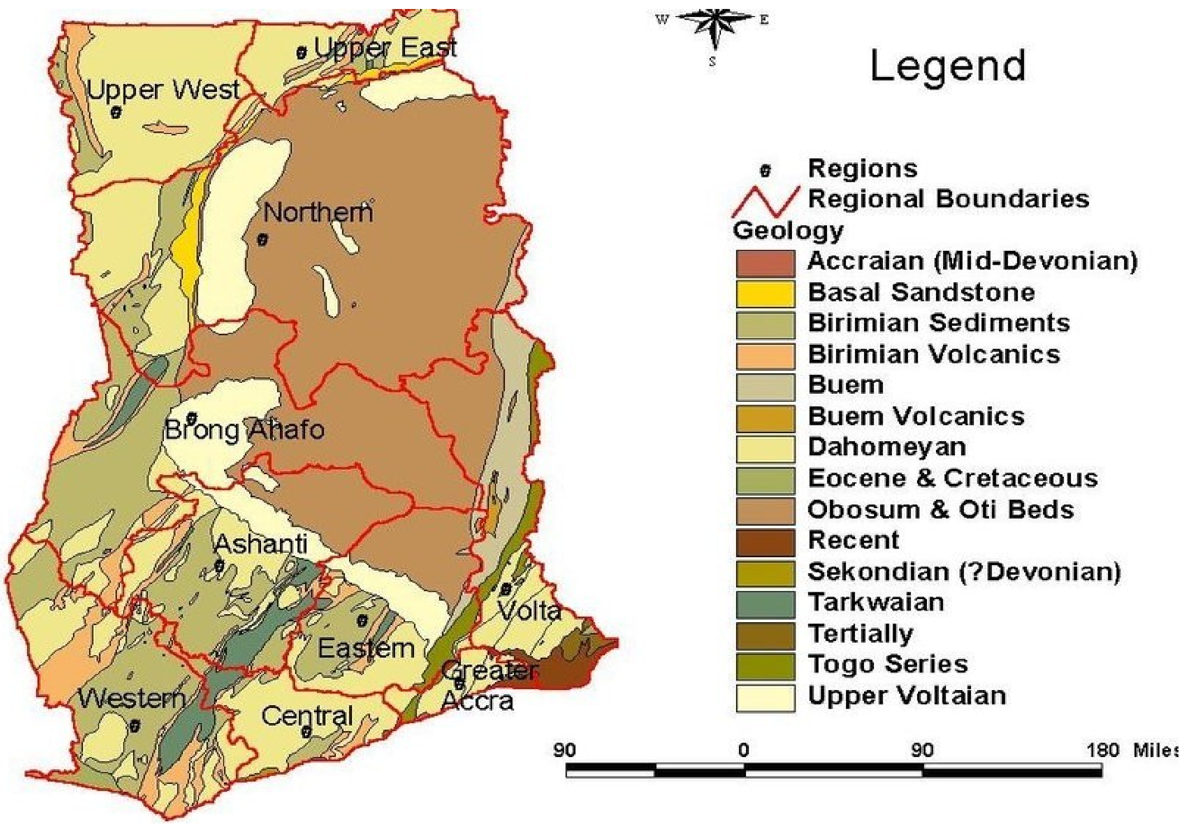

Fig 2: Geological Map of Ghana.

2.6 Seismicity of the Site.

The history of seismic activity in Southern Ghana dates back to 1862. Most of the earthquakes in Ghana occur in the western part of Accra at the junction of the two major fault systems namely, the Coastal boundary fault and Akwapim fault zone. According to seismologist, most of the epicentres are located south of Weija in Accra, suggesting that there is little activity north-eastward along the Akwapim range and westward along the Coastal boundary fault. Most earth tremors in Ghana are related to the level of activity of the faults. The proposed site is not in a seismically active zone. In addition, there are no known faults located within the project area. However, in 1939 a major earthquake which flattened Accra with its attendant casualties was felt in Eastern Ghana and Togo.

The maximum intensity of the shocks was estimated to be 6.5 on the Richter scale. Between 1987 and 1990, over eight seismic events with magnitude ranging from 2.5 to 4 on the Ritcher scale have occurred with epicentres in the Accra – Tema area; (Kato, 1990). Several minor shocks are picked up by seismic monitors may be available to foundation designers. This has been adapted for estimating the Peak Horizontal Ground acceleration in the various Hazard zones demarcated in the zonation map presented below.

	ZONE 4	ZONE 3	ZONE 2
Max. Intensity (Imm)	IX	VII	V
Max. Magnitude (MI)	6.5	6.5	6.5
Average Epicentral Distance (R)	20km	40km	100km
Peak Ground Acceleration (a)	347cm/sec	112cm/sec	32cm/sec
a/g	0.35	0.12	0.03

Table 1. Isoseismic classification of Southern Ghana

This suffices to state that, Project location qualifies within the Zone 4, or having a value of horizontal ground acceleration with peak ground acceleration of 347cm/sec.

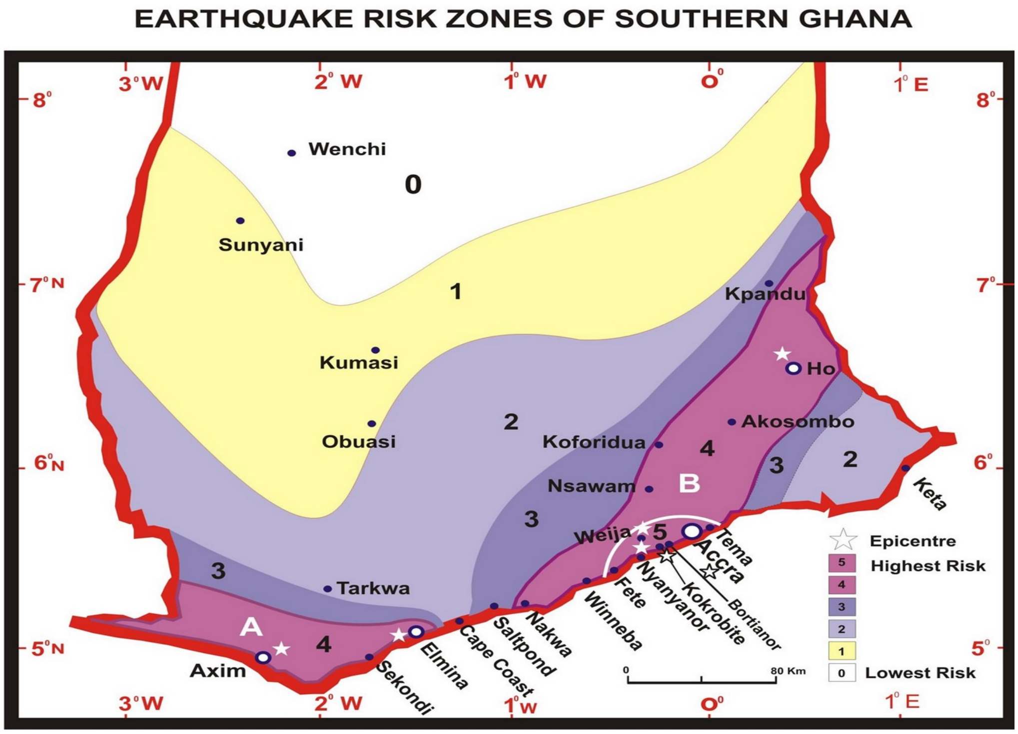

Fig 3: Earthquake Zones of Southern Ghana.

This suffices to state that, Project location qualifies within the Zone 4, or having a value of horizontal ground acceleration with peak ground acceleration of 347cm/sec.

Location	PGA values (g) Present study	Previous studies			
Accra	0.20	0.14–0.57	0.15	0.08–0.16	0.35
		Amponsah et al. (2009)	Kumapley (1996)	GSHAP	CSDCS (BRRI 1990)
Weija (Accra–West)	0.20	0.2	0.15	0.08	0.35
Ho	0.10		0.10 Kumapley (1996)	0.04	0.25
				Anon. (1988)	CSDCS (BRRI 1990)
Cape Coast	0.026		0.15 Kumapley (1996)	0.02	0.15
				Anon. (1988)	CSDCS (BRRI 1990)

Table 2: PGA Values of Southern Ghana.

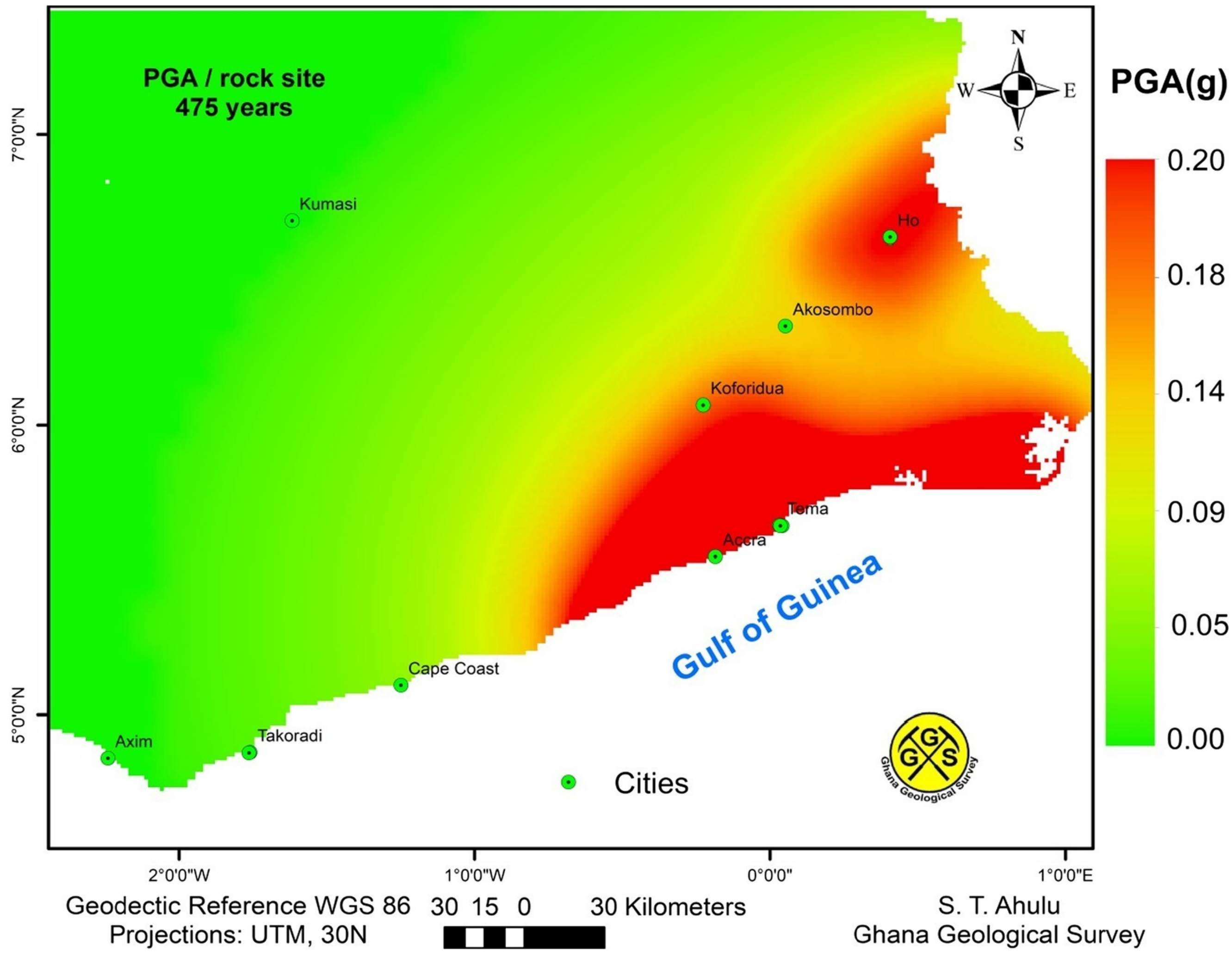

Fig 4; Earthquake Zones of Southern Ghana.

However, from the table and figure above by Sylvanus T. Ahulus; Sylvester Kojo Danuor; Daniel K. Asiedu , Probabilistic hazard assessment of Southern Part of Ghana, PGA value of 0.2g for Accra and Tema corresponds to probability of such events occurring to 0.1, and is expected to be exceeded with probabilities of 10, 30 and 60% in 10, 50 and 100 years respectively. Thus, the probability of occurrence of such a likely B scenario earthquake is moderate. In the same vein, if Accra and Tema zones are likely to experience 0.2 g every 10 years, then it means the acceleration to 475 years is high, and therefore Accra and Tema is a highly hazard zone.

3.0 SUBSOIL INVESTIGATION

The investigation of the proposed 5-Storey School Project included fieldwork and laboratory analysis detailed engineering analysis of the hydrological cycle was undertaken to confirm the movement of moisture.

3.1 Fieldwork

The fieldwork consisted of performing Dynamic cone penetrations test (D.C.P.T). This enables the variation of the soil strength with depth to be evaluated. The fieldwork also comprised reconnaissance of the reference site area, trial pit excavation to establish the various soil layers and logging of the soil layers was undertaken, in addition sampling of disturbed and undisturbed soil samples for laboratory testing to establish the particle size distribution and the Atterberg Limits.

3.2 Dynamic Cone Penetration Test.

The bearing capacity of the foundation soil was evaluated using the dynamic cone penetrometer (D.C.P) made to German specifications (DIN 4094).

The equipment has:

Hammer weight	10kg
Anvil of weight	4.5kg
Height of fall of hammer	575mm
Cone Diameter	25mm
Cone of surface area	$5cm^2$
Degree cone tip	60^0

3.3 Performance of the Test

The equipment was driven at five test points selected within the area of the proposed site. The DCP test was done to help determine the allowable bearing capacity of the soil supplied for the foundation system for the above project.

The number of blows required for the cone to penetrate 10cm into the ground was recorded for various depths. The test was terminated when the number of blows required for the cone to penetrate 10cm exceeds 50 blows. The blow count per 10cm penetration are converted to soil resistance (kN/m^2) or (Kpa).

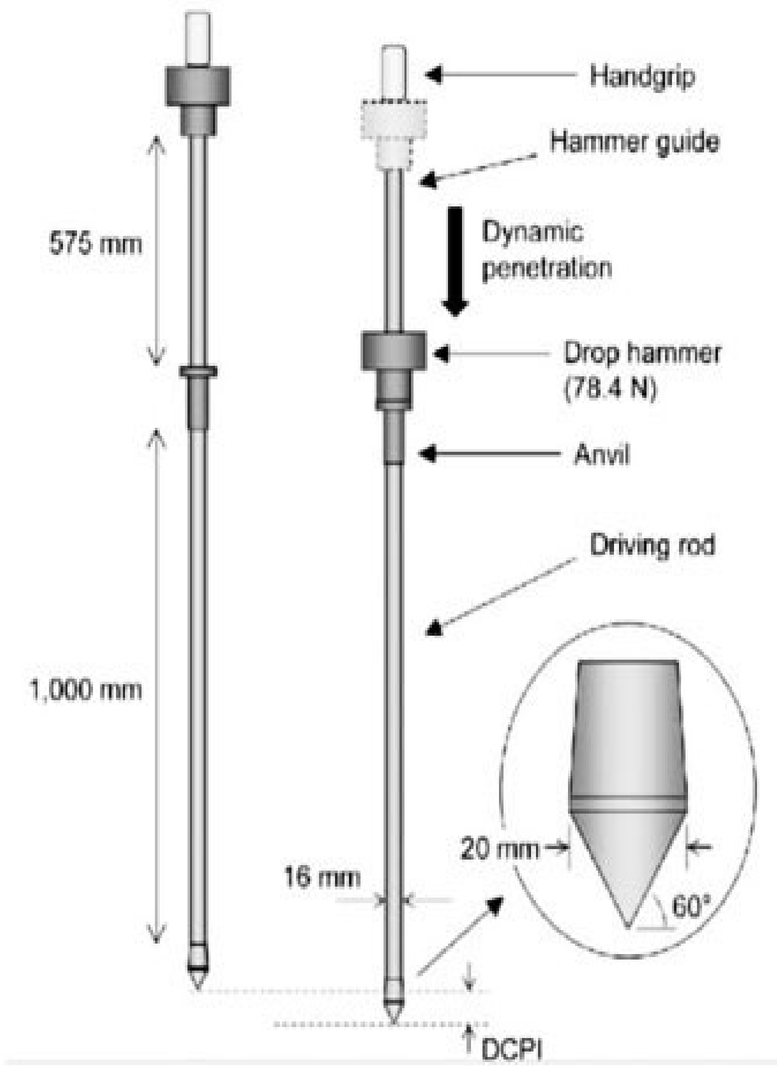

Fig 5: DCPT Equipment.

The cone penetrometer was used to probe at four test points in the vicinity of the proposed development at the discretion of the supervising structural engineer. (See the appendix B of this report for the test points).

3.4 Ground Exploration

Four investigative trial pits were dugged at the site to expose the ground for the soil strata to be determined and identified. This enabled the soil profile to be assessed. Also disturbed soil samples for laboratory testing was recovered for the various layers and preserved. Excavation was terminated at a depth of 2.1m, 3.4, 3.4 and 4.0m below the excavated ground level. Six bulk disturbed samples of the soil namely DS1 to DS6, were recovered and preserved.

3.5 Engineering Tests on Samples

The following standard engineering tests were performed on selected representative samples of soil recovered from the trial pits in soft ground. Results of all laboratory tests summarized and presented in Appendix D. All tests were based on BS 1377; Method of Test for Civil Engineering Soils. Results of laboratory test and field data were analyzed for the formation of recommendations given in this report.

3.6 Classification Tests

Standard tests were performed for the determination of;

- natural moisture content;
- Atterberg limits;
- particle size distribution by sieving and by pipette method; and

3.7 Chemical Tests

Standard tests for the determination of

- pH of groundwater
- sulphate content of groundwater/soil, and
- chloride content of groundwater/soil

The tests results are presented in Appendix D.

3.8 Geological Hazards

Strong ground shaking during an earthquake can result in ground failure conditions such as: soil liquefaction, lateral spreading, differential compaction, and excessive ground vibration. The first three are discussed below; ground vibration is accounted for by developing the seismic input for structural analysis and is tackled in the next section.

3.9 Liquefaction

Liquefaction is the phenomenon whereby under dynamic loads, saturated soils (mainly loose sands and also some loose silty or clayey soils) lose their strength and stiffness due to the build-up of excess pore pressure. The factors affecting liquefaction susceptibility include the type and compactness of the soil, natural water content and plasticity of fines, gradation, and the magnitude and duration of the ground motion. From the previous sections, the site is composed mainly of an upper surficial of light brown / whitish red silty sandy GRAVEL, followed by dark greyish brown silty CLAY and underlain by light reddish brown silty sandy GRAVEL, with dense sandstone. Noting the composition, denseness and plasticity of these soil layers, the soils at the site are not prone to liquefaction.

4.0 Lateral Spreading

Lateral spread is the finite, lateral displacement of sloping ground (0.1 to <6 percent) as a result of soil liquefaction during an earthquake. This occurs when a soil mass slides laterally on a liquefied layer in a downslope direction. The magnitude of lateral spreading movements depends on various factors including, earthquake magnitude, distance between the site and seismic event, thickness of the liquefied layer, ground slope, etc. As stated in the previous section, the liquefaction potential for the site is considered negligible, lateral spread potential is therefore also considered negligible.

4.1 Seismic Design Parameters

4.1.1 PGA Estimation

A detailed seismic hazard assessment study is yet to be conducted for the country. On a macro-scale hazard for the African region, a Global Seismic Hazard Assessment study was conducted. This provided the Greater Accra Region with a rock peak ground acceleration of 0.16g for an annual exceedance probability (AEP) of AEP of 10% in 50 years. This is noted in the literature to be a lower bound estimate. From the above, and based on a knowledge of the seismic hazard of the area, the geologic setting of Southern Ghana, and ground motion estimates from other similar locations in the world with similar seismo-tectonic features, a deep rock PGA (i.e., zero-period spectral acceleration) with an annual exceedance probability of 10% in 50 years of 0.2g could be assumed for the site. For all new structures to be constructed within the site, a deep rock PGA of 0.2g is recommended to be used for design as proposed by Sylvanus T. Ahulus; Sylvester Kojo Danuor; Daniel K. Asiedu.

4.1.2 Seismic Input Definition

The above noted PGA values are used in combination with the EC8 Type I standard design spectrum, and an estimate of the seismic site class, to define the elastic design response spectrum to be used as seismic input for structural analysis. The EC8 code recommends two types of design spectra: Type 1 for locations where earthquake magnitudes greater than 5.5 dominate the seismic hazard, and Type 2 are for locations where magnitudes below 5.5 dominate the hazard response. Since there had not been any past earthquakes experienced in the upper west region, the Type 2 spectrum was chosen. According to EC8, the seismic site class can be estimated using either shear wave velocity data or SPT/ DCPT blow counts for the soil/rock medium within the top 30 m of the site soil/rock profile. Based on the depth-averaged DCPT blow counts, the site can be classified as a seismic class B site although this is equivalent to seismic class D of the ASCE 7 classification.

According to EC8, buildings whose seismic resistance is of importance in view of the consequences associated with collapse, like schools, assembly halls, cultural institutions are tagged with an importance category II. Based on this, the proposed five storey commercial building can be classified to be an importance category II structure, and the seismic input defined above, should be multiplied by a factor of 1.2 to obtain the input elastic design spectrum. Furthermore, the importance category of any new construction on the site must be assessed and considered in the development of the final input elastic design spectrum.

4.0 ENGINEERING CONSIDERATION

4.1 Ground Conditions and Soil Characteristics

The excavation exposed at the proposed project site revealed that the soil at the site mainly consist of light brown / whitish red silty sandy GRAVEL, followed by dark greyish brown silty CLAY and underlain by light reddish brown silty sandy GRAVEL, with dense sandstone. The soil profile as revealed by the trial pits is shown in the Appendix. Logs of the DCP tests penetrated the various soil strata and it encountered refusal at a depth of 2.1m to 4.0m. Based on the observations of the geologic features of the reference site area, the estimated depth to weathered bedrock could be within 7m to 8m.

4.2 Groundwater Investigation

Groundwater was encountered within the depth explored at 1.4m below existing ground level. However, the soil recovered at the site was very firm. In addition, samples encountered within the various layers have a moderate potential to retain water and this shows that the swell potential of the soil there will be minimal. In this situation, the exposure of the concrete to sulphate and chloride attack is likely to be in a lower range. However, it is recommended that concrete for the substructure should be designed and produced to a dense consistency to make it less permeable. It is therefore recommended that some form of dewatering system be provided where necessary.

4.3 Chemical classification

Chemical analysis performed on the soil samples recovered from the reference site in the laboratory Indicated that the concentration of chemicals (sulfate and chloride) known to be injurious to Portland cement concrete are found to be in lower quantities.

Test	Results	Limits
pH	6.7	5-7
Sulphate Content (mg/l)	53.5-54.4	200
Chloride Content (mg/l)	61.6-64.3	300

4.4 Soil Bearing Capacity Estimation

The lightweight dynamic cone penetration 'r', defined as the number of blows required for advancing the cone by 10cm may be converted into unit resistance R_D of the ground in kN/m2 or kPa using the formula:

$$R_D(KPa) = \frac{m^2H}{Aem.\ P.}$$

For shallow foundations, the ultimate bearing capacity q_u may be obtained from the unit R_D by the following relationship.

$$q_u = \frac{R_D}{20}.kPa.$$

For simplicity, the ultimate bearing capacity may be obtained from the approximate relationship:

$\mathbf{q_u = 30\ r}$ (kPa)

The individual DCP results, the ultimate bearing capacities for the site has been presented in Appendix B.

- Alternatively, the strength characteristics of the sub-soils may be determined by converting the blow counts collected in the previously discussed steps above into Standard Penetration Test (SPT) N-values using the relationship below:

$$\mathbf{N_{SPT} = 0.7N_{DCP}}$$

Since foundations are typically constructed in excavations, the total overburden stress, $\mathbf{q_{ob}}$, removed at the foundation level as result of the excavation has to be accounted for in order to obtain the net ultimate bearing capacity, thus:

$$\mathbf{q_{netu} = q_u - q_{ob}}$$

Consequently, the allowable bearing capacity q_{all} can be obtained applying the appropriate factors of safety (FOS) to the net ultimate bearing capacity, q_{netu}.

The allowable bearing pressure is obtained by applying the appropriate factor of safety and the choice of safety factor should be based on the extent of subsurface investigation, reliability of the estimated loads, and importance of the structure and consequences of failure. A safety factor of 3.0 is recommended in this case. A graph showing allowable bearing pressure against depth is provided in the Appendix

Table 3.0 - Safe Bearing Capacity and accompanying Settlement

Depth (m)	Allowable Bearing Capacity (KN/m^2)	Estimated Settlement (mm)
1.0	40	25
1.5	50	25
2.0	50	25
2.5	120	25
3.0	300	25
3.5	500	25

4.5 Settlement Consideration

Settlement magnitude was estimated base on values from various test conducted on the soil samples, the nature of incoming load, the strata and strengths of the soil layers, foundation depth and type, and the average estimated bearing capacity which is based on desiccation and differential settlements. Therefore, for the recommended safe bearing capacity given in Table 3.0, potential settlement could be 25mm or less

4.6 Foundation Depth and Type

The nature of the sub-soils on this site is such that shallow foundations can be considered. The allowable bearing capacities shown in our report, was obtained by dividing the ultimate bearing capacity by a factor of safety of 3.0. Reference should be made to this chart when designing foundation pads. The load bearing capacities of soils within the load stress bulb should be evaluated, since it is the soils within this region that actually feels the effects of the transferred loads. The zones of interest based on this principle extends below the foundation pad to depths approximately twice (2x) the width of the footing.

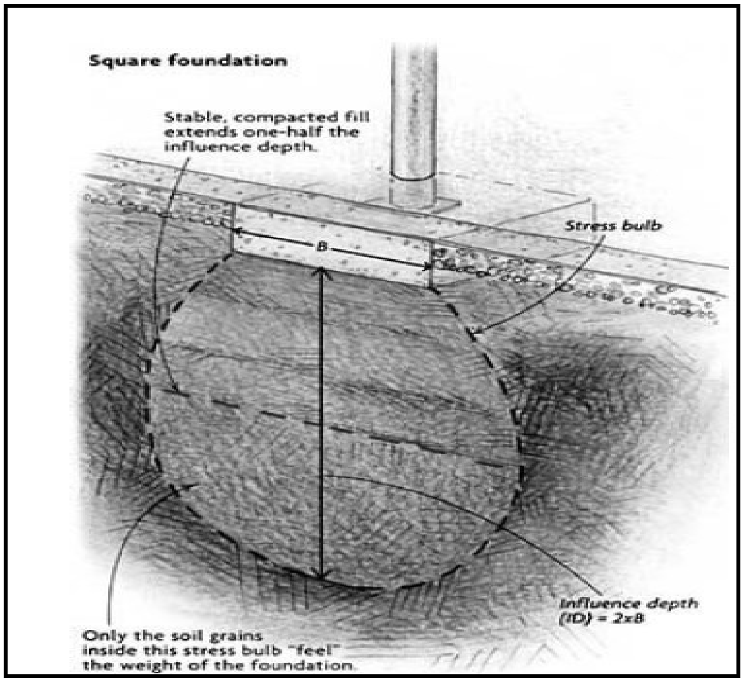

Fig 6: Foundation showing concentration of stress bulb

In the event whereby conventional pad and column style of foundation is to be used, we recommend that ground beams be incorporated to mitigate the probability of any long-term differential settlement and lateral movements of the individual pads.

Although the site is regarded as an area with low frequency of earthquake, it is advisable to design the foundation against possible earth movements. It is recommended that in the structural analysis of the structure against earth movement a horizontal ground acceleration of 0.2g be used. (g is acceleration due to gravity; 9.81m/s^2).

4.7 Excavation and Shoring

In accordance with Ref. [21], upper-surficial soils on the project site can be classified to range between Type A & B soils. Ref. [21] sets the maximum allowable slope for Type B soils with a maximum depth of 3 m to be 45˚. Maximum allowable slopes for the proposed excavation for the structures should thus be 45˚, or 1:1 (H:V). In cases where cut slopes are considered to be unstable, some form of shoring should be provided, or flatter slopes should be used.

4.8 Site Drainage and Grading

Given the good natural slope of the site, it is feasible to provide a simple but effective surface drainage scheme to collect surface water from rainfall off the site without difficulty. Effective drainage must be designed and constructed to drain off surface water rain-offs.

4.9 Corrosivity of Subsoils

The soil reaction with metals and other materials vary from weakly acids to weakly alkaline. pH values are in the range of .0 to 8.0. Moderate pH value of 6.7 was measured on vertosols within exchangeable sodium. Corrosivity occurs among other factors, which may include but not limited to high sodium concentration, soil moisture content, and availability of oxygen within pores. The visual survey of the area was generally show low soil moisture content. However, the soil is competent enough to withstand corrosivity; appropriate engineering practices must be put in place to protect any buried material from corroding.

5.0 SOURCES OF CONSTRUCTION MATERIALS

It must be pointed out that it may not be easy to find natural gravel material satisfying the specification as hardcore fill or base material; or in particularly, plasticity requirements of the material then a blended material may be employed. A mixture of 60% natural gravel with 40% crushed aggregate may provide a satisfactory solution at a relatively reasonable cost.

5.1 Re-Use of Excavated Material

Excavated material at the site is suitable as backfill or fill for foundation construction purposes. Placement of imported gravel material as hardcore filling may be carried out in loose lift thickness not exceeding 0.3m and compacted to 95% of maximum dry density (MDD) and at optimum moisture content (OMC).

5.2 Construction Monitoring

Although required excavations for the new construction are not that deep, it would be good to ensure that all surrounding buildings are carefully surveyed prior to start of construction and monitored periodically during construction. In addition, a crack survey of all surrounding buildings is recommended to be performed prior to construction, in order to forestall any unmerited future legal claims being placed on the client.

5.3 Geotechnical Construction Services

A review all project plans and specifications will be necessary to check that they conform to the intent of our recommendations. During construction, our field engineers should provide on-site observation and testing during installation of building foundations, shoring, earthworks, etc. These observations will allow us to compare actual with anticipated soil conditions and check to ensure that the contractor's work conforms to recommended geotechnical aspects for the project.

5.4 Construction Expedients.

Drains should not be laid too close and at a depth lower than the foundation footing. In addition, the bottom of any soak away facility in the vicinity of the structures should be well below the recommended foundation level. In addition, the structures should be with aprons at ground level to preclude the ingress of surface and ground water into the foundation.

Table 4: Relationship between Differential Swell and Degree of Expansiveness

Degree of Expansiveness	Differential Free Swell (DFS) (%)
Low	**Less than 20**
Moderate	**20-35**
High	**35-50**
Very High	**Greater than 50**

The test on the soil samples to assess their degree of expansiveness gave the following results 32.1 %. The potential of a soil to swell depends on factors like the difference between the moisture content at the site at the time of construction and moisture conditions that will materialize under conditions associated with the completed structure. If the moisture content during construction is considerably less than the moisture conditions under conditions associated with the completed structure, the soil will swell appreciably if the soil has a high swell potential. If on the other hand, the moisture content during construction is higher than the moisture conditions under associated with the completed structure, the soil will shrink. Analysis of results obtained, indicated that the foundation soils have a moderate potential to swell under moisture variations.

5.5 Swell Potential of Foundation Soil

Though the residual soils derived from the site are known to have a moderate volumetric change when subjected to changes in the moisture regime, the proposed foundation would have to be strengthened in order to prevent differential settlement.

6.0 SUMMARY AND RECOMMENDATIONS

After the review of the laboratory test results, field and site data, we recommend the following:

- The soil type overlaying this parcel of land is mainly consist of light brown / whitish red silty sandy GRAVEL, followed by dark greyish brown silty CLAY and underlain by light reddish brown silty sandy GRAVEL, with dense sandstone, of medium plasticity; hence, the effects of swell and uplift forces on the structure will be minimal.
- Due to the high bearing capacity of (490KN/m2) at the proposed project site, we strongly recommend that the depth of excavation should be 2.1m below existing ground level.
- A conventional isolated pad foundation can be adopted and be placed on a competent stratum for the footing anchorage base on the condition of the ground and the nature of the proposed structure.
- A gross allowable bearing capacity of 490KN/m2 may be applied for the design of the foundation substructure at the proposed site.
- Good compaction around the column with Lateritic gravel is encouraged.
- Precautions are to be taken in the design of the foundations and the building itself against earth or ground movement.
- Concrete for the substructure and superstructure should be properly designed to a dense consistency and be well placed.
- Tight construction control and the use of good constructional material must be adhered to strictly to prevent any damage to any of the structural members.
- Proper drainage system should be provided to prevent surface and underground water from penetrating into the foundation due to the nature of the soil beneath.
- In addition, with respect to ground water movement, it is therefore recommended that hardcore or waterproofing material should be utilized at the base level to prevent ingress of water into the substructure due to the swampy nature of the site.
- Adequate concrete cover should be provided to prevent the potential sulphate attack.
- Seismic effects should be considered in the design of the structure.

6.1 Disclaimer

The recommendations given in this report are based on conditions encountered during the field investigations. The investigations indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. They may not necessarily reflect strata variations that may exist between such locations. Subsurface conditions at other locations may differ from conditions occurring at these indicated locations.

The passage of time may result in a change in the conditions at these locations. If any variations in subsurface conditions from those described in this report are noted during construction, then, the recommendations in this report must be re-evaluated. There may also be special conditions prevailing at the site, though unlikely, which may not have been identified by the investigation. If such conditions are encountered, author of this report must be notified without delay.

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APPENDIX A

SITE PICTURES

THE DCP TEST

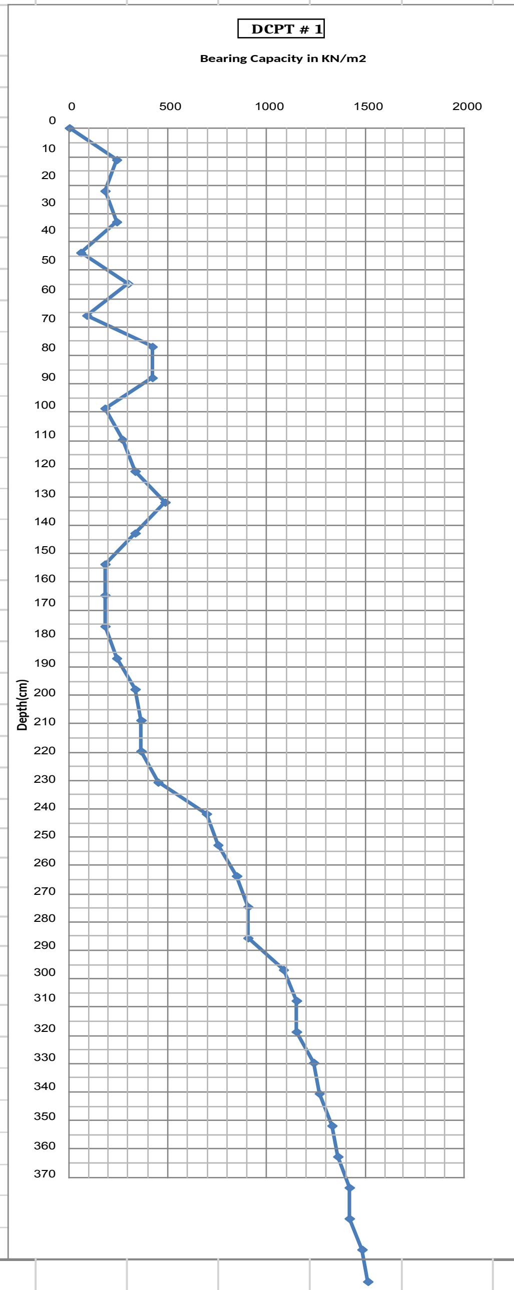
GOOGLE MAP LOCATION

PROPOSED DEVELOPMENT

APPENDIX B
DCPT LOGS

Project Title:	PROPOSED 5-STOREY SCHOOL	**Date**	26-Mar-22
Project Location:	ADWEMADOR	**Coordinates**	
Client:	ANSAH AGYEMANG BARIMAH		
Engineer:			
Site ID:			

DCPT 1		Penetration Index (DPI)	Bearing Capacit	Equivalent CBR
Depth (cm)	**Blows**	**mm/blow**	**kN/m²**	**%**
0	0	0.0	0	0
10	8	12.5	240	17
20	6	16.7	180	13
30	8	12.5	240	17
40	2	50.0	60	4
50	10	10.0	300	22
60	3	33.3	90	6
70	14	7.1	420	32
80	14	7.1	420	32
90	6	16.7	180	13
100	9	11.1	270	20
110	11	9.1	330	25
120	16	6.3	480	37
130	11	9.1	330	25
140	6	16.7	180	13
150	6	16.7	180	13
160	6	16.7	180	13
170	8	12.5	240	17
180	11	9.1	330	25
190	12	8.3	360	27
200	12	8.3	360	27
210	15	6.7	450	35
220	23	4.3	690	56
230	25	4.0	750	62
240	28	3.6	840	70
250	30	3.3	900	76
260	30	3.3	900	76
270	36	2.8	1080	93
280	38	2.6	1140	99
290	38	2.6	1140	99
300	41	2.4	1230	108
310	42	2.4	1260	111
320	44	2.3	1320	116
330	45	2.2	1350	119
340	47	2.1	1410	125
350	47	2.1	1410	125
360	49	2.0	1470	131
370	50	2.0	1500	134



C orrelation Between DCP and CBR

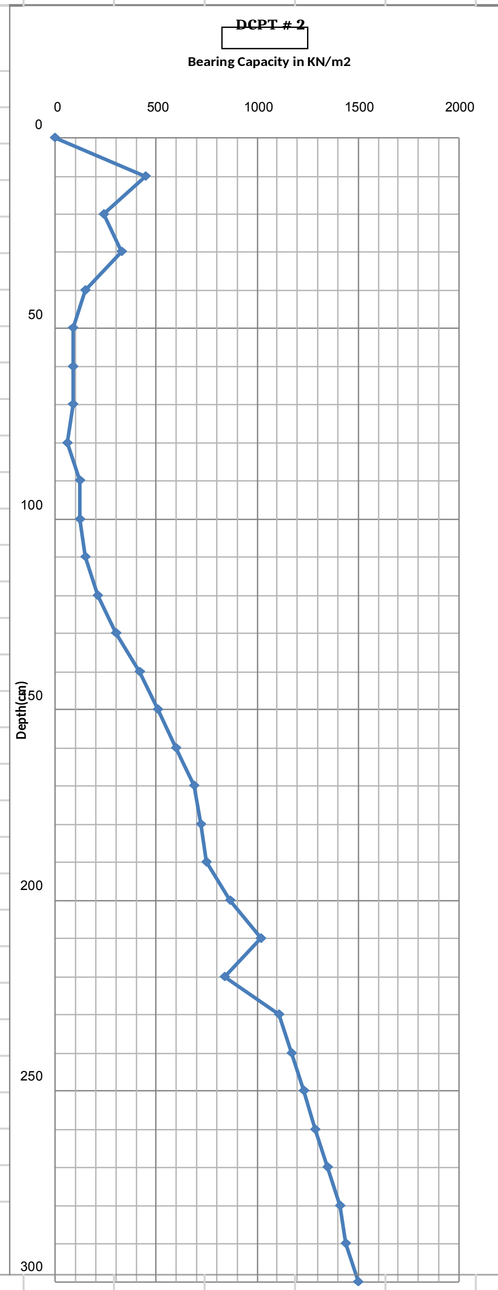
Log (CBR) = 2.465 - 1.12Log (DPI)

simplified: **CBR =** $\frac{292}{DPI^{1.12}}$

APPROVED BY:

Project Title:	PROPOSED 5-STOREY SCHOOL BUILDING	**Date**	26-Mar-22
Project Location:	ADWENADOR	**Project No.**	
Client:	ANSAH AGYEMANG BARIMAH	**Coordinates**	
Engineer:			
Site ID:			

DCPT 2		Penetration Index (DPI)	Bearing Capacity	Equivalent CBR
Depth (cm)	**Blows**	**mm/blow**	**kN/m²**	**%**
0	0	0.0	0	0
10	15	6.7	450	35
20	8	12.5	240	17
30	11	9.1	330	25
40	5	20.0	150	10
50	3	33.3	90	6
60	3	33.3	90	6
70	3	33.3	90	6
80	2	50.0	60	4
90	4	25.0	120	8
100	4	25.0	120	8
110	5	20.0	150	10
120	7	14.3	210	15
130	10	10.0	300	22
140	14	7.1	420	32
150	17	5.9	510	40
160	20	5.0	600	48
170	23	4.3	690	56
180	24	4.2	720	59
190	25	4.0	750	62
200	29	3.4	870	73
210	34	2.9	1020	87
220	28	3.6	840	70
230	37	2.7	1110	96
240	39	2.6	1170	102
250	41	2.4	1230	108
260	43	2.3	1290	113
270	45	2.2	1350	119
280	47	2.1	1410	125
290	48	2.1	1440	128
300	50	2.0	1500	134



C orrelation Between DCP and CBR

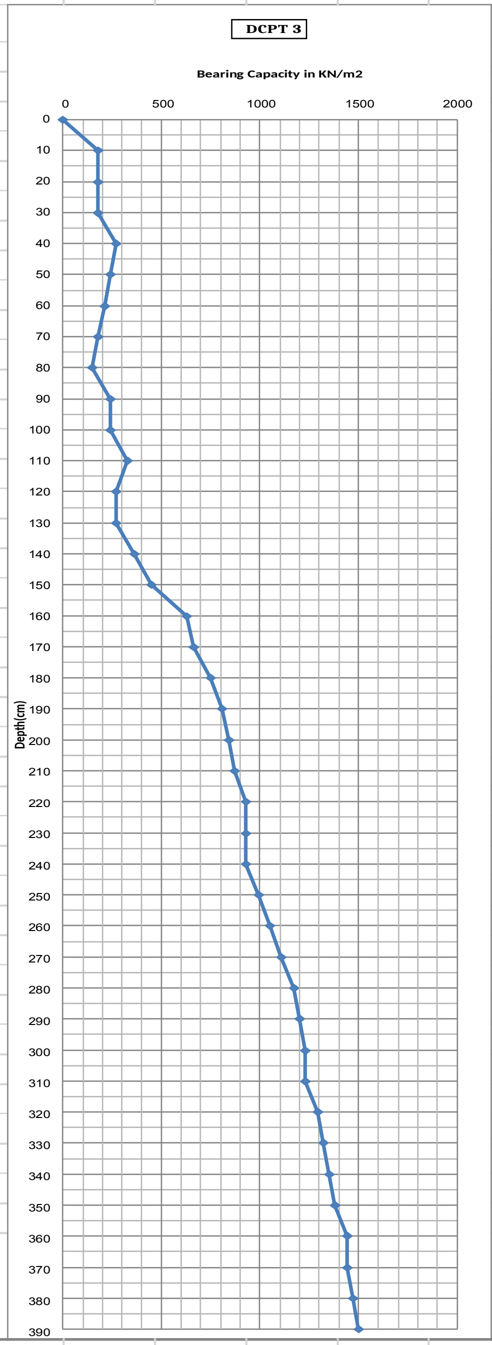
Log (CBR) = 2.465 - 1.12Log (DPI)

simplified: **CBR =** $\frac{292}{DPI^{1.12}}$

A PPROVED BY:

Project Title:	PROPOSED 5-STOREY SCHOOL BUILDING	**Date**	26-Mar-22
Project Location:	ADWEMADOR	**Project No.**	0
Client:	ANSAH AGYEMANG BARIMAH	**Coordinates**	
Engineer:			
Site ID:			

DCPT 3		Penetration Index (DPI)	Bearing Capacity	Equivalent CBR
Depth (cm)	Blows	mm/blow	kN/m²	%
0	0	0.0	0	0
10	6	16.7	180	13
20	6	16.7	180	13
30	6	16.7	180	13
40	9	11.1	270	20
50	8	12.5	240	17
60	7	14.3	210	15
70	6	16.7	180	13
80	5	20.0	150	10
90	8	12.5	240	17
100	8	12.5	240	17
110	11	9.1	330	25
120	9	11.1	270	20
130	9	11.1	270	20
140	12	8.3	360	27
150	15	6.7	450	35
160	21	4.8	630	51
170	22	4.5	660	54
180	25	4.0	750	62
190	27	3.7	810	67
200	28	3.6	840	70
210	29	3.4	870	73
220	31	3.2	930	79
230	31	3.2	930	79
240	31	3.2	930	79
250	33	3.0	990	84
260	35	2.9	1050	90
270	37	2.7	1110	96
280	39	2.6	1170	102
290	40	2.5	1200	105
300	41	2.4	1230	108
310	41	2.4	1230	108
320	43	2.3	1290	113
330	44	2.3	1320	116
340	45	2.2	1350	119
350	46	2.2	1380	122
360	48	2.1	1440	128
370	48	2.1	1440	128
380	49	2.0	1470	131
390	50	2.0	1500	134



Correlation Between DCP and CBR

Log (CBR) = 2.465 - 1.12Log (DPI)

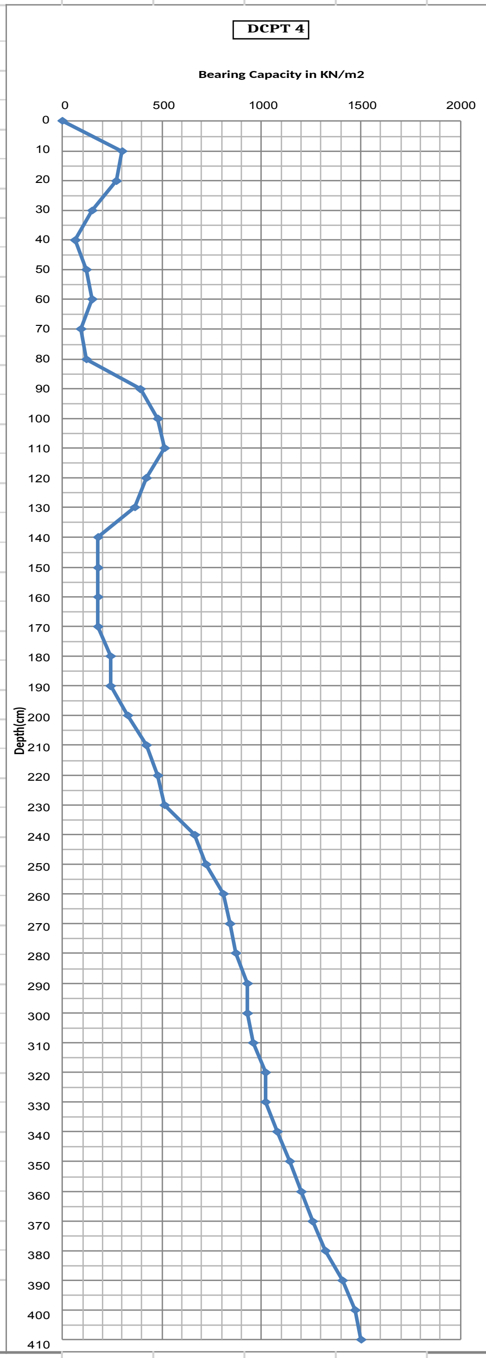
simplified: **CBR =** $\frac{292}{DPI^{1.12}}$

APPROVED BY:

Project Title:	PROPOSED 5-STOREY SCHOOL BUILDING	
Project Location:	ADWEMADOR	
Client:	ANSAH AGYEMANG BARIMAH	
Engineer:		
Site ID:		

Date	26-Mar-22
Project No.	0
Coordinates	

DCPT 4		Penetration Index (DPI)	Bearing Capacity	Equivalent CBR
Depth (cm)	**Blows**	**mm/blow**	**kN/m²**	**%**
0	0	0.0	0	0
10	10	10.0	300	22
20	9	11.1	270	20
30	5	20.0	150	10
40	2	50.0	60	4
50	4	25.0	120	8
60	5	20.0	150	10
70	3	33.3	90	6
80	4	25.0	120	8
90	13	7.7	390	30
100	16	6.3	480	37
110	17	5.9	510	40
120	14	7.1	420	32
130	12	8.3	360	27
140	6	16.7	180	13
150	6	16.7	180	13
160	6	16.7	180	13
170	6	16.7	180	13
180	8	12.5	240	17
190	8	12.5	240	17
200	11	9.1	330	25
210	14	7.1	420	32
220	16	6.3	480	37
230	17	5.9	510	40
240	22	4.5	660	54
250	24	4.2	720	59
260	27	3.7	810	67
270	28	3.6	840	70
280	29	3.4	870	73
290	31	3.2	930	79
300	31	3.2	930	79
310	32	3.1	960	81
320	34	2.9	1020	87
330	34	2.9	1020	87
340	36	2.8	1080	93
350	38	2.6	1140	99
360	40	2.5	1200	105
370	42	2.4	1260	111
380	44	2.3	1320	116
390	47	2.1	1410	125
400	49	2.0	1470	131
410	50	2.0	1500	134



C orrelation Between DCP and CBR

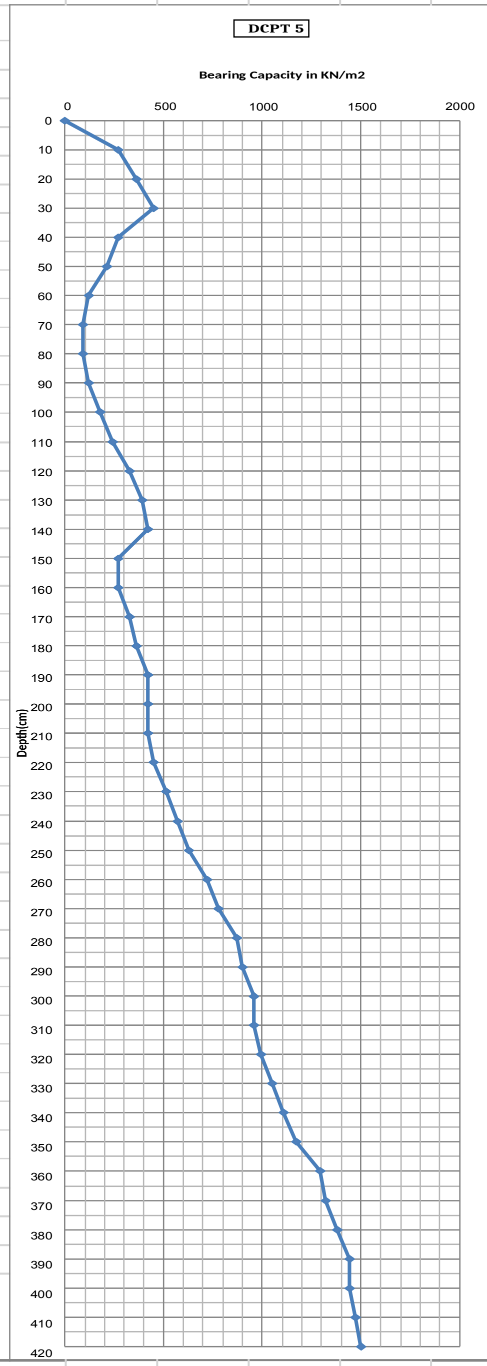
Log (CBR) = 2.465 - 1.12Log (DPI)

simplified: **CBR =** $\frac{292}{DPI^{1.12}}$

APPROVED BY:

Project Title:	PROPOSED 5-STOREY SCHOOL BUILDING	**Date**	26-Mar-22
Project Location:	ADWEMADOR	**Project No.**	0
Client:	ANSAH AGYEMANG BARIMAH	**Coordinates**	
Engineer:			
Site ID:			

DCPT 5		Penetration Index (DPI)	Bearing Capacity	Equivalent CBR
Depth (cm)	Blows	mm/blow	kN/m²	%
0	0	0.0	0	0
10	9	11.1	270	20
20	12	8.3	360	27
30	15	6.7	450	35
40	9	11.1	270	20
50	7	14.3	210	15
60	4	25.0	120	8
70	3	33.3	90	6
80	3	33.3	90	6
90	4	25.0	120	8
100	6	16.7	180	13
110	8	12.5	240	17
120	11	9.1	330	25
130	13	7.7	390	30
140	14	7.1	420	32
150	9	11.1	270	20
160	9	11.1	270	20
170	11	9.1	330	25
180	12	8.3	360	27
190	14	7.1	420	32
200	14	7.1	420	32
210	14	7.1	420	32
220	15	6.7	450	35
230	17	5.9	510	40
240	19	5.3	570	45
250	21	4.8	630	51
260	24	4.2	720	59
270	26	3.8	780	65
280	29	3.4	870	73
290	30	3.3	900	76
300	32	3.1	960	81
310	32	3.1	960	81
320	33	3.0	990	84
330	35	2.9	1050	90
340	37	2.7	1110	96
350	39	2.6	1170	102
360	43	2.3	1290	113
370	44	2.3	1320	116
380	46	2.2	1380	122
390	48	2.1	1440	128
400	48	2.1	1440	128
410	49	2.0	1470	131
420	50	2.0	1500	134



Correlation Between DCP and CBR

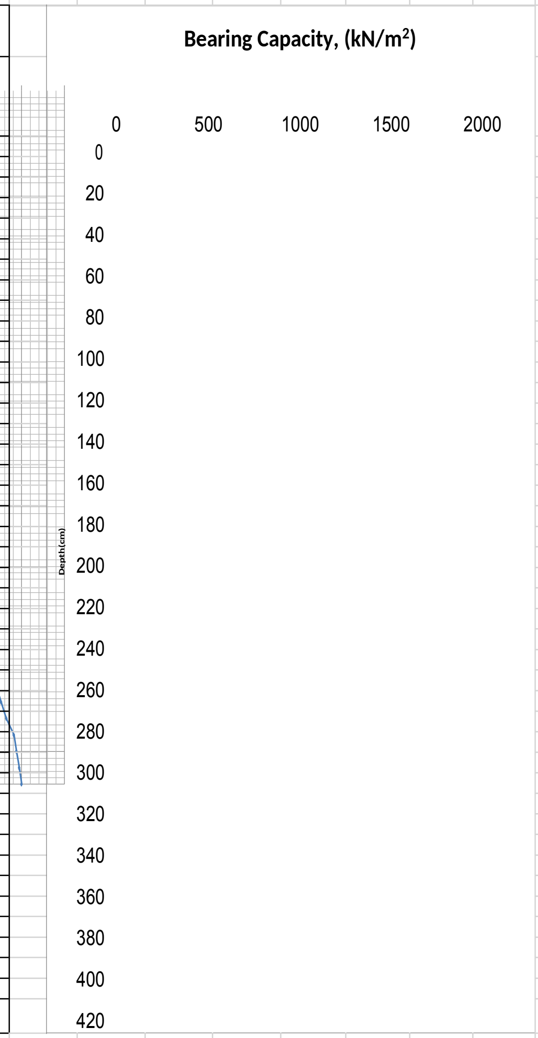
Log (CBR) = 2.465 - 1.12Log (DPI)

simplified: $\textbf{CBR} = \dfrac{292}{DPI^{1.12}}$

APPROVED BY:

Project Title:	PROPOSED 5-STOREYSCHOOL BUILDING	Date:	26/03/2022
Project Location	ADWEMADOR		
Client:	ANSAH AGYEMANG BARIMAH		
Architect:			
Site ID:		FOS: 3	

	DCPT 1	DCPT 2	DCPT 3	DCPT 4	DCPT 5	Ultimate Capacities	Allowable Capacities
Depth (cm)	Bearing Capacities (KPa)	Bearing Capacities (KPa)	Bearing Capacities (KPa)	Bearing Capacities (KPa)	Bearing Capacities (KPa)	Bearing Capacities (KPa)	Bearing Capacities (KPa)
0	0	0	0	0	0	0	0
10	240	450	180	300	270	180	60
20	180	240	180	270	360	180	60
30	240	330	180	150	450	150	50
40	60	150	270	60	270	60	20
50	300	90	240	120	210	90	30
60	90	90	210	150	120	90	30
70	420	90	180	90	90	90	30
80	420	60	150	120	90	60	20
90	180	120	240	390	120	120	40
100	270	120	240	480	180	120	40
110	330	150	330	510	240	150	50
120	480	210	270	420	330	210	70
130	330	300	270	360	390	270	90
140	180	420	360	180	420	180	60
150	180	510	450	180	270	180	60
160	180	600	630	180	270	180	60
170	240	690	660	180	330	180	60
180	330	720	750	240	360	240	80
190	360	750	810	240	420	240	80
200	360	870	840	330	420	330	110
210	450	1020	870	420	420	420	140
220	690	840	930	480	450	450	150
230	750	1110	930	510	510	510	170
240	840	1170	930	660	570	570	190
250	900	1230	990	720	630	630	210
260	900	1290	1050	810	720	720	240
270	1080	1350	1110	840	780	780	260
280	1140	1410	1170	870	870	870	290
290	1140	1440	1200	930	900	900	300
300	1230	1500	1230	930	960	930	310
310	1260		1230	960	960	960	320
320	1320		1290	1020	990	990	330
330	1350		1320	1020	1050	1020	340
340	1410		1350	1080	1110	1080	360
350	1410		1380	1140	1170	1140	380
360	1470		1440	1200	1290	1200	400
370	1500		1440	1260	1320	1260	420
380			1470	1320	1380	1320	440
390			1500	1410	1440	1410	470
400				1470	1440	1440	480
410				1500	1470	1470	490
420					1500	1500	500



Note:
* The ultimate bearing capacities indicated, are the minimum measured bearing capacities at any given depth.
** A minimum factor of safety (FOS) of 3.0 should be applied accordingly;
i.e the ultimate bearing capacities is divided by the FOS to give the allowable bearing capacities values
*** Final FOS to be used is to be determined by the design engineer

APPROVED BY:

APPENDIX C
TEST RESULTS

SOIL TEST SUMMARY RESULTS

geo-materiaux engineering ltd

GEO-MATERIAUX ENGINEERING LIMITED (GMEL LAB)

CLIENT: ANSAH AGYEMANG BARIMAH & RITA AGYEMANG BARIMAH

PROJECT: PROPOSED 5-STOREY SCHOOL PROJECT

DATE : 29 / 03 /2022

SAMPLE IDENTIFICATION	SAMPLE SOURCE	SAMPLE DESCRIPTION	DEPTH (m)	SIEVE ANALYSIS PERCENTAGE BY WEIGHT RETAIN ON BS SIEVE			ATTERBERG LIMITS				MOISTURE CONTENT
				GRAVEL %	SAND %	SILT / CLAY %	LL %	PL %	PI %	SWELL %	NMC %
				2mm - 75 mm	0.075mm - 2mm	0.002mm - ˂0.002mm	-	-	-		-
TRIAL PIT 3	1ST LAYER	Light reddish brown silty sandy gravel, mixed with broken concrete and household waste	0.0-0.5	50.7	32.1	17.2	22.5	17.6	4.9		6.7
	2ND LAYER	Dark greyish brown silty CLAY	0.5-3.2	8.6	28.3	63.1	47.2	32.6	14.6		15.8
	3RD LAYER	Light reddish brown silty sandy clay lateritic gravel	3.2-3.6	44.2	29.7	26.1	34.4	25.2	9.2	32.1	28.2

RANSFORD TETTEH
LABORATORY TECHNICIAN

WIREDU KWABENA ERIC
LABORATORY MANAGER / ENGINEER

GEO-MATERIAUX ENGINEERING LIMITED
LABORATORY MANAGER