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Chapter 01 General Description

1.1 INTRODUCTION

In civil engineering, subsoil investigation, also referred to as geotechnical investigation, plays a critical role in determining the soil's stratigraphy and pertinent physical properties beneath a specific location. The aim of this investigation is to ensure the safety and longevity of the substructure that will support the superstructure at that location. To accurately assess the characteristics of the soil strata that will impact the construction project, it is essential to conduct a thorough field survey and precise geotechnical investigation.

Geotechnical investigations offer engineers valuable insights into the subsurface conditions at the site, which are utilized to plan, design, and construct structures that are secure, functional, and durable. The accuracy and adequacy of these investigations are paramount to the project's success, as they directly influence the design, construction, project cost, and overall safety.

A comprehensive subsoil investigation encompasses both field and laboratory investigations. The investigations are conducted in accordance with relevant ASTM Standards and BNBC 2020, ensuring adherence to established protocols and guidelines.

1.2 SCOPE OF WORK

The subsoil investigation involves drilling and sampling activities in the field. Several boreholes are drilled to a depth and comprehensive data regarding the subsoil is recorded throughout the drilling process. This data includes soil descriptions, standard penetration test results, blow counts, water levels, and other relevant information.

Following the drilling and sampling, the obtained soil samples are carefully sealed, labeled, and transported to the laboratory in accordance with ASTM D4220 standards. Selected samples undergo various tests conducted in accordance with ASTM standards.

The final report encompasses several key components. Firstly, it presents the characteristics of the subsurface soil conditions through detailed soil profiles. Each soil layer is described in detail in accordance with USCS soil classification system. Then analysis and results of all the laboratory tests are presented with curves.

The report provides recommendations for bearing capacity for shallow foundation as well as deep foundation. Additionally, the report includes the assigned seismic design class, measurements of the groundwater table elevation obtained during the investigation.

Furthermore, the report provides geotechnical information required to assess potential geotechnical problems such as consolidation settlement, volumetric strain, etc. The report also assesses seismic hazards through soil liquefaction analysis.

Overall, the report provides a comprehensive understanding of the subsurface soil conditions, offers information required for foundation design, and addresses potential geotechnical challenges and seismic hazards identified during the investigation.

1.3 PROJECT DETAILS

Project Title	Consultancy Services For Detailed Feasibility Study For Construction Of 7.6 Mwp (Dc) Solar Photovoltaic Grid-Connected Power Plant At Kaptai, Rangamati, Bangladesh
Client Name	Bangladesh Power Development Board (Bpdb)
Location	KAPTAI, RANGAMATI, BANGLADESH
No of Boreholes	04 (Four)
Period of Investigation	November, 2023
Seismic Zone	III (According to BNBC 2020)
Seismic Intensity of Site	Severe
Seismic Zone Co-efficient	0.28
Site Classification	SD
Liquefaction Potential	Exists as per Idriss and Boulanger 2014 Method

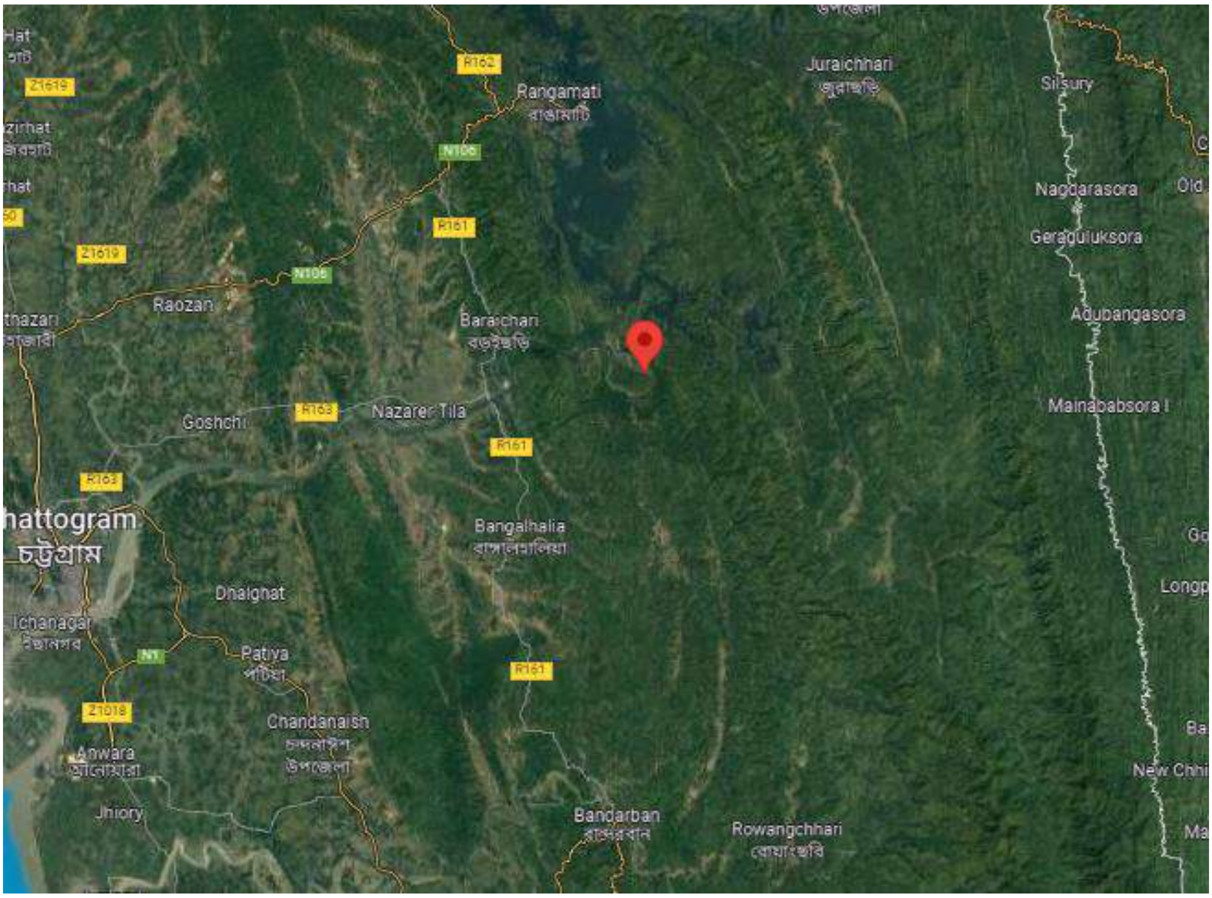


Fig 1.1 Location of Site on Map

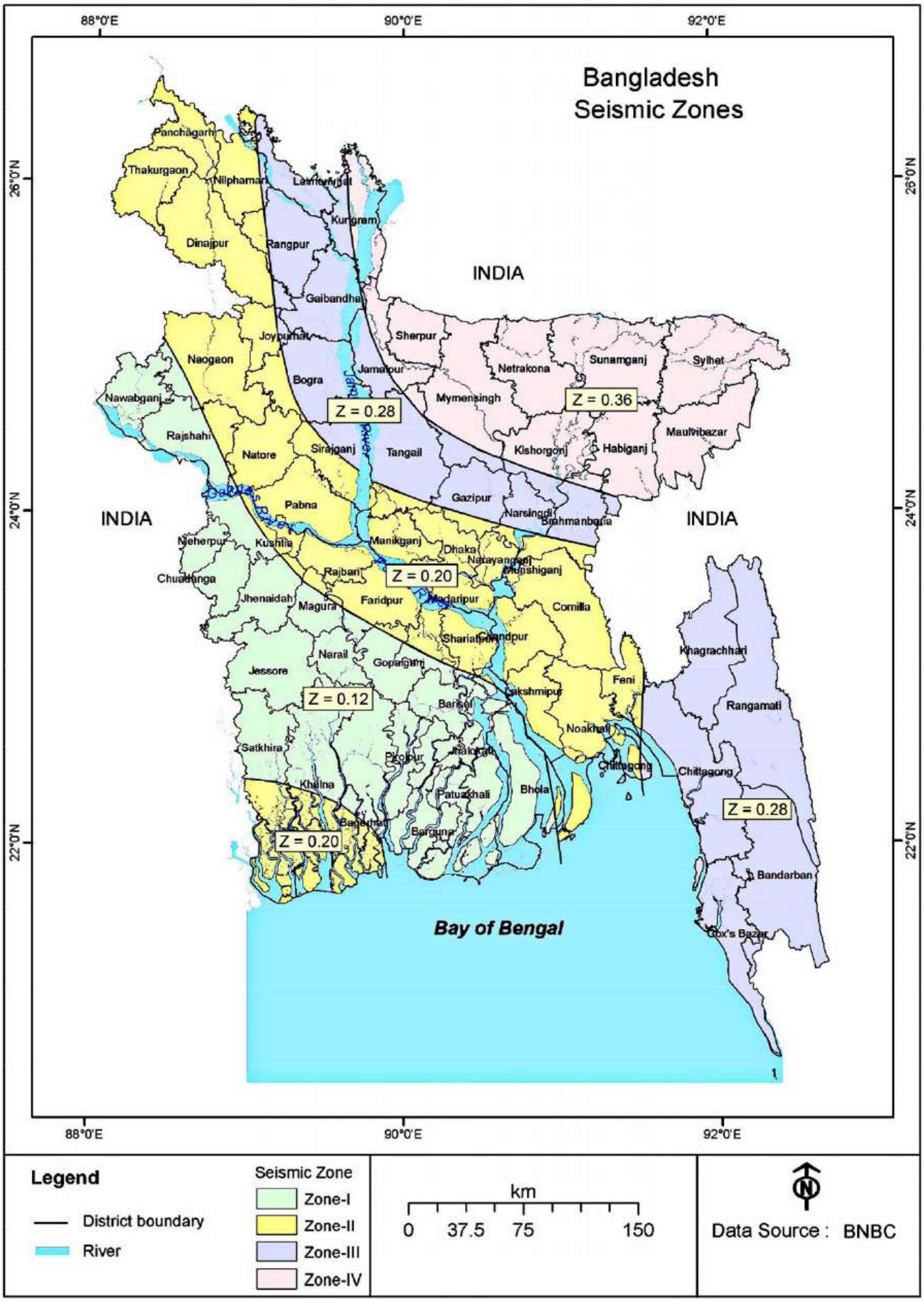


Fig 1.2: Seismic zoning map of Bangladesh (As Per BNBC 2020)

1.4 DETAILS OF BOREHOLES

Borehole No.	Depth of Borehole (m)	Reduced Level (R.L) (m)	Ground Water Level (m)	Date of Drilling
01	25.5	(-) 0.60	(-) 2.15	03-11-2023
02	15	(-) 0.65	(-) 4.25	05-11-2023
03	15	(+) 0.30	(-) 2.90	05-10-2023
04	15	(-) 0.60	(-) 2.15	06-11-2023

NOTE: T.B.M is taken from the center of the road, as ±0.

1.5 QUANTITY OF TESTS

Name of Test	Quantity (In Number)
Moisture Content Determination	47
Specific Gravity Determination	04
Atterberg Limit Test	12
Grain Size Analysis	16
Hydrometer Test	16
Direct Shear Test	02
Unconfined Compression Strength Test	04
Consolidation Test	02

Chapter 02 Geotechnical Investigation

2.1 FIELD INVESTIGATIONS

All the tests are executed as per standard procedure as in ASTM specification.

Exploratory Boring Drilling

Drilling is executed by wash boring method. A hole is started by driving vertically a 10 cm diameter steel casing into the ground to some depth and then the formation ground casing is broken up by repeated drops of a chopping bit attached to the lower end of the drilling pipe. The upper end of the same is forced at high pressure through the pressure pipe. Forced slurry or water emerges at high velocity through the pores of the chopping bit and returns to the surface through the annular space between the drilling pipe and the side of the casing or hole, carrying with it the broken-up soils. In this way, drilling is advanced up to a level of 15 cm above the depth where the Standard Penetration Test (SPT) has to be executed.

Standard Penetration Test

The Standard Penetration Test (SPT) is extensively utilized for determining the in-situ properties of soil. This test is particularly suitable for cohesionless soils, as there is now a well-established correlation between the SPT value and the soil's strength. The test procedures can be found in ASTM standards D1586, D1587, and D6066.

To conduct the test, a split spoon sampler is driven into the soil through a borehole with a diameter ranging from 55 to 100 mm, at the desired depth. A 63.5 kg (140 lb) hammer is dropped onto a drill rod from a height of 750 mm (30 inches). The number of blows, denoted as N, required to achieve a penetration of 300 mm (12 inches), is considered as the penetration resistance. The initial blows for the first 150 mm (6 inches) of penetration are excluded to avoid seating errors. The N-value is determined by counting the blows necessary to increase the penetration from 150 mm to 450 mm.

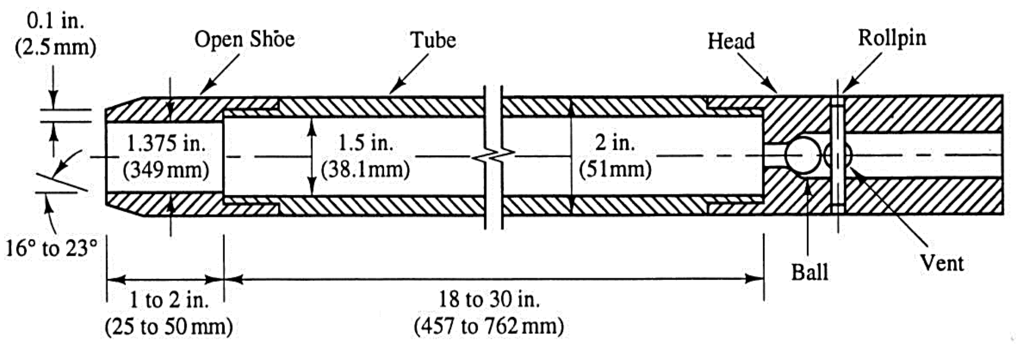


Fig 2.1 Schematic Diagram of Split Spoon Sampler

Several factors contribute to the variation of the standard penetration number N at a given depth for similar soil profiles. Among the factors are the SPT hammer efficiency, borehole diameter, sampling method, rod length, water table and overburden pressure important. The most two common types of SPT hammers used in the field are the safety hammer and donut hammer. Generally, SPT is conducted in 1.5 m interval. In hard formations, the testing is discontinued if N value is found to be over 50 in three continuous depths.

Extraction of Soil Sample

Soil samples are collected during sub-surface exploration to determine the engineering properties of the soil. Soil samples are generally into two categories:

Disturbed Samples (DS)

These are the samples in which the natural structure of the soil gets disturbed during sampling. However, these samples truly represent the composition and the mineral content of the soil. Disturbed samples can be used to determine the index properties of the soil such as grain size, plasticity characteristics, specific gravity etc.

The disturbed samples are collected with the help of split spoon sampler used during standard penetration test. Each sample is removed from the sampler in field, examined carefully and then classified by a technician. All the samples are duly preserved in air-tight bags, properly marked and then shifted to the laboratory for testing.

Undisturbed Samples (UD)

These are the samples in which the natural structure of the soil and the water content are retained. However, it may be mentioned that it is impossible to get truly undisturbed sample. Some disturbance is inevitable during sampling, even when the frantic care is taken. Undisturbed Samples are used for determining Shear Strength, Unit Weight, Void Ratio, Compression Index (C), Unconfined Compression Strength, Angle of Internal Friction (f) etc. The Undisturbed Samples of cohesive soils are obtained with the help of thin-walled Shelby Tubes of 3" (76mm) diameter. The samples recovered in the Shelby tubes are properly marked and wax sealed at both ends and then transmitted to the laboratory for test.

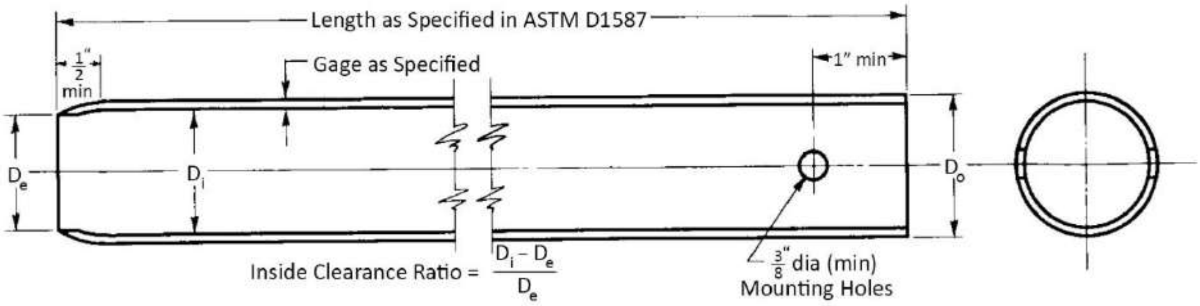


Fig 2.2 Schematic Diagram of Thin-walled Shelby Tube

Measuring Ground Water Table Location

Ground Water Levels along the project alignment are measured during drilling operation and 24 hours afterwards by rope or rod sounding. Specific ground water level readings are indicated on the Boring Logs. It is noted; however, ground water levels fluctuate seasonally, climatically and due to other factors not evident at the time of field explorations.

2.2 LABORATORY INVESTIGATIONS

All Laboratory tests are done on soil samples collected either in disturbed or undisturbed state as per ASTM procedures. The description of tests is as follows,

Moisture Content Determination

Most laboratory tests in soil mechanics require the determination of water content. Water content is defined as

$$\mathrm{w} = \frac{\text{weight of water present in a given soil}}{\text{mass weight of dry soil}}$$

Water content is usually expressed in percent. This test is performed as per ASTM D2216.

Specific Gravity Determination

The specific gravity of a given material is defined as the ratio of the weight of a given volume of the material to the weight of an equal volume of distilled water. Specific gravity G_s is defined as,

$$G_s = \frac{\text{unit weight (or density)of soil solids}}{\text{unit weight (or density)of water}}$$

For further details concerning the specific gravity test, see ASTM D854, "Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer."

Sieve Analysis

A basic element of a soil classification system is the determination of the amount and distribution of the particle sizes in the soil. The distribution of particle sizes larger than 0.075 mm (No. 200 sieve) is determined by sieving.

A sieve is a piece of laboratory equipment that consists of a pan with a screen at the bottom. Commonly used U.S. Standard Sieve numbers and their sieve opening are as follows:

Sieve No.	4	8	16	30	40	50	60	100	140	170	200
Sieve Opening (mm)	4.75	2.36	1.18	0.6	0.425	0.3	0.25	0.15	0.106	0.088	0.075

The laboratory test procedures for performing a sieve analysis are presented in ASTM D6913. The basic steps include first determining the initial dry mass of the soil (M). Then the soil is washed on the No. 200 sieve in order to remove all the fines (i.e., silt and clay size particles). The purpose of the washing of the soil on the No. 200 sieve is to ensure that all the fines and surface coatings are washed-off of the granular soil particles.

Hydrometer Test

A sedimentation process is used to determine the particle distribution for fines (i.e., silt and clay size particles finer than the No. 200 sieve). A hydrometer is used to obtain the necessary data during the sedimentation process. The hydrometer test is based on Stokes law, which relates the diameter of a single sphere to the time required for the sphere to fall a certain distance in a liquid of known viscosity.

The idea for the hydrometer test is that a larger, and hence heavier, soil particle will fall faster through distilled water than a smaller, and hence lighter, soil particle. The hydrometer test uses the diameter of an equivalent sphere as the definition of particle size. If the amount of fines (i.e., percent passing No. 200 sieve) is less than 5 percent, typically a hydrometer test is not performed.

Likewise, if the percent passing the No. 200 sieve is between 5 percent and 15 percent, the soil may be non-plastic and once again a hydrometer test may be unnecessary for classifying the soil. Usually if the percent passing the No. 200 sieve is greater than 15 percent, a hydrometer test could be performed. This test is performed as per ASTM D7928.

Atterberg Limit Test

The term plasticity is applied to silts and clays and indicates an ability to be rolled and molded without breaking apart. The Atterberg limits are defined as the water content corresponding to different behavior conditions of silts and clays. For laboratory testing details, please see ASTM D4318. The term Atterberg limits refers to the liquid limit (LL) and plastic limit (PL), defined as follows:

Liquid Limit (LL)

The water content corresponding to the behavior change between the liquid and plastic state of a silt or clay. The liquid limit is determined by spreading a pat of soil in a brass cup, dividing it in two by use of a grooving tool, and then allowing it to flow together from the shock caused by repeatedly dropping the cup in a standard liquid limit device.

In terms of specifics, the liquid limit is defined as the water content at which the pat of soil cut by the grooving tool will flow together for a distance of 0.5 in. (12.7 mm) under the impact of 25 blows in a standard liquid limit device.

Plastic Limit (PL)

The water content corresponding to the behavior change between the plastic and semi-solid state of a silt or clay. The plastic limit is determined by pressing together and rolling a small portion of the plastic soil into a thread approximately 1/8 in. (3.2 mm) in diameter so that its water content is slowly reduced with the end result that the thread of soil crumbles apart.

Direct Shear Test

The direct shear test is a laboratory or field test used by geotechnical engineers to measure the shear strength properties of soil material, or of discontinuities in soil or rock masses. The test is performed as per ASTM D3080.

The test is performed on three or four specimens from a relatively undisturbed soil sample. A specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample. A confining stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails, or through a specified strain. The load applied and the strain induced is recorded at frequent intervals to determine a stress–strain curve for each confining stress. Several specimens are tested at varying confining stresses to determine the shear strength parameters, the soil cohesion (c) and the angle of internal friction, commonly known as friction angle (Ø) .

The results of the tests on each specimen are plotted on a graph with the peak (or residual) stress on the y-axis and the confining stress on the x-axis. The y-intercept of the curve which fits the test results is the cohesion, and the slope of the line or curve is the friction angle. Direct shear tests can be performed under several conditions. The sample is normally saturated before the test is run, but can be run at the in-situ moisture content.

The rate of strain can be varied to create a test of undrained or drained conditions, depending whether the strain is applied slowly enough for water in the sample to prevent pore-water pressure buildup. Direct shear test machine is required to perform the test. The test using the direct shear machine determinates the consolidated drained shear strength of a soil material in direct shear. The advantages of the direct shear test over other shear tests are the simplicity of setup and equipment used, and the ability to test under differing saturation, drainage, and consolidation conditions.

Unconfined Compression Strength Test

This test is performed as per ASTM D2166 - Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions.

According to the ASTM standard, the unconfined compressive strength is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test.

For soils, the undrained shear strength is necessary for the determination of the bearing capacity of foundations. The undrained shear strength of clays is commonly determined from an unconfined compression test. The undrained shear strength of a cohesive soil is equal to one-half the unconfined compressive strength when the soil is under the f = 0 condition (f = the angle of internal friction).

Consolidation Test

This test is performed as per ASTM D2435 - Standard Test Method for One-Dimensional Consolidation Properties of Soils. This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve can be plotted. This data is useful in determining the compression index, the recompression index and the pre-consolidation pressure (or maximum past pressure) of the soil.

The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of a structure or an earth fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.

Standards Followed for Laboratory Tests

All the tests are done as per standard procedure as in ASTM specification.

LABORATORY TESTS	SPECIFICATION
WATER CONTENT TEST (MOISTURE CONTENT)	ASTM D2216
SPECIFIC GRAVITY TEST	ASTM D854
SIEVE ANALYSIS	ASTM D6913
HYDROMETER TEST	ASTM D7928
ATTERBERG LIMITS TEST	ASTM D4318
DIRECT SHEAR TEST	ASTM D3080
UNCONFINED COMPRESSION STRENGTH TEST	ASTM D2166
CONSOLIDATION TEST	ASTM D2435

2.3 SOIL CLASSIFICATION

Soils are widely varied in their grain-size distribution. Various types of engineering works require the identification and classification of soil in the field.

For engineering purposes, there are two major systems that are presently used in the United States. They are: (i) American Association of State Highway and Transportation Officials (AASHTO) Classification System and (ii) Unified Soil Classification System.

Unified Soil Classification System

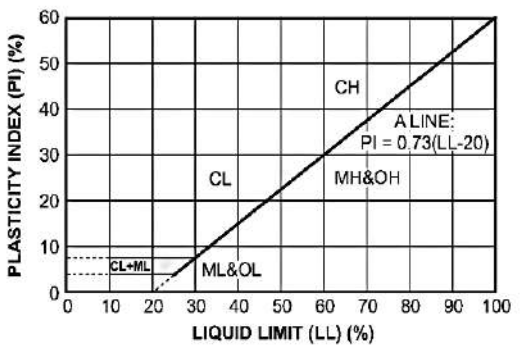
This classification system was first developed in 1942 by Arthur Casagrande for airfield construction during World War **II.** More recently, the American Society of Testing and Materials (ASTM) introduced a more definite system for group name of soils. Unlike the AASHTO system, the Unified system uses symbols to represent the soil types and the index Properties of the soil.

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART				LABORATORY CLASSIFICATION CRITERIA		
COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)						
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)					
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines		GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines		GP	Not meeting all gradation requirements for GW	
	Gravels with fines (More than 12% fines)					
	GM	Silty gravels, gravel-sand-silt mixtures		GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
	GC	Clayey gravels, gravel-sand-clay mixtures		GC	Atterberg limits above "A" line with P.I. greater than 7	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)					
	SW	Well-graded sands, gravelly sands, little or no fines		SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
	SP	Poorly graded sands, gravelly sands, little or no fines		SP	Not meeting all gradation requirements for GW	
	Sands with fines (More than 12% fines)					
	SM	Silty sands, sand-silt mixtures		SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
	SC	Clayey sands, sand-clay mixtures		SC	Atterberg limits above "A" line with P.I. greater than 7	
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)						
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity				
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays				
	OL	Organic silts and organic silty clays of low plasticity				
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts				
	CH	Inorganic clays of high plasticity, fat clays				
	OH	Organic clays of medium to high plasticity, organic silts				
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils				

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent	GW, GP, SW, SP
More than 12 percent	GM, GC, SM, SC
5 to 12 percent	Borderline cases requiring dual symbols

PLASTICITY CHART



Grain Size Classification of Soil

Particle size, also called grain size, refers to the diameter of individual grains of sediment, or the lithified particles in clastic rocks. The term may also be applied to other granular materials. This is different from the crystallite size, which refers to the size of a single crystal inside a particle or grain. A single grain can be composed of several crystals. Granular material can range from very small colloidal particles, through clay, silt, sand, gravel, and cobbles to boulders.

DIVISIONS	PARTICLE SIZE
Boulders	Retained on 300 mm sieve
Cobbles	Pass 300 mm sieve, retained on 75 mm sieve
Gravels	Pass 75 mm, retained on 4.75 mm
Sand	Pass 4.75 mm, retained on 0.075 mm
Silt	From 0.074 mm to 0.005 mm
Clay	Less than 0.005 mm

2.4 LIMITATIONS

While interpreting and subsequently ascertaining the aforesaid test results, observations and bearing capacity obtained thereof, due considerations should be attributed towards capriciousness of the soil properties, restricted number of bore holes and their locations, some limitations and constraints usually associated with such sub soil exploration work and some simplifying assumptions made during subsequent analysis. Some of the limitations are as follows,

1. Amount of soil samples collected from site was not sufficient to conduct all the tests at every 1.5 m interval.
2. In grain size analysis, there might be few errors. These errors may be occurred due to improper pulverization, washing or grinding of the samples.
3. In grain size analysis, if amount of soil samples retained on the pan from sieve shaking were found to be less than 12%, the hydrometer test is avoided. But when wash sieve method is used, some percentage of soil could be added to fine grained portion.
4. Where tests were not possible to conduct, soil parameters were calculated from correlations of laureate and well-established theories for estimating foundation capacities.
5. The thin-walled Shelby Tubes of 3" (76mm) diameter was penetrated in to the undisturbed soil formation by applying rapid but continuous force. Several undisturbed samples were collected in the Shelby tube. Unconfined compression strength tests and Consolidation tests were performed on those samples.

Chapter 03 Geotechnical Principles of Analysis

3.1 EVALUATION OF FOUNDATION CAPACITY

BEARING CAPACITY ANALYSIS FOR SHALLOW FOUNDATION

There are many established bearing capacity equations from different authors which shall be used for calculating bearing capacity. The conventional method of designing foundations is based on the concept of bearing capacity. One meaning of the verb to bear is to support or hold up.

Generally, bearing capacity refers to the ability of a soil to support or hold up a foundation and structure. The ultimate bearing capacity of a soil refers to the loading per unit area that will just cause shear failure in the soil. It is given the symbol Q_{ult}. The allowable bearing capacity (symbol Q_{all}) refers to the loading per unit area that the soil is able to support without unsafe movement. It is the "design" bearing capacity.

The allowable load is equal to allowable bearing capacity multiplied by area of contact between foundation and soil. The allowable bearing capacity is equal to the ultimate bearing capacity divided by the factor of safety. A factor of safety of 2.5 to 3 is commonly applied to the value of Q_{ult}. Care must be taken to ensure that a footing design is safe with regard to

(1) Foundation failure (collapse) and

(2) Excessive settlement.

The basic principles governing bearing capacity theory as developed by Terzaghi (Terzaghi and Peck, 1967) can be better followed by referring to Fig-3.1. As load (Q) is applied, the footing undergoes a certain amount of settlement as it is pushed downward, and a wedge of soil directly below the footing's base moves downward with the footing. The soil's downward movement is resisted by shear resistance of the foundation soil along slip surfaces cde and cfg and by the weight of the soil in sliding wedges acfg and bcde. For each set of assumed slip surfaces, the corresponding load Q that would cause failure can be determined. The set of slip surfaces giving the least applied load Q (that would cause failure) is the most critical; hence, the soil's ultimate bearing capacity (q) is equal to the least load divided by the footing's area.

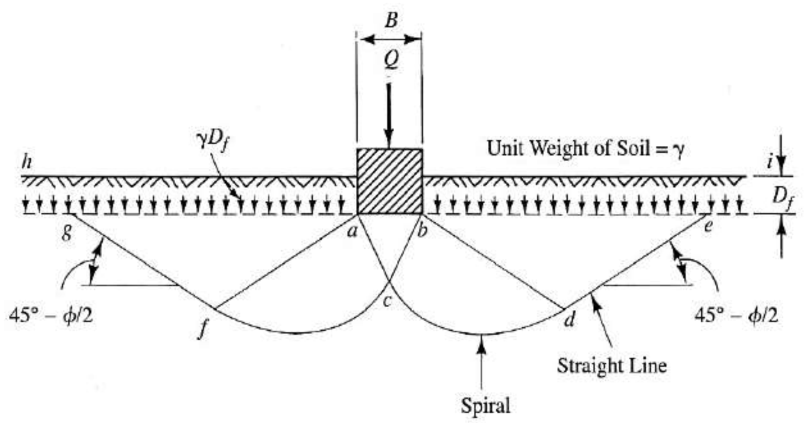


Fig 3.1: Bearing capacity failure in soil under a rough rigid foundation

TERZAGHI'S THEORY:

Terzaghi assumed the method of superposition to be valid presented the unit ultimate bearing capacity in the form

$$Q_{ult} = cN_c + qN_q + \frac{1}{2}\gamma B\gamma N_\gamma$$

Here,

Q_{ult} = Ultimate bearing capacity
c = Cohesion of Soil
N_c, N_q, N_γ = Terzaghi's bearing capacity factors
γ = Effective unit weight of soil
B = Width of footing

Values of the Terzaghi dimensionless bearing capacity factors for different values of φ can be obtained from these equations:

$$N_q = e^{\pi \tan\varphi} \tan^2(45^0 + \frac{\phi}{2})$$
$$N_c = \cot\phi \; (N_q - 1)$$
$$N_\gamma = (N_q - 1) \; \tan(1.4\,\phi)$$

Terzaghi derived the equation for ultimate bearing capacity of strip footing in which the case the problem is essentially two-dimensional. But in the case of square or circular footing the problem becomes three-dimensional and more complicated from mathematical point of view. In the absence of rigorous theoretical analysis, Terzaghi suggested:

For square footing

$$Q_{ult} = 1.3 * c * N_c + q * N_q + 0.4 * B * \gamma * N_\gamma$$

For circular footing

$$Q_{ult} = 1.3 * c * N_c + q * N_q + 0.3 * B * \gamma * N_\gamma$$

For quite some time the equation obtained for strip footing was used in the case of rectangular footing. Later rectangular footing was distinguished from strip footing as one for which L≥B and the following equation was suggested:

$$Q_{ult} = \left(1 + \frac{0.3B}{L}\right) * c * N_c + q * N_q + \left(1 - \frac{0.2B}{L}\right) * 0.5 * B * \gamma * N_\gamma$$

Limitations in Terzaghi's analysis:

1. Terzaghi's analysis assumes the plastic zones develop fully before failure occurs. This is true only in the case of dense cohesion less soils and stiff cohesive soils.
2. The value of Phi is assumed to remain constant. But F can change as soil gets compressed.
3. The failure zones are assumed not to extend above the base level of footing. Thus, the shearing resistance of soil surrounding it above its base level is neglected. The error due to this assumption increases as the depth of footing is increased.
4. The load is assumed to be vertical and acting concentrically with uniform pressure distribution at the base.

MEYERHOF'S THEORY:

Meyerhof considered the effects of shearing resistance within the soil above foundation level, the shape and roughness of foundation, and derived a general bearing capacity equation.

According to Meyerhof,

For vertical Load:

$$Q_{ult} = cN_c s_c d_c + qN_q s_q d_q + \frac{1}{2}\gamma B\gamma N_\gamma s_\gamma d_\gamma$$

For inclined load:

$$Q_{ult} = cN_c s_c d_c i_c + qN_q s_q d_q i_q + \frac{1}{2}\gamma B\gamma N_\gamma s_\gamma d_\gamma i_\gamma$$

Here,
Q_{ult} = Ultimate bearing capacity
c = Cohesion of Soil
N_c, N_q, N_γ = Meyerhof's bearing capacity factors
γ = Effective unit weight of soil
B = Width of footing
s, d, i = shape, depth and inclination factors

For any ϕ,
$s_c = 1 + 0.2K_P\frac{B}{L}$,
$d_c = 1 + 0.2\sqrt{K_P}\frac{D}{B}$,
$i_c = i_q = (1 - \frac{\alpha}{90^0})^2$.

For $\phi > 10^0$,
$s_q = s_\gamma = 1 + 0.1K_p\frac{B}{L}$,
$d_q = d_\gamma = 1 + 0.1\sqrt{K_P}\frac{D}{B}$,
$i_q = (1 - \frac{\alpha}{90^0})^2$.

For $\phi = 0^0$,
$s_q = s_\gamma = 1$,
$d_q = d_\gamma = 1$,
$i_\gamma = 0$.

Here,
$K_p = \tan^2\left(45^0 + \frac{\phi}{2}\right)$
α = angle of resultant measured from vertical axis.

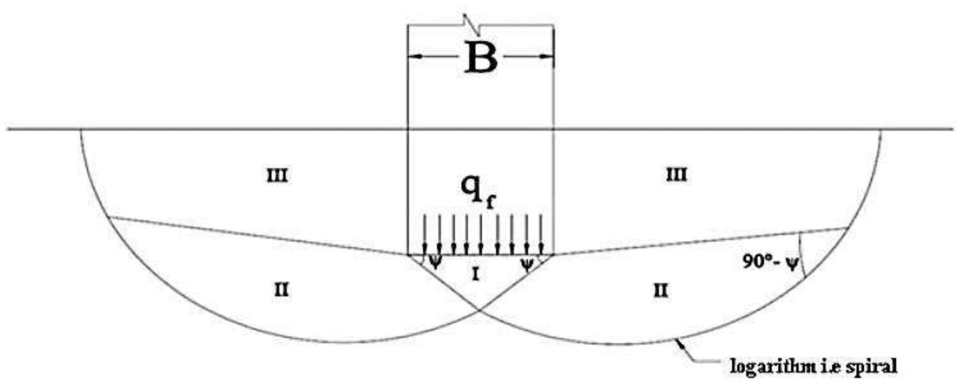


Fig 3.2: Failure zones assumed in Meyerhof's theory

HANSEN'S THEORY:

J. Brinch Hansen (1970) proposed what is referred to as general bearing capacity equation.

$$Q_{ult} = cN_c s_c d_c i_c g_c b_c + qN_q s_q d_q i_q g_q b_q + \frac{1}{2}\gamma B_\gamma N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

where,

$\mathrm{s_c} = 0.2 * \frac{\mathrm{B}}{\mathrm{L}}$, when ϕ = 0

$\mathrm{s_c} = 1 + \frac{\mathrm{N_q}*B}{\mathrm{N_c}*L}$, when ϕ >0

S_q = 1 + (B/L) tanϕ

$\mathrm{S_\gamma} = 1 - 0.4 * \frac{\mathrm{B}}{\mathrm{L}}$

$\mathrm{d_c} = 0.4 * \frac{\mathrm{D}}{\mathrm{B}}$, for ϕ = 0 and D/B ≤ 1

$\mathrm{d_c} = 0.4 * \tan^{-1} D/B$ for ϕ = 0 and D/B> 1

$\mathrm{d_c} = 1 + 0.4 * \frac{\mathrm{D}}{\mathrm{B}}$ for ϕ > 0 and D/B ≤ 1

$\mathrm{d_c} = 1 + 0.4 * \tan^{-1} D/B$ for ϕ > 0 and D/B> 1

$\mathrm{d_q} = 1 + 2\, tan\phi * (1 - sin\phi)^{\,2}$* (D/B) for D/B ≤ 1

$\mathrm{d_q} = 1 + 2\, tan\phi * (1 - sin\phi)^{\,2} * \tan^{-1} D/B$ for D/B>1

To a great extent, Hansen's work is an extension of Meyerhof's analysis, as is evident from comparison between the two equations. To include conditions for footing on slope Hansen has introduced two additional factors viz., the ground factors and base factors.

BEARING CAPACITY FROM SPT FOR GRANULAR SOIL:

The SPT is widely used to obtain the bearing capacity of soils directly. One of the earliest published relationships was that of Terzaghi and Peck (1967). This has been widely used, but an accumulation of field observations has shown these curves to be overly conservative. Meyerhof (1956, 1974) published equations for computing the allowable bearing capacity for a 25-mm settlement. These could be used to produce curves similar to those of Terzaghi and Peck and thus were also very conservative. Considering the accumulation of field observations and the stated opinions of the authors and others, this author adjusted the Meyerhof equations for an approximate 50 percent increase in allowable bearing capacity to obtain the following:

$$\mathrm{q_a} = \frac{N}{F_1} K_d \qquad B \leq F_4$$

$$\mathrm{q_a} = \frac{N}{F_2}\left(\frac{B + F_3}{B}\right) K_d \qquad B > F_4$$

Where $\mathrm{q_a}$ = allowable bearing pressure for ΔH_0 = 25-mm or 1-in. settlement, kPa or ksf

$K_d = 1 + 0.33\, \frac{D}{B} \leq 1.33$ [as suggested by Meyerhof (1965)]

F factor as follows:

	N_{55}		N_{70}	
	SI	Fps	SI	Fps
F_1	0.05	2.5	0.04	2.0
F_2	0.08	4	0.06	3.2
F_3	0.30	1	Same	Same
F_4	1.20	4		

METHOD	BEST FOR
Terzaghi	Very cohesive soils where $D/B \leq 1$ or for a quick estimate of Q_{ult} to compare with other methods. Do not use for footings with moments and/or horizontal forces or for tilted bases and/or sloping ground.
Hansen, Meyerhof, Vesić	Any situation that applies, depending on user preference or familiarity with a particular method.
Hansen, Vesić	When base is tilted; when footing is on a slope or when D/B> 1.

3.2 ULTIMATE GEOTECHNICAL CAPCITY OF PILE

The ultimate geotechnical capacity, ($Q_{ultimate}$) of a pile consists of skin friction or shaft friction or side shear, (Q_s) and end bearing at the base or tip of the pile, (Q_b).

So, the Ultimate Geotechnical Capacity (axial) of a pile is

$$Q_{ultimate} = Q_s + Q_b$$

Here,
$Q_{ultimate}$ = Ultimate Geotechnical Capacity (axial) of a pile
Q_s = Skin Friction
Q_b = End Bearing

The skin friction, Q_s and end bearing, Q_b can be calculated as:

$$Q_s = A_s f_s$$
$$Q_b = A_b f_b$$

A_s = skin friction area (perimeter area) of the pile = Perimeter × Length
f_s = skin frictional resistance on unit surface area.
A_b = end bearing area of the pile = Cross-sectional area of pile tip (bottom)
f_b = end bearing resistance on unit tip area of pile.

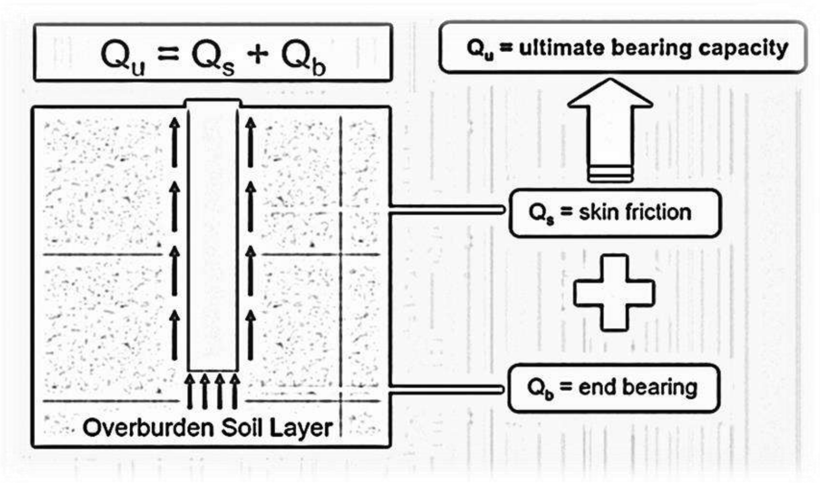


Fig 3.3: Ultimate Bearing Capacity of Pile

ESTIMATION OF ULTIMATE GEOTECHNICAL CAPACITY OF BORED PILE

For Cohesive Soil

A modified Terzaghi bearing capacity equation is used to find the pile capacity. The α-method that is based on total stress analysis and is normally used to estimate the short-term load capacity of piles embedded in fine grained soils. In this method, a coefficient a is used to relate the undrained shear strength "*c*" to the adhesive stress (f_s) along the pile shaft. As such,

$$f_s = \alpha * c$$

Here,
α = adhesion factor between pile and soil
c = cohesion of the soil
Values of α varies with cohesion of the soil.

So, Skin Friction of Pile for Cohesive soil, $Q_s = A_s * \alpha * c$

The end bearing in such a case is found by analogy with shallow foundations and is expressed as:

$$Q_b = c_b * (N_c)_b * A_b$$

N_c is a bearing capacity factor and for deep foundation the value is usually 9. "c" is the undrained shear strength of soil at the base of the pile. The suffix b's are indicatives of base of pile.

The general equation for $N_c$is, however, as follows.

$$N_c = 6\left[1 + 0.2\left(\frac{L}{D_b}\right)\right] \leq 9$$

So, End Bearing of Pile for Cohesive soil, $Q_b = 9 * c * A_b$
Ultimate Pile Capacity in Cohesive soil,
$Q_{ultimate} = A_s * \alpha * c + 9 * c * A_b$

For Cohesionless Soil

A modified version of the Terzaghi bearing capacity equation is widely used for pile design. The third term or the density term in the Terzaghi bearing capacity equation is negligible in piles and hence usually ignored. The lateral earth pressure coefficient (K) is introduced to compute the skin friction of piles. The β -method is based on an effective stress analysis and is used to determine both the short term and long-term pile load capacities.

As such,

$$f_s = K * \sigma'_z * \tan\delta$$

Here,
K =lateral earth pressure coefficient.
σ'_z = effective stress at the perimeter of the pile. (Varies with the depth. Usually, value at the midpoint of the pile is obtained).
δ = friction angle between pile and soil.
So, Skin Friction for Cohesion less soil is

$$Q_s = K * \sigma'_z * \tan\delta * A_S$$

The base resistance for Cohesion less soil is obtained from the following equation:

$$Q_b = A_b f_b = A_b * \sigma_t * N_q$$

Here,
σ_t = effective stress at the tip of the pile
N_q = bearing capacity coefficient

So, The Ultimate Geotechnical Capacity of Pile for Cohesion less soil is

$$Q_{ultimate} = K * \sigma'_z * \tan\delta * A_S + A_b * \sigma_t * N_q$$

According to the Nordlund Method, the ultimate capacity of a pile in cohesion less soil is the sum of the shaft resistance and the toe resistance. These piles, which were used to develop the method's design curves, had pile widths generally in the range of 250 to 500 mm (10 to 20inches). The Nordlund Method tends to overpredict pile capacity for piles with widths larger than 600 mm (24 inches).

Nordlund suggests the shaft resistance is a function of the following variables:
1. The friction angle of the soil.
2. The friction angle on the sliding surface.
3. The taper of the pile.
4. The effective unit weight of the soil.
5. The pile length.
6. The minimum pile perimeter.
7. The volume of soil displaced.

The Nordlund Method equation for computing the ultimate capacity of a pile is as follows:

$$Q_{ult} = \sum_{d=0}^{d=D} K_\delta * C_F * \sigma'_z * \frac{\sin(\delta + \omega)}{\cos \omega} * A_s + \alpha_t * N_q * A_b * \sigma_t$$

Here,
d = Depth
D = Embedded pile length.
K_δ = Coefficient of lateral earth pressure at depth d.
C_F = Co-relation factor for K_δ when $\delta \neq \varphi$
ω = Angle of pile taper from vertical
α_t = Dimensionless factor (dependent on pile depth-width relationship).

ESTIMATION OF ULTIMATE GEOTECHNICAL CAPACITY OF DRILLED SHAFT

For Cohesive Soil

Skin friction resistance in cohesive soil according to α-method is:

$$f_s = \alpha * s_u$$

α = adhesion factor =0.55 for undrained shear strength ≤ 190 kPa (4000 psf)
s_u = undrained shear strength of soil along the shaft
For higher values of s_u the value of α may be taken from the following figure obtained from test data of previous investigators.

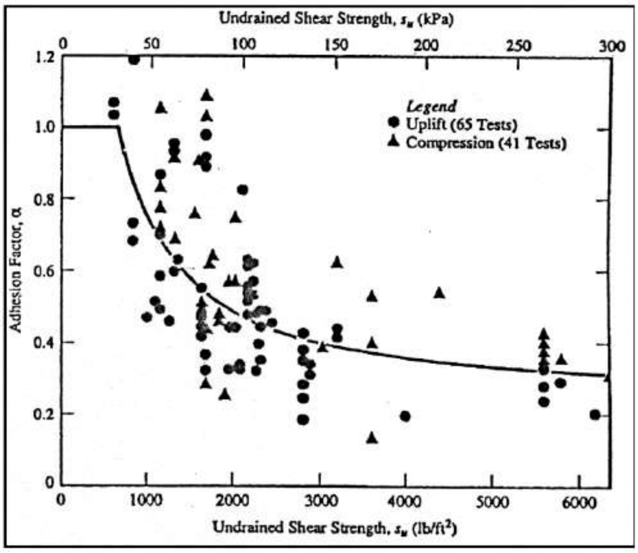


Fig 3.4: Adhesion factor α for drilled shaft (after Kulhawy and Jackson, 1989)

So, Skin Friction of Pile for Cohesive soil, $\mathrm{Q_s} = \mathrm{A_s} * \alpha * s_u$

The end bearing is obtained from following equation:

$$\mathrm{f_b} = \mathrm{N_c} * \mathrm{S_u} \leq 4000 \text{ kPa}$$

$$\mathrm{N_c} = 6\,(1 + 0.2 * \frac{L}{D}) \leq 9$$

Here,
f_b = End bearing stress
S_u = undrained shear strength of soil along the shaft
N_c = Bearing capacity factor
L = Length of the pile (Depth to the bottom of the shaft)
D_b = Diameter of the shaft base

If the base diameter is more than 1900 mm, the value of f_b could produce settlements greater than 25 mm, which would be unacceptable for most buildings. To keep settlement within tolerable limits, the value of f_b should be reduced to f_b' by multiplying a factor F_r such that:

$$\mathrm{f_b}' = \mathrm{F_r} * \mathrm{f_b}$$

$$\mathrm{F_r} = \frac{2.5}{120 w_1 * \frac{D_b}{B_r} + w_2} \leq 1.0$$

$$\mathrm{w_1} = 0.0071 + 0.0021 * \frac{L}{D_b} \leq 0.0015$$

$$\mathrm{w_2} = 1.59 \sqrt{\frac{\mathrm{s_u}}{\sigma_\mathrm{r}}};\ 0.5 \leq \mathrm{w_2} \leq 1.5$$

B_r = Reference width=1 ft = 0.3 m = 12 inch = 300 mm
σ_r = Reference stress = 100 kPa = 2000 psf

For Cohesionless Soil

Skin friction resistance in cohesionless soil is usually determined using the β-method. The relevant equation is reproduced again:

$$f_s = \beta * \sigma'_z = K * \sigma'_z * \tan\phi_s$$

Here,
K =lateral earth pressure coefficient.
σ'_z = Effective vertical stress at mid-point of soil layer
ϕ_s = Soil shaft interface friction angle

The values of K and ϕ_s can be obtained from the following Table from the soil friction angle, φ and preconstruction coefficient of lateral earth pressure K_0. However, K_0 is very difficult to determine. An alternative is to compute β directly using the following empirical relation.

$$\beta = 1.5 - 0.135\sqrt{\frac{z}{B_r}}$$

Where, B_r = Reference width=1 ft = 0.3 m = 12 inch = 300 mm
z = Depth from the ground surface to the mid-point of the strata

TYPICAL ϕ_s/ϕ AND K/K_0 VALUES FOR THE DESIGN OF DRILLED SHAFT			
Construction Method	**φ_s/φ**	**Construction Method**	**K/K_0**
Open hole or temporary casing	1.0	Dry construction with minimal side wall disturbance and prompt concreting	1.0
Slurry method – minimal slurry cake	1.0	Slurry construction – good workmanship	1.0
Slurry method – heavy slurry cake	0.8	Slurry construction – poor workmanship	2/3
Permanent casing	0.7	Casing under water	5/6

The unit end bearing capacity for drilled shaft in cohesionless soils will be less than that for driven piles because of various reasons like soil disturbance during augering, temporary stress relief while the hole is open, larger diameter and depth of influence etc. The reasons are not well defined, as such the following empirical formula developed by Reese and O' Nell (1989) may be suggested to use to estimate end bearing stress.

$$f_b = 0.60\,\sigma_r\,N \leq 4500\ \text{kPa}$$

Here, f_b = Unit bearing resistance
$\sigma_{r'}$ = Reference stress = 100 kPa = 2000 psf

N = Mean SPT value for the soil between the base of the shaft and a depth equal to two times the base diameter below the base. No overburden correction is required (N= N_{60})

If the base diameter is more than 1200 mm, the value of f_b could produce settlements greater than 25 mm, which would be unacceptable for most buildings. To keep settlement within tolerable limits, the value of f_b should be reduced to f_b' by multiplying a factor F_r such that:
So, The Ultimate Geotechnical Capacity of Pile for Cohesion less soil is

$$\mathrm{f_b}' = F_r * \mathrm{f_b}$$

$$\mathrm{F_r} = 4.17 * \frac{B_r}{D_b} \leq 1.0$$

Where, B_r = Reference width=1 ft = 0.3 m = 12 inch = 300 mm
D_b = Base diameter of drilled shaft

3.3 LIQUEFACTION POTENTIAL ANALYSIS

Liquefaction is a phenomenon during which soil (mainly fine sand and silty sand) losses its shear strength significantly and behaves as a fluid. During earthquakes, due to generation of excess pore pressure, effective stress will be reduced, and the soil may undergo complete liquefaction or decrease in shear strength causing settlement and lateral spreading of soil mass.

This phenomenon continues until the excess pore water pressure dissipates. Factors influencing are – soil type, relative density, confining pressure, stress due to earthquake, duration of earthquake and drainage condition etc.

Depending upon the location and extension of liquefiable soil layer below foundation, the foundation of any structure is susceptible to damage due to liquefaction. Hence, predicting soil liquefaction resistance is an important step in the engineering design of any engineering structure in seismic prone region.

Liquefaction potential is estimated based on cyclic stress ratio and cyclic resistance ratio. In this subsoil investigation report, liquefaction potential is analyzed based on Idriss and Boulanger 2014 Method. According to this method,

Cyclic stress ratio, $CSR_{Mw,\sigma'v} = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 * \frac{a_{max}}{g} * \frac{\sigma_{vo}}{\sigma'_{vo}} * r_d$

Where, r_d = Stress reduction coefficient = exp[α(z) + β(z)*M_w]

α = - 1.012 - 1.126 * sin{(z/11.73) + 5.133}

β = 0.106 + 0.118 * sin{(z/11.28) + 5.142}

Cyclic Resistance Ratio,

$CRR_{Mw=7.5,\sigma'v=1atm}$ = exp $\left[\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right]$

$(N_1)_{60cs}$ = Equivalent clean sand SPT-value = $(N_1)_{60} + \Delta(N_1)_{60}$

$\Delta(N_1)_{60}$ = exp $\left[1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right]$

Magnitude scaling factor, MSF = $1 + (MSF_{max} - 1)\{8.64\exp\left(\frac{-M}{4}\right) - 1.325\}$

$$MSF_{max} = 1.09 + \left(\frac{(N_1)_{60cs}}{31.5}\right)^2 \leq 2.2$$

Overburden correction factor, $K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_v}{P_a}\right) \leq 1.1$; where, $C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60cs}}} \leq 0.3$

Factor of Safety Against Liquefaction, $F_L = \frac{CRR_{M_W=7.5,\sigma'_v=1atm} * MSF * K_\sigma}{CSR_{M_W,\sigma'_v}}$

When the factor of safety is $F_L < 1$, liquefaction is said to take place.

LIQUEFACTION REVIEW AND EVALUATION CRITERIA:

Criteria for Liquefaction review:
Evaluation of Liquefaction Resistance of Soils: Idriss and Boulanger 2014 Method.
Peak ground acceleration, (PGA): As per site condition during investigation.
Moment Magnitude, M_w: 7.5
Water Table: As per site condition during investigation.

Criteria for evaluating Liquefaction possibility:

a) **Soil Type:**

In terms of the Soil Types most susceptible to liquefaction, Ishihara (1985) states: The hazard associated with soil liquefaction during earthquakes has been known to be encountered in deposits consisting of fine to medium sand and sands containing low plasticity fines. The soil types susceptible to liquefaction are non-plastic (cohesionless) soils. An approximate listing of cohesionless soils from least to must resistant to liquefaction is clean sands, non-plastic silty sands, non-plastic silt and gravels.

There could be numerous exceptions to this sequence. For sample Ishihara (1985 and 1993) describes the case of tailings derived from the industry that were essentially composed of ground-up rocks and were classified as rock flour. Ishihara (1985 and 1993) states that the rock flour in a water saturated slate did not possess significant cohesion and behaved as if it were clean sand. These tailings were shown to exhibit as low a resistance to liquefaction as clean sand. Seed et al. (1983) stated that based on both laboratories testing and field performance, the great majority of cohesive soils will not liquefy during earthquakes.

Using criteria originally staled by Seed and Idriss (1982) and subsequently confirmed by Youd and Oilstrap (1999) in order for a cohesive soil to liquefy, it must meet all the following three criteria:

1. The soil must have less than 15 percent of the particles, based on dry weight, that are finer than 0.005 mm (i.e., percent finer at 0.005 mm < 15 percent).
2. The soil must have a liquid limit (LL) that is less than 35.
3. The water content, w of the soil must be greater than 0.9 of the liquid limits.

However, according to Finn et. Al. 1994 and BNBC 2020, fine grained soils (silty clays/ clayey silt) are susceptible to liquefaction if:

1. Fraction finer than 0.005 mm ≤ 10%
2. Liquid limit (LL) ≤ 36%
3. Natural water content ≥ 0.9 × LL
4. Liquidity index ≥ 0.75

b) **Soil Depth:** Soil depth less than 22.5 m.

3.4 CORRECTION & STANDARDIZATION OF SPT-N

According to BNBC 2020 Appendix D and on the basis of field observations, it appears reasonable to standardize the field SPT number as a function of the input driving energy and its dissipation around the sampler around the surrounding soil, the variations in testing procedures may be at least partially compensated by converting the measured B to N_{60} as follows.

$$\mathrm{N}_{60} = \frac{E_H C_B C_R C_S N}{0.6}$$

Here,
N_{60} = Corrected SPT N-value for field procedures
E_H = Hammer efficiency
C_B = Borehole diameter correction
C_S = Sampler correction
C_R = Rod length correction
N = Measured SPT N-value field

This correction is to be done irrespective of the type of soil encountered.

Corrections of SPT Value for Overburden Pressure for All Types of Cohesionless Soils:

In cohesionless soils, the overburden pressure affects the penetration resistance. For SPT made at shallow levels, the values are usually too low. At a greater depth, the same soil at the same density index would give higher penetration resistance.

As the correction factor came to be considered only after 1957, all empirical data published before 1957 like those by Terzaghi is for uncorrected values of SPT. Since then, a number of investigators have suggested overburden correction. As such, all field SPT values are to be corrected by the correction factor given by them as:

$$\mathrm{C_N} = 0.77 \log\left(\frac{2000}{\sigma_0}\right)$$

Here,
σ_0 = is the effective overburden pressure. Thus, $(N_1)_{60} = C_N \times N_{60}$
The maximum value of correction factor C_N is 2.

Corrections of SPT Value for Water Table (Dilatancy) in case of Fine Sand and Silty Sand:

In addition to corrections of overburden, investigators suggested corrections of SPT-values for water table in the case of fine sand or silt below water table. Apparently, high N-values may be observed especially when observed values is higher than 15 due to dilatancy effect. In such cases, following correction is recommended (Terzaghi and Peck, 1948).

$$(N_1)_{60(\text{corrected})} = 15 + 0.5\,[(N_1)_{60} - 15]$$

Where, $(N_1)_{60(\text{corrected})}$ is the corrected $(N_1)_{60}$ for water table. For coarse sand this correction is not required. In applying this correction, overburden correction is applied first and then this dilatancy correction is used.

Chapter 04
Recommendation

4.1 BASIC REQUIREMENTS FOR FOUNDATION DESIGN

Design of a foundation for building structure requires following three geotechnical parameters:

1. Bearing capacity
2. Settlement
3. Seismic information of the site

According to section 3.4, BNBC 2020 and based on the results of our field investigation, laboratory testing and engineering judgement, our recommendations regarding above mentioned geotechnical parameters are as follows:

1. Bearing Capacity:

The prime objective of this investigation is to study the sub-soil characteristics at this site and to assess the Ultimate bearing capacity & Settlement of the sub-soil.

A. Shallow Foundation Capacity:

All the soil parameters required for calculating the soil bearing capacity are given in this report. Consultant may choose the convenient method to calculate the bearing capacity. A minimum factor of safety 2 to 3 Should be adopted to obtain safe bearing capacity while considering general shear failure and when dead load and normal live load is used. Thirty three percent (33%) overstressing above allowable pressure shall be allowed in case of design considering wind or seismic loading.

B. Pile Capacity

i. **Cast-In-Situ (Bored) Pile Capacity**

Two different theories were used to calculate the ultimate geotechnical capacity of cast in situ pile. The carrying capacities of bored RC piles for 24" dimension of various embedment length are given in Attachment-3. This report contains all the essential soil parameters needed to calculate the soil bearing capacity. The consultant may select both the convenient method and dimensions for assessing the carrying capacity of the bored pile.

ii. **Precast (Driven) Pile Capacity**

The carrying capacities of precast piles are given in Attachment-3 for 16" dimension.This report contains all the essential soil parameters needed to calculate the soil bearing capacity. The consultant may select both the convenient method and dimensions for assessing the carrying capacity of Driven pile. Maximum L/B (Length/Width) ratio should be less than 50 for driven precast pile.

N.B: The actual pile capacity should be determined by carrying out load tests at site as per BNBC 2020.

2. Settlement:

After finalizing the required geometry of the foundations, it is expected that for the above-mentioned allowable bearing capacities, the settlement of this structure would remain within tolerable margin. Total settlement should be evaluated as per BNBC 2020.

3. Seismic Information of The Site:

Seismic information of the site according to BNBC-2020 are given here:

Borehole No.	[As per Table 6.2.13, BNBC 2020]		[As per Figure 6.2.24, BNBC 2020]	
	Soil Classification	**Seismic Intensity**	**Seismic Zone**	**Seismic Zone Coefficient**
01	SD	SEVERE	ZONE - III	0.28
02	N/A	SEVERE	ZONE - III	0.28
03	N/A	SEVERE	ZONE - III	0.28
04	N/A	SEVERE	ZONE - III	0.28

N/A=Not applicable as the boring depth Is less than 30m.

Note: To determine the site class, soil data for a depth of 30 meters is required. If we extend the layers below to reach this depth, we can suggest a site class. For borehole 1 based on soil data from 1.5m to 30m, the site class can be suggested as SD.

4.2 RECOMMENDATION

It is relevant to mention here that this report should be comprehensively examined and not fragmented or selectively referenced. Such an approach is vital in order to prevent any potential misinterpretation of the information presented. The report encompasses the findings related to the sub-soil condition, which have been derived from the borehole locations and subsequent analysis of representative soil samples obtained from those specific locations.

It is acknowledged that there may exist local variations in the sub-soil conditions that have not been explicitly addressed through the existing bore hole locations. However, this sub-soil investigation report offers recommendations and findings that serve as a basis for addressing and mitigating any such local sub-soil variations.

By thoroughly considering the provided findings, one can effectively overcome potential challenges posed by these unexplored variations. This approach ensures informed decision-making and accurate conclusions pertaining to the sub-soil conditions. It is imperative to bear in mind that a comprehensive understanding of the report is crucial to avoid any misinterpretation and to facilitate appropriate actions and measures based on the entirety of the sub-soil investigation report.

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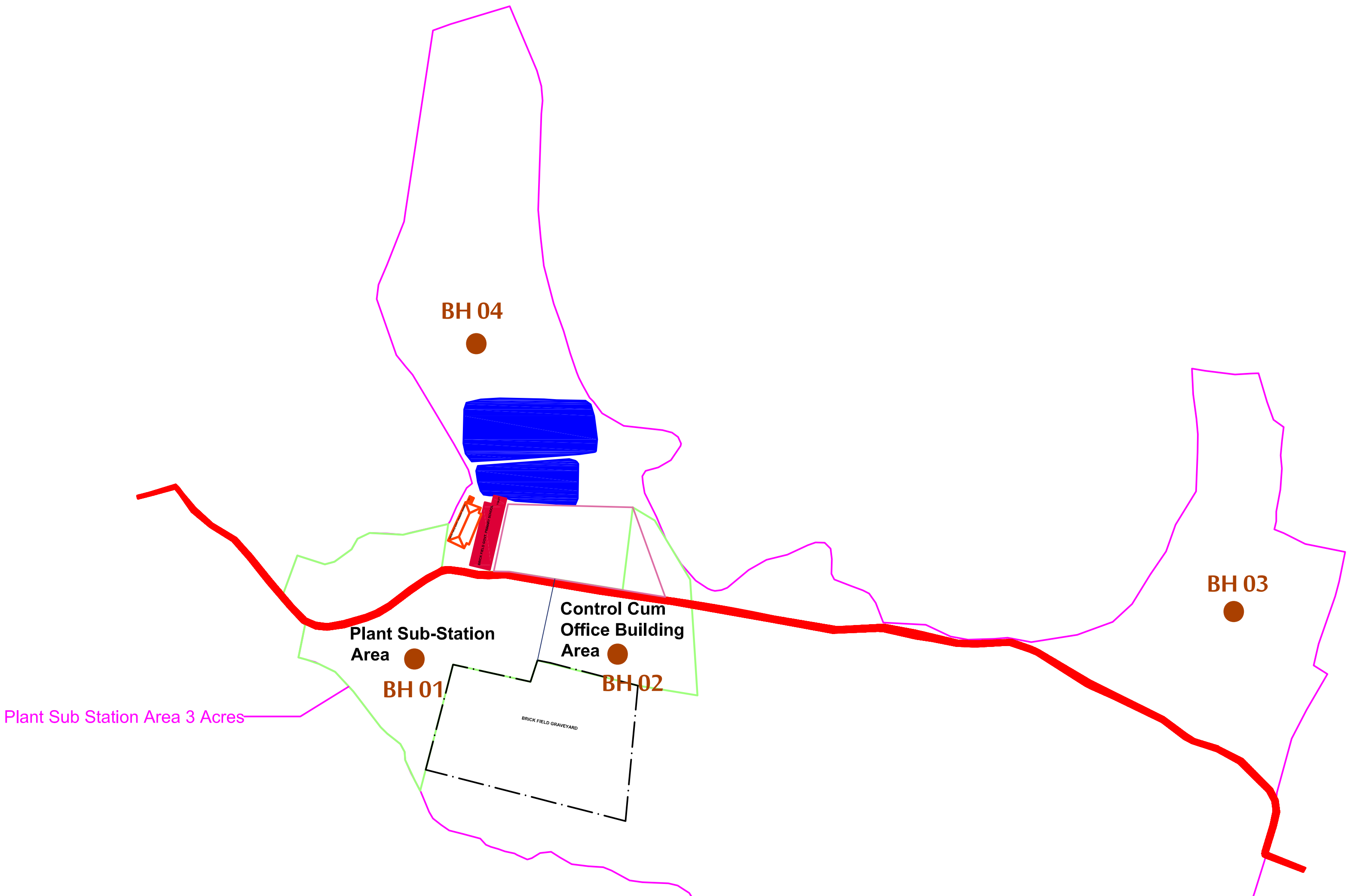
APPENDICES

APPENDIX-01

BOREHOLE LOCATION PLAN

Kaptai 7.6 MW Solar PV Project Area

GEOSCAPE Consultants Ltd.



Note:

1. **Project Area = 100,402 m2 or 24.80 Acres (Including all)**
2. **Area for Plant Substation and Control Cum Office Building = 12,145 m2 or 3 Acres)**

BORE HOLE LOCATION PLAN

APPENDIX-02

LABORATORY TEST RESULTS

APPENDIX-2A

BORE LOG

Geoscape Consultants Ltd.

House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

BORE LOG

BH NO : 01

CLIENT : BANGLADESH POWER DEVELOPMENT BOARD(BPDB)

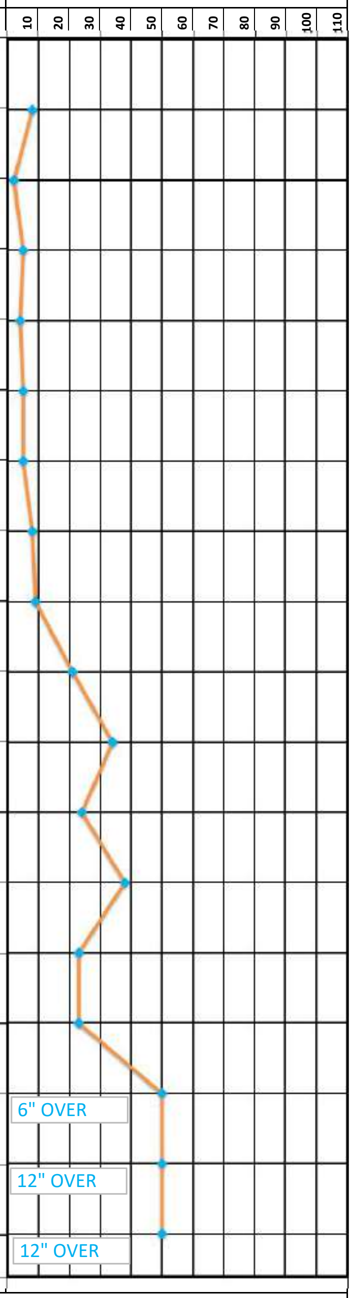
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH

LOCATION : KAPTAI, RANGAMATI, BANGLADESH

LAT (deg): N 22.483800
LONG (deg): E 92.238058
DATE OF BORING: 03-11-2023
T.B.M. = 0, CONSIDERED AT ROAD LEVEL
REDUCED LEVEL, R.L = (-) 0.60 m
GROUND WATER LEVEL = (-) 2.15 m

DEPTH (m)	SAMPLE NO.	SAMPLE INDEX	LAYER THICKNESS	STRATA COMPOSITION	USCS	PLASTICITY	CONSISTENCY OR COMPACTNESS	LITHOLOGICAL LOG	S.P.T VALUE
0-1.5			1.5 m	Silty Clay					
1.5	D-01, U-01		1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Medium Stiff/ Stiff		8
3	D-02		1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Very Soft/ Soft		2
4.5	D-03		1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Medium Stiff		5
6	D-04		1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Soft/ Medium Stiff		4
7.5	D-05		3 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Medium Stiff		5
9	D-06								5
10.5	D-07		1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Medium Stiff/ Stiff		8
12	D-08		1.5 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Loose		9
13.5	D-09		1.5 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Medium Dense		21
15	D-10		1.5 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Dense		34
16.5	D-11		1.5 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Medium Dense		24
18	D-12		1.5 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Dense		38
19.5	D-13		3 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Medium Dense		23
21	D-14								23
22.5	D-15		3.5 m	Mostly **Silt** with Little Clay	**CL-ML,** Silty Clay	Plastic	Hard		50
24	D-16								50
25.5	D-17								50

S.P.T NUMBER
(Number of Blows per 0.3 m)



SOIL INDEX		LEGENDS			
DISTURBED SAMPLE		CLAY		ORGANIC SOIL	
UNDISTURBED SAMPLE		SAND		FILL/RUBBISH	
		SILT		MICA	

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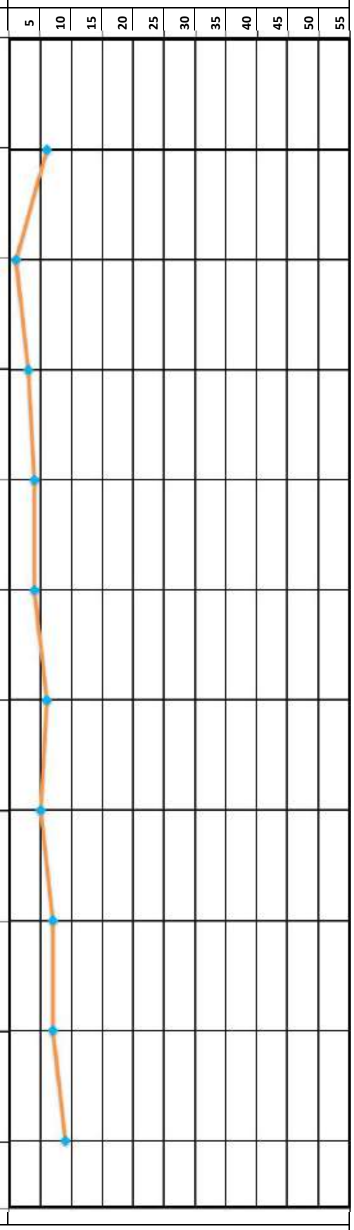
BORE LOG

BH NO : 02

CLIENT : BANGLADESH POWER DEVELOPMENT BOARD(BPDB)	LAT (deg): N 22.483770
	LONG (deg): E 92.239518
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	DATE OF BORING: 05-11-2023
	T.B.M. = 0, CONSIDERED AT ROAD LEVEL
LOCATION : KAPTAI, RANGAMATI, BANGLADESH	REDUCED LEVEL, R.L = (-) 0.65 m
	GROUND WATER LEVEL = (-) 4.25 m

DEPTH (m)	SAMPLE NO.	SAMPLE INDEX	LAYER THICKNESS	STRATA COMPOSITION	USCS	PLASTICITY	CONSISTENCY OR COMPACTNESS	LITHOLOGICAL LOG	S.P.T VALUE
0-1.5			1.5 m	Silty Clay				Clay	
1.5	D-01	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Medium Stiff	Clay	6
3	D-02	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Very Soft	Clay	1
	U-01	Undisturbed							
4.5	D-03	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Soft	Clay	3
6	D-04	Disturbed	3 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Soft/ Medium Stiff	Clay	4
7.5	D-05	Disturbed							4
9	D-06	Disturbed	3 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Medium Stiff	Clay	6
10.5	D-07	Disturbed							5
12	D-08	Disturbed	3 m	Mostly **Silt** with Little Clay	**ML,** Silt	Plastic	Medium Stiff	Silt	7
13.5	D-09	Disturbed							7
15	D-10	Disturbed	0.5 m	Mostly **Silt** with Little Clay	**ML,** Silt	Plastic	Stiff	Silt	9

S.P.T NUMBER (Number of Blows per 0.3 m)



SOIL INDEX		LEGENDS	
DISTURBED SAMPLE		CLAY	ORGANIC SOIL
		SAND	FILL/RUBBISH
UNDISTURBED SAMPLE		SILT	MICA

Mohammed

FIELD SUPERVISER

Iftekhar Ahmed

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House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

BORE LOG

BH NO : 03

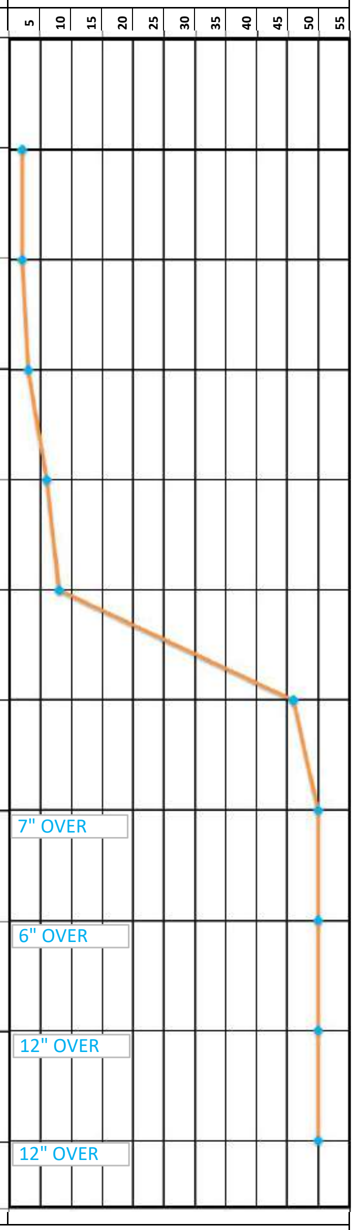
CLIENT : BANGLADESH POWER DEVELOPMENT BOARD(BPDB)

PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH

LOCATION : KAPTAI, RANGAMATI, BANGLADESH

LAT (deg): N 22.483592
LONG (deg): E 92.241908
DATE OF BORING: 05-11-2023
T.B.M. = 0, CONSIDERED AT ROAD LEVEL
REDUCED LEVEL, R.L = (+) 0.30 m
GROUND WATER LEVEL = (-) 2.90 m

DEPTH (m)	SAMPLE NO.	SAMPLE INDEX	LAYER THICKNESS	STRATA COMPOSITION	USCS	PLASTICITY	CONSISTENCY OR COMPACTNESS	LITHOLOGICAL LOG	S.P.T VALUE	S.P.T NUMBER (Number of Blows per 0.3 m)
0-1.5			1.5 m	Silty Sand				Sand		
1.5	D-01	Disturbed	3 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Very Loose	Sand	2	
3	D-02	Disturbed							2	
4.5	D-03	Disturbed	1.5 m	Mostly **Silt** with Little Clay	**CL-ML,** Silty Clay	Plastic	Soft	Silt	3	
6	D-04	Disturbed	1.5 m	Mostly **Silt** with Little Clay	**CL-ML**, Silty Clay	Plastic	Medium Stiff	Silt	6	
7.5	D-05, U-01	Disturbed, Undisturbed	1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Medium Stiff/ Stiff	Clay	8	
9	D-06	Disturbed	1.5 m	Mostly **Clay** with Some Silt	**CH,** Fat Clay	Plastic	Hard	Clay	46	
10.5	D-07	Disturbed	3 m	Mostly **Sand** with Little Silt and Few Clay	**SM,** Silty Sand	Non-Plastic	Very Dense	Sand	50	7" OVER
12	D-08	Disturbed							50	6" OVER
13.5	D-09	Disturbed	2 m	Mostly **Silt** with Little Clay	**ML,** Silt	Plastic	Hard	Silt	50	12" OVER
15	D-10	Disturbed							50	12" OVER



SOIL INDEX

DISTURBED SAMPLE

UNDISTURBED SAMPLE

LEGENDS

CLAY
SAND
SILT
ORGANIC SOIL
FILL/RUBBISH
MICA

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House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

BORE LOG

BH NO : 04

CLIENT : BANGLADESH POWER DEVELOPMENT BOARD(BPDB)

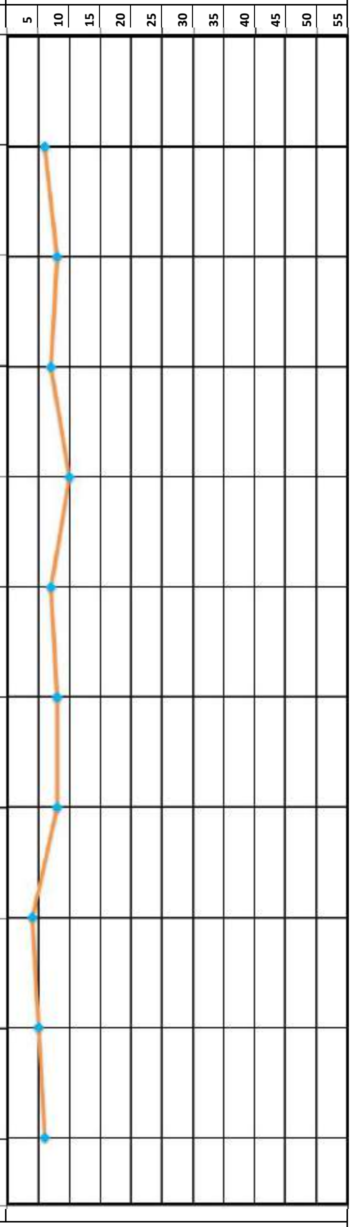
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH

LOCATION : KAPTAI, RANGAMATI, BANGLADESH

LAT (deg): N 22.483800
LONG (deg): E 92.238058
DATE OF BORING: 06-11-2023
T.B.M. = 0, CONSIDERED AT ROAD LEVEL
REDUCED LEVEL, R.L = (-) 0.60 m
GROUND WATER LEVEL = (-) 2.15 m

DEPTH (m)	SAMPLE NO.	SAMPLE INDEX	LAYER THICKNESS	STRATA COMPOSITION	USCS	PLASTICITY	CONSISTENCY OR COMPACTNESS	LITHOLOGICAL LOG	S.P.T VALUE
0-1.5			1.5 m	Silty Clay				Clay	
1.5	D-01, U-01	Disturbed, Undisturbed	1.5 m	Mostly **Silt** with Some Clay	**CH,** Fat Clay	Plastic	Medium Stiff	Clay	6
3	D-02	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CH,** Fat Clay	Plastic	Medium Stiff/ Stiff	Clay	8
4.5	D-03	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CH,** Fat Clay	Plastic	Medium Stiff	Clay	7
6	D-04	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CL,** Lean Clay	Plastic	Stiff	Clay	10
7.5	D-05	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CH,** Fat Clay	Plastic	Medium Stiff	Clay	7
9	D-06	Disturbed	1.5 m	Mostly **Silt** with Some Clay	**CH,** Fat Clay	Plastic	Medium Stiff/ Stiff	Clay	8
10.5	D-07	Disturbed	1.5 m	Mostly **Silt** with Little Clay	**ML,** Silt	Plastic	Medium Stiff/ Stiff	Silt	8
12	D-08	Disturbed	1.5 m	Mostly **Silt** with Little Clay	**ML,** Silt	Plastic	Soft/ Medium Stiff	Silt	4
13.5	D-09	Disturbed	2 m	Mostly **Silt** with Little Clay	**ML,** Silt	Plastic	Medium Stiff	Silt	5
15	D-10	Disturbed						Silt	6

S.P.T NUMBER
(Number of Blows per 0.3 m)



SOIL INDEX		LEGENDS			
DISTURBED SAMPLE		CLAY		ORGANIC SOIL	
		SAND		FILL/RUBBISH	
UNDISTURBED SAMPLE		SILT		MICA	

Mohammad
FIELD SUPERVISER

CHECKED BY

APPENDIX-2B

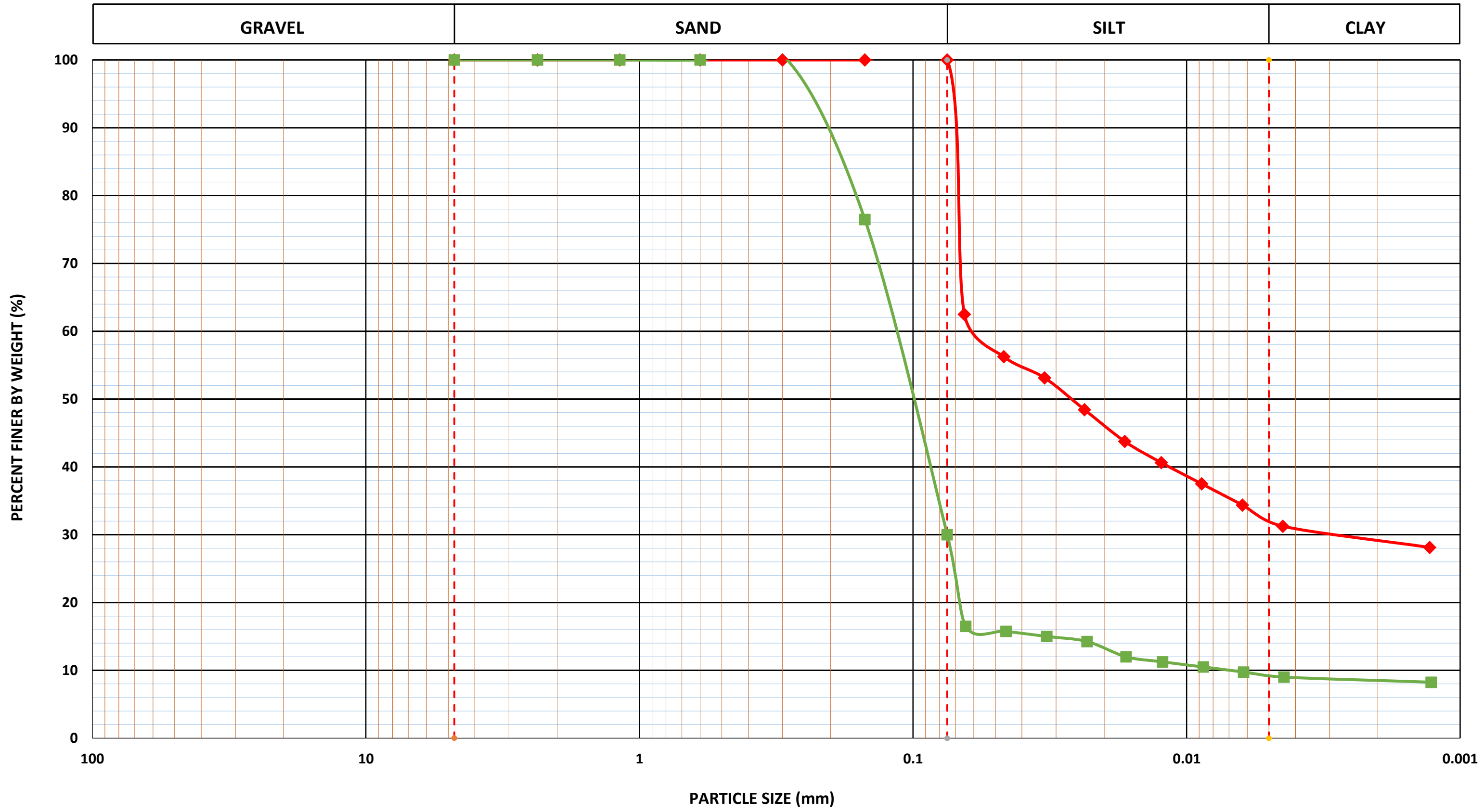
GRAIN SIZE DISTRIBUTION CURVES

Geoscape Consultants Ltd.

House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)	DATE OF BORING:	03/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	DATE OF TEST:	12/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD	ASTM D 6913 & ASTM D 7923

GRAIN SIZE DISTRIBUTION



BH NO.	SAMPLE NO.	DEPTH (m)	LEGEND	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	D_{60}	D_{30}	D_{10}	C_c	C_u	FG/CG	USCS
1	D-3	4.5		36	22.39	13.61	0	68	32						FG	CL
	D-8	12					70	21	9	0.123	0.075	0.007	6.5331	17.5714	CG	SM

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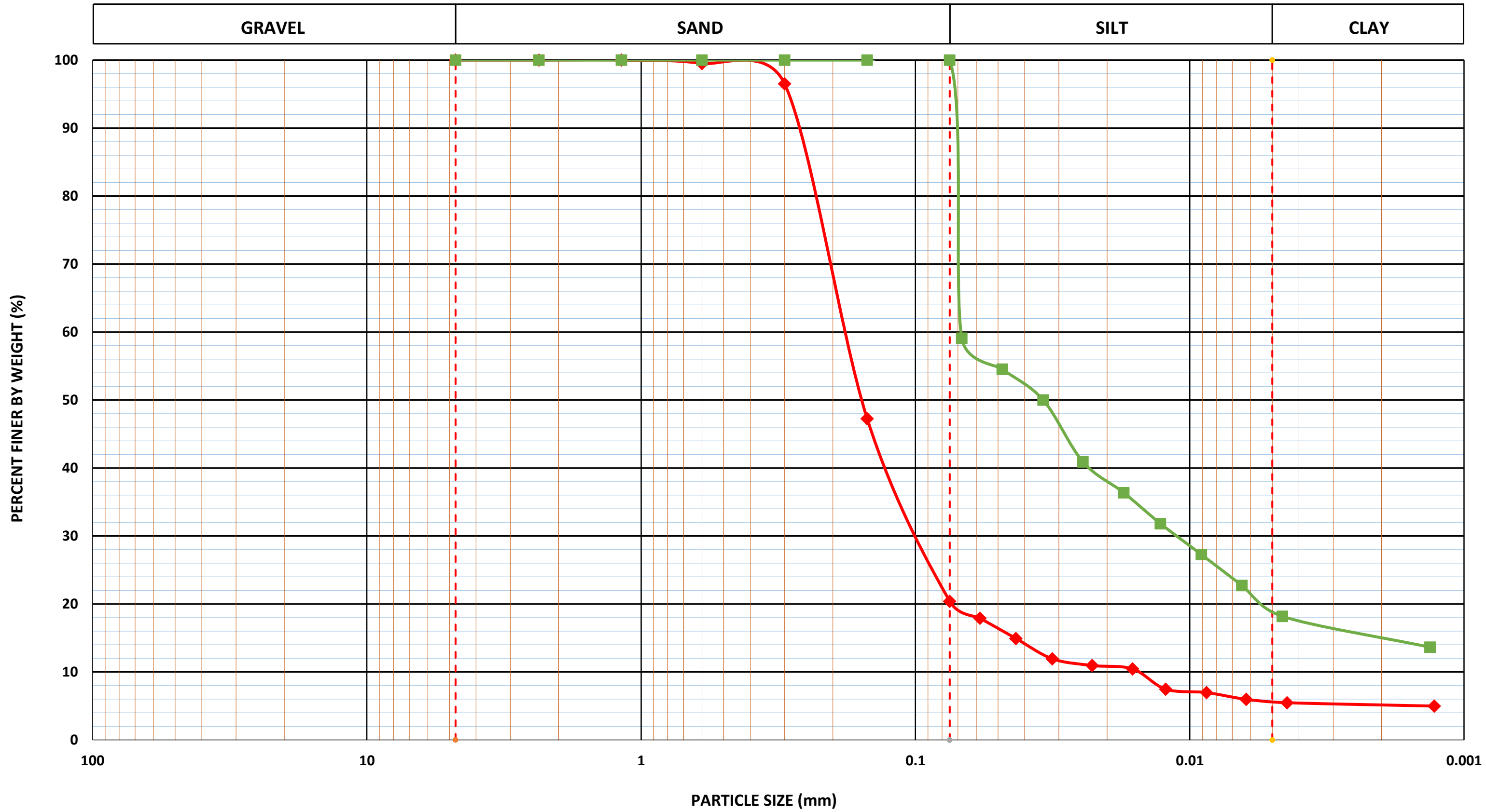
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)	DATE OF BORING:	03/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	DATE OF TEST:	12/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD	ASTM D 6913 & ASTM D 7923

GRAIN SIZE DISTRIBUTION



BH NO.	SAMPLE NO.	DEPTH (m)	LEGEND	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	D_{60}	D_{30}	D_{10}	C_C	C_U	FG/CG	USCS
1	D-11	16.5					80	15	5	0.189	0.102	0.016	3.4405	11.8125	CG	SM
	D-16	24		28	22.32	5.68	0	81	19						FG	CL-ML

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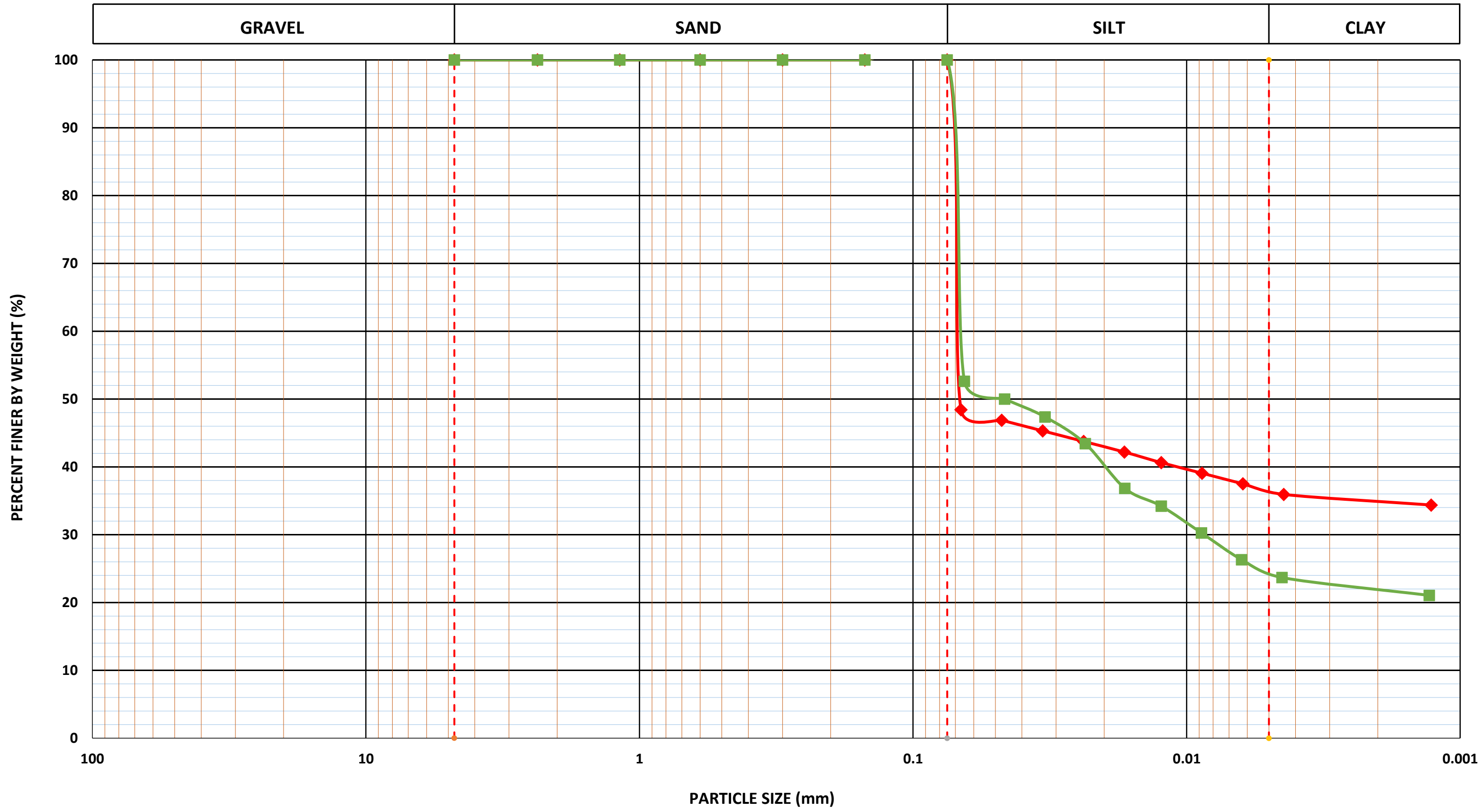
CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)	DATE OF BORING:	05/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	DATE OF TEST:	12/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD	ASTM D 6913 & ASTM D 7923

GRAIN SIZE DISTRIBUTION



BH NO.	SAMPLE NO.	DEPTH (m)	LEGEND	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	D_{60}	D_{30}	D_{10}	C_C	C_U	FG/CG	USCS
2	D-5	7.5		35	21.26	13.74	0	64	36						FG	CL
	D-10	15		39	26.96	12.04	0	76	24						FG	ML

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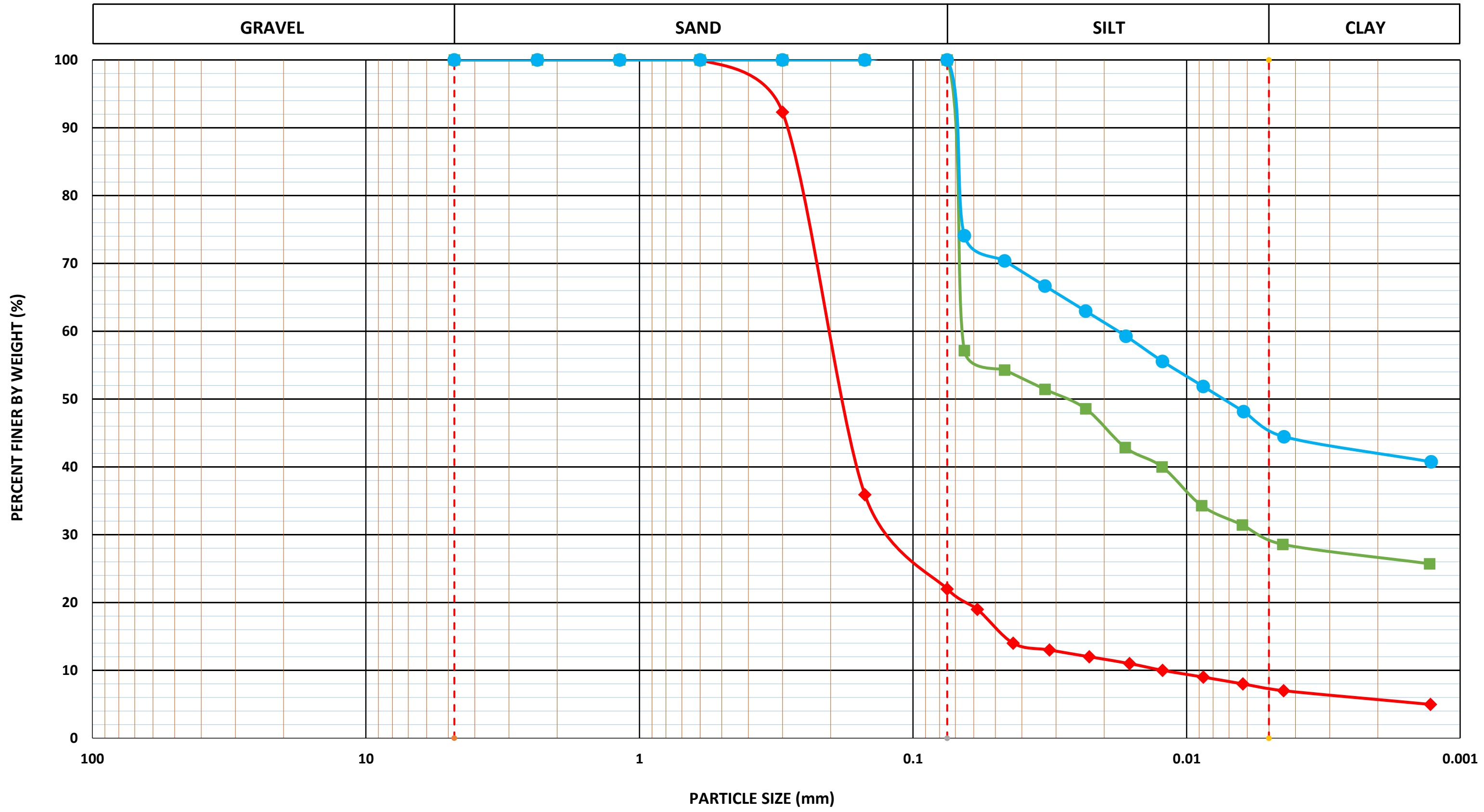
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)		DATE OF BORING:	05/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH		DATE OF TEST:	12/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD	ASTM D 6913 & ASTM D 7923

GRAIN SIZE DISTRIBUTION



BH NO.	SAMPLE NO.	DEPTH (m)	LEGEND	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	D_{60}	D_{30}	D_{10}	C_C	C_U	FG/CG	USCS
3	D-1	1.5					78	15	7	0.214	0.118	0.012	5.4221	17.8333	CG	SM
	D-4	6		21	14.62	6.38	0	71	29						FG	CL-ML
	D-5	7.5		45	25.69	19.31	0	55	45						FG	CL

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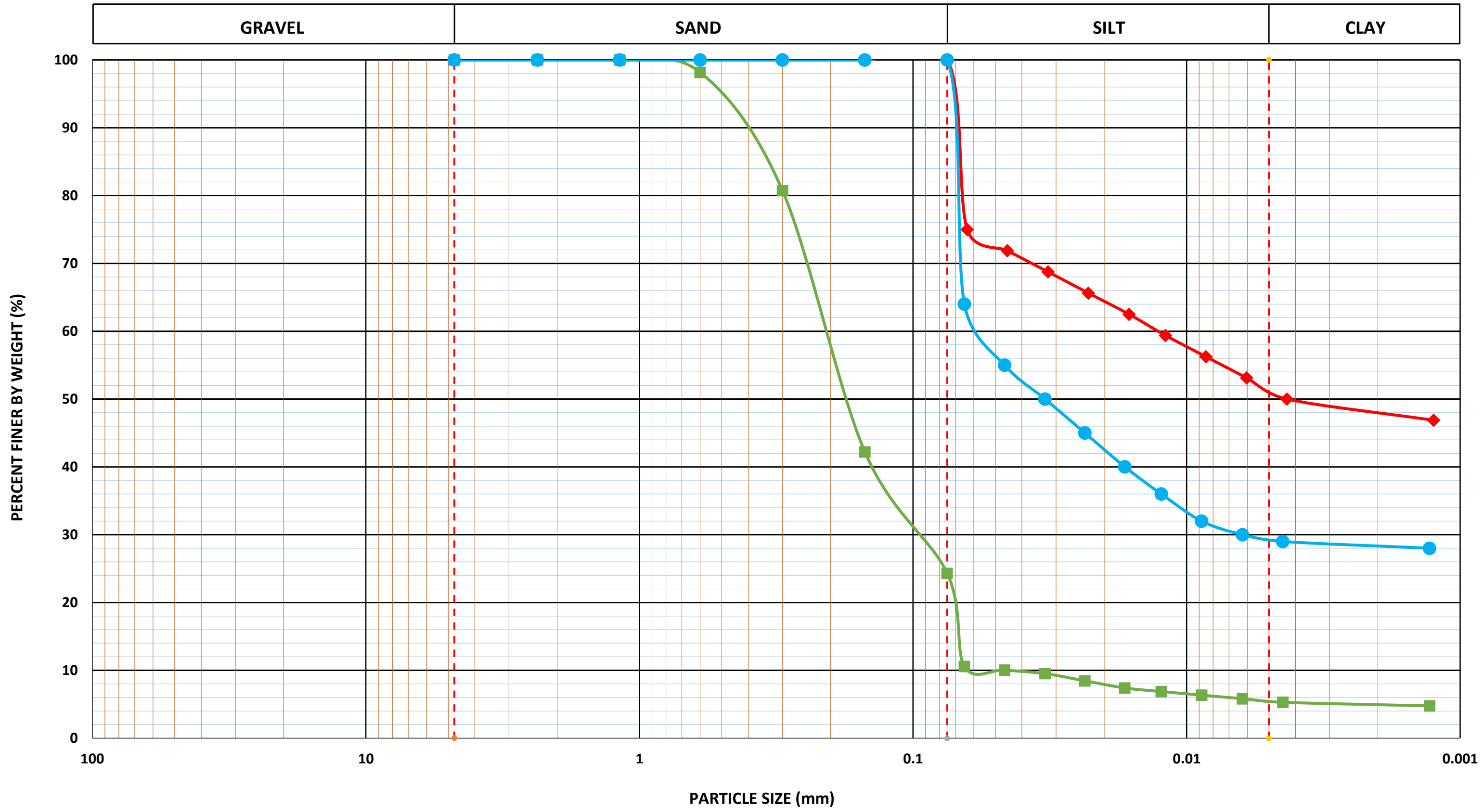
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)	DATE OF BORING:	05/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	DATE OF TEST:	12/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD	ASTM D 6913 & ASTM D 7923

GRAIN SIZE DISTRIBUTION



BH NO.	SAMPLE NO.	DEPTH (m)	LEGEND	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	D_{60}	D_{30}	D_{10}	C_c	C_u	FG/CG	USCS
3	D-6	9		55	31.58	23.42	0	49	51						FG	CH
	D-8	12					76	19	5	0.219	0.099	0.045	0.9945	4.8667	CG	SM
	D-10	15		33	25.17	7.83	0	71	29							ML

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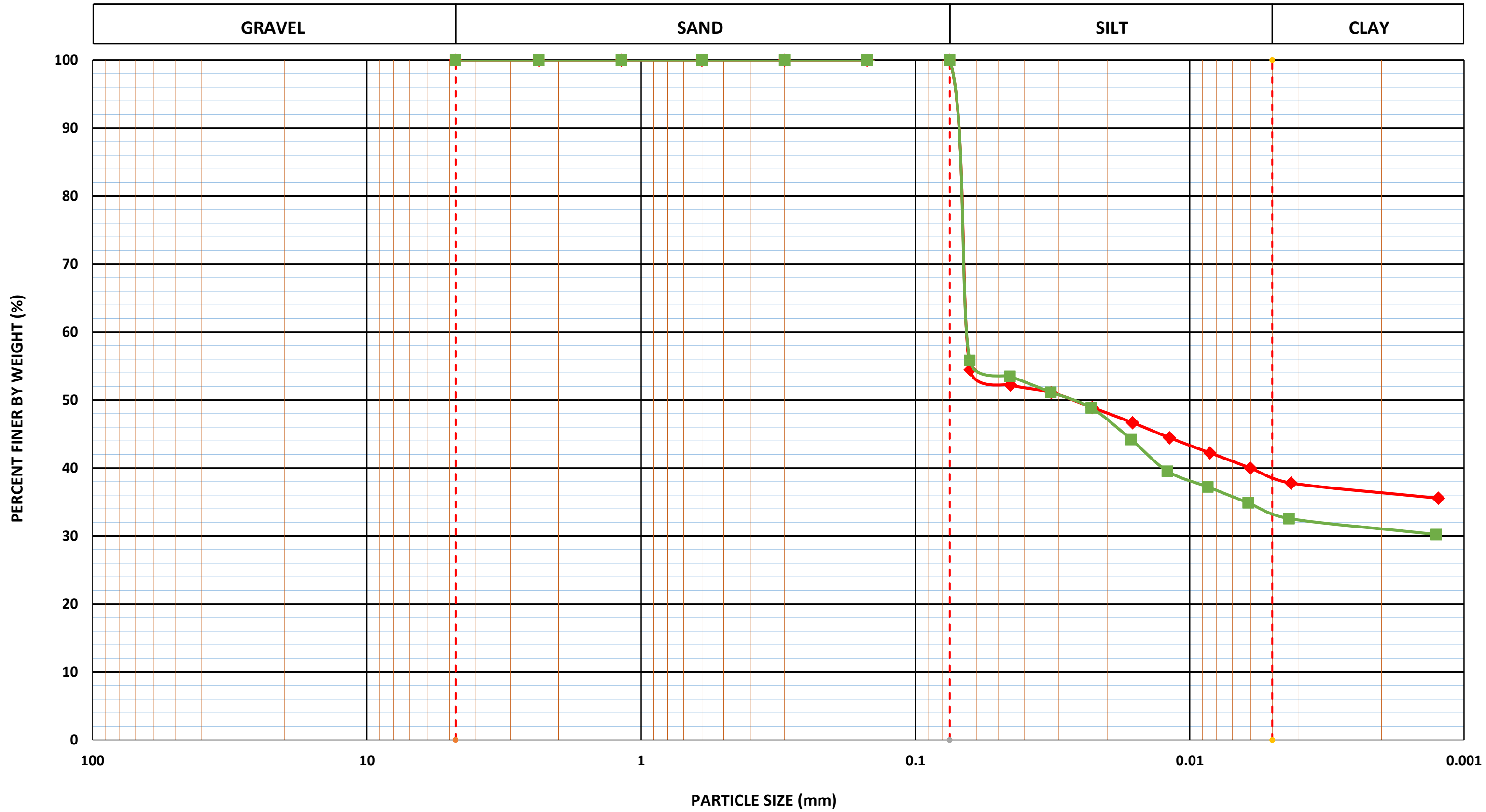
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)		DATE OF BORING:	06/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH		DATE OF TEST:	12/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD	ASTM D 6913 & ASTM D 7923

GRAIN SIZE DISTRIBUTION



BH NO.	SAMPLE NO.	DEPTH (m)	LEGEND	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	D_{60}	D_{30}	D_{10}	C_c	C_u	FG/CG	USCS
4	D-2	3		54	35.65	18.35	0	62	38						FG	CH
	D-4	6		48	23.48	24.52	0	67	33						FG	CL

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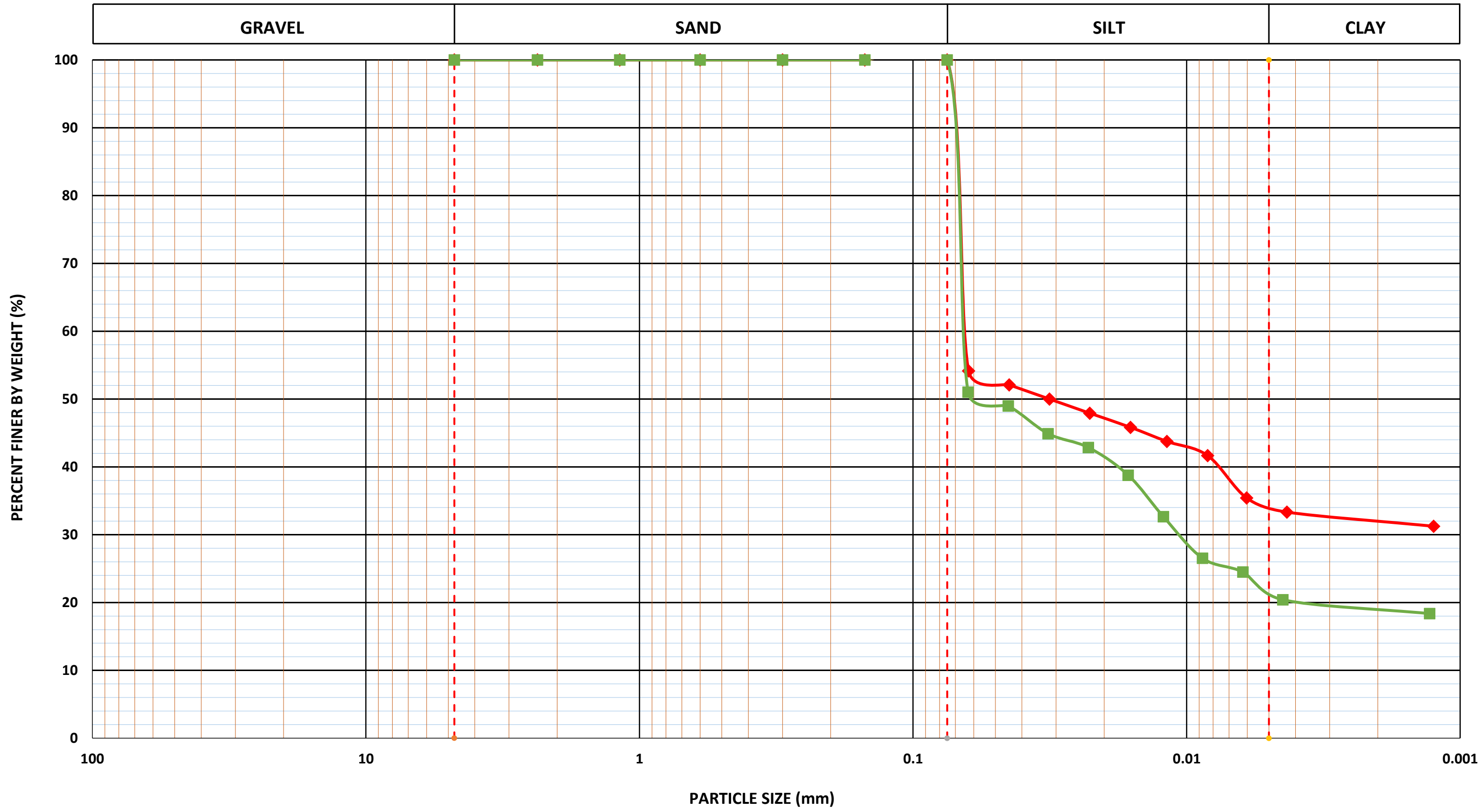
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)		DATE OF BORING:	06/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH		DATE OF TEST:	12/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD	ASTM D 6913 & ASTM D 7923

GRAIN SIZE DISTRIBUTION



BH NO.	SAMPLE NO.	DEPTH (m)	LEGEND	LL	PL	PI	SAND (%)	SILT (%)	CLAY (%)	D_{60}	D_{30}	D_{10}	C_c	C_u	FG/CG	USCS
4	D-5	7.5		54	23.42	30.58	0	66	34						FG	CH
	D-9	13.5		36	27.12	8.88	0	79	21						FG	ML

APPENDIX-2C

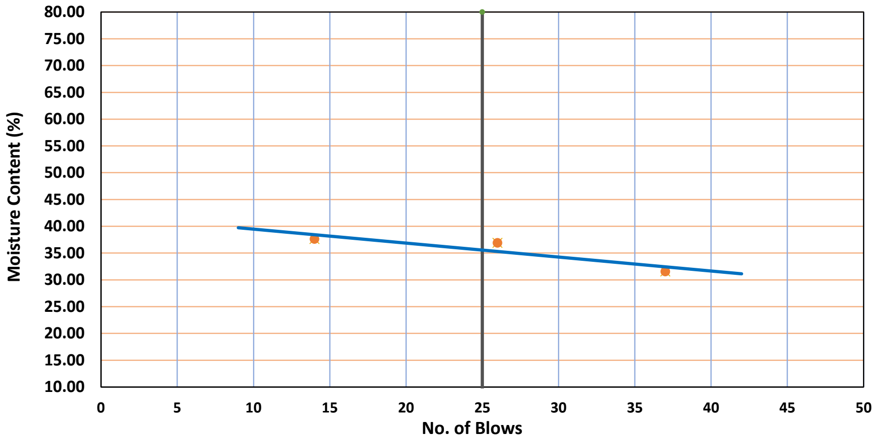
LIQUID LIMIT ANALYSIS CURVES

Geoscape Consultants Ltd.

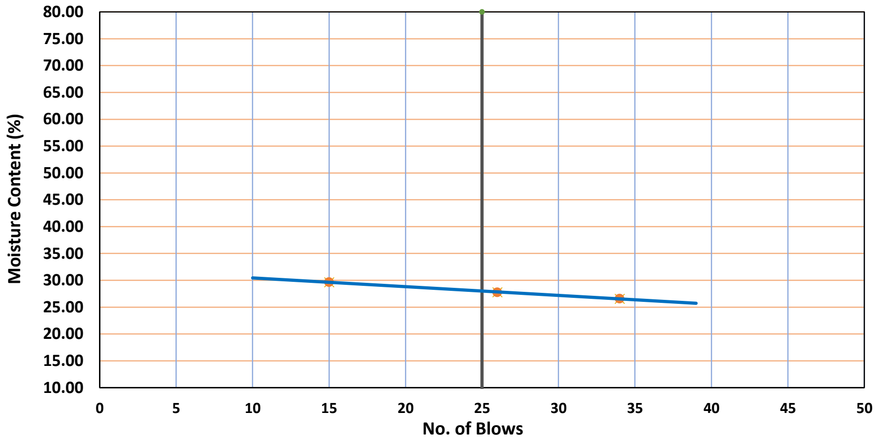
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)		DATE OF TEST:	12/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD:	ASTM D4318
LOCATION: KAPTAI, RANGAMATI, BANGLADESH			

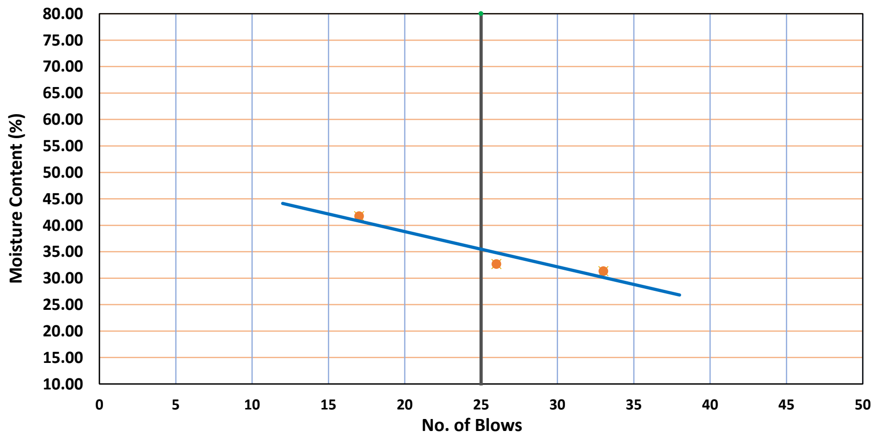
LIQUID LIMIT ANALYSIS CURVE



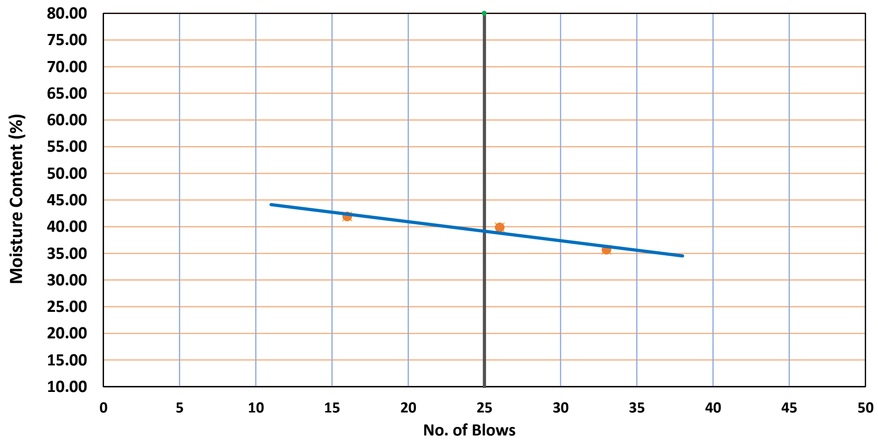
BH. NO:	1	DEPTH (m)	4.5	LIQUID LIMIT	36	PLASTIC LIMIT	22.39



BH. NO:	1	DEPTH (m)	24	LIQUID LIMIT	28	PLASTIC LIMIT	22.32



BH. NO:	2	DEPTH (m)	7.5	LIQUID LIMIT	35	PLASTIC LIMIT	21.26



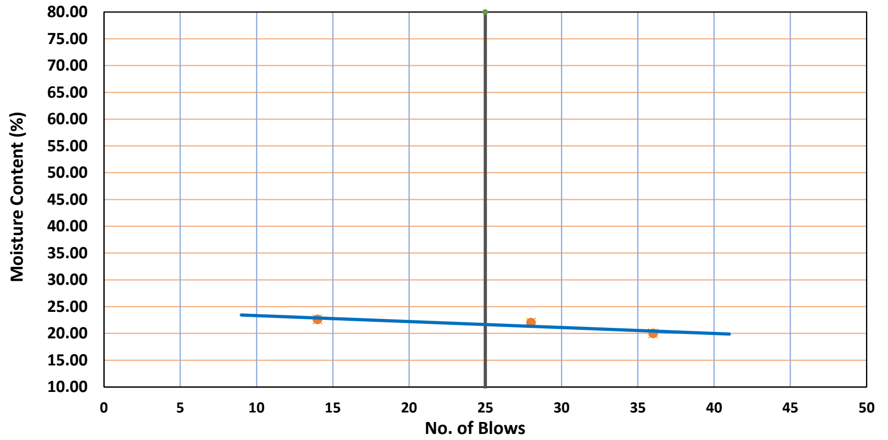
BH. NO:	2	DEPTH (m)	15	LIQUID LIMIT	39	PLASTIC LIMIT	26.96

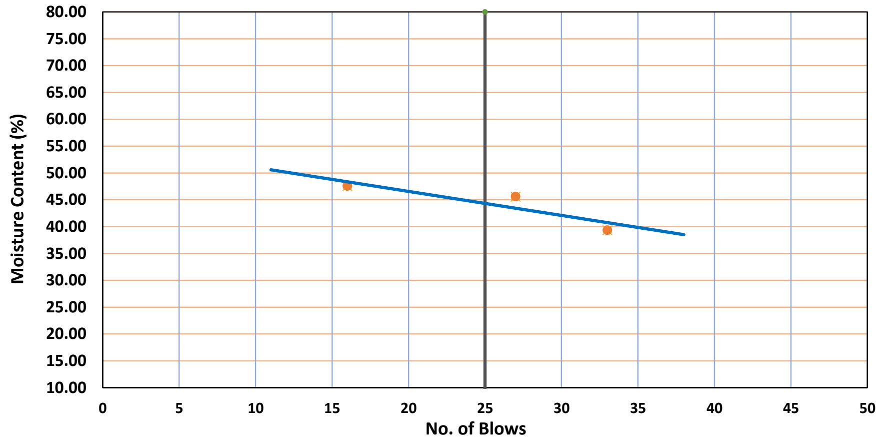
Geoscape Consultants Ltd.

House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

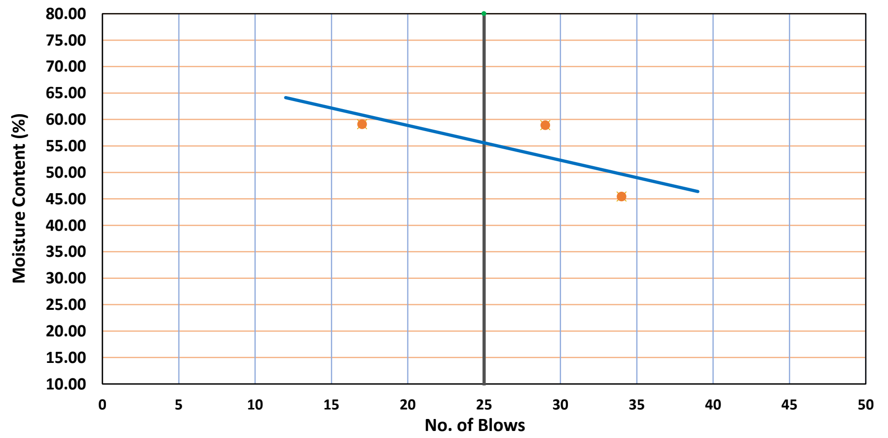
CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)	DATE OF TEST:	12/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD:	ASTM D4318
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		

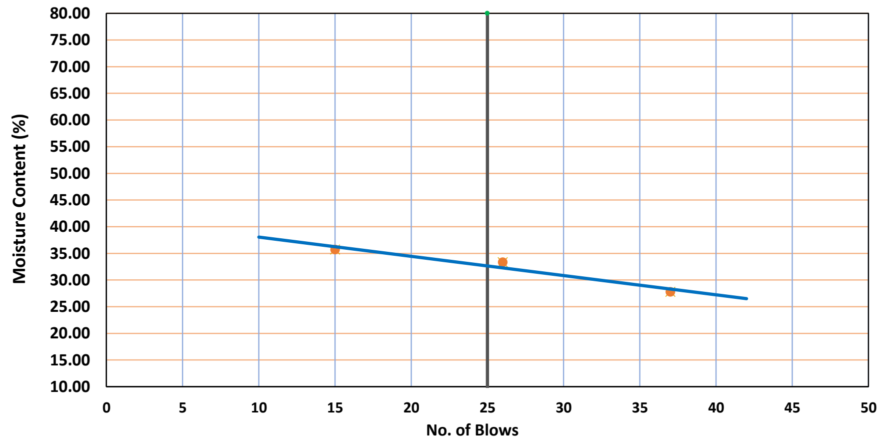
LIQUID LIMIT ANALYSIS CURVE





BH. NO:	3	DEPTH (m)	6	LIQUID LIMIT	22	PLASTIC LIMIT	14.62
BH. NO:	3	DEPTH (m)	7.5	LIQUID LIMIT	45	PLASTIC LIMIT	25.69





BH. NO:	3	DEPTH (m)	9	LIQUID LIMIT	55	PLASTIC LIMIT	31.58
BH. NO:	3	DEPTH (m)	15	LIQUID LIMIT	33	PLASTIC LIMIT	25.17

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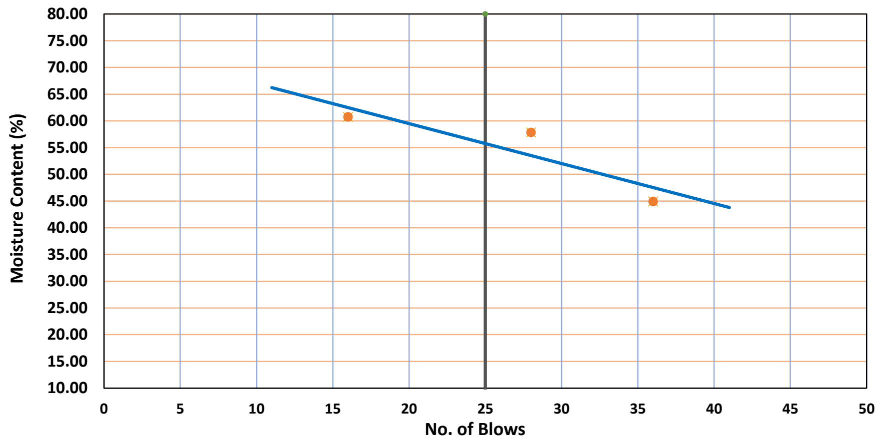
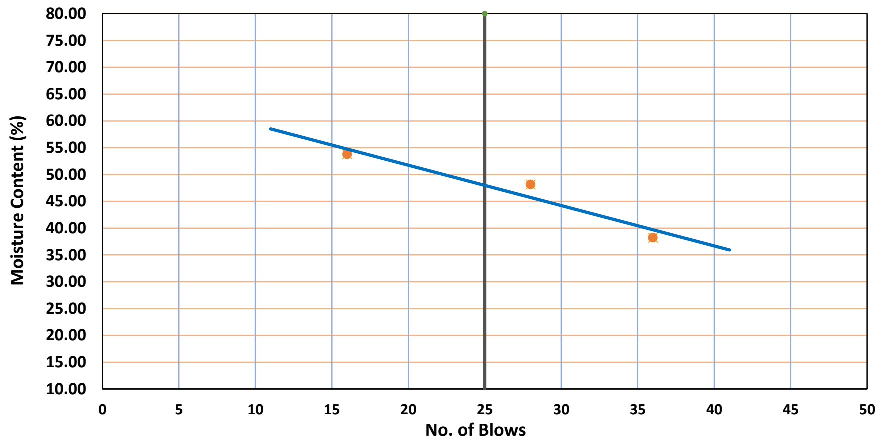
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

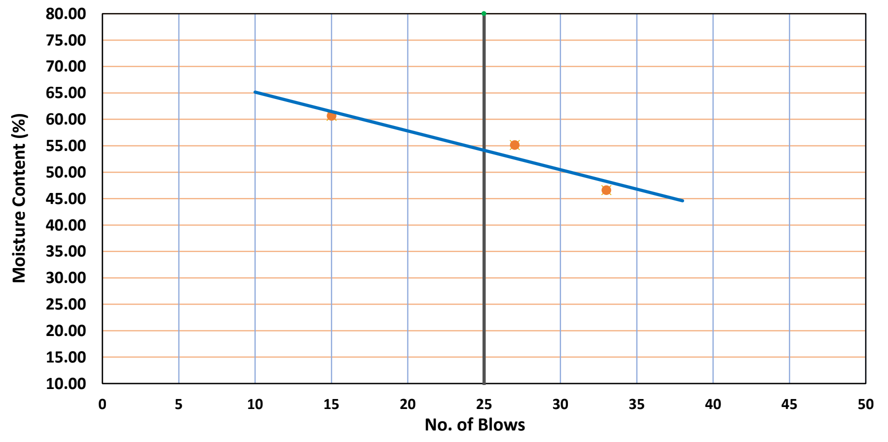
CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)	DATE OF TEST:	12/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD:	ASTM D4318
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		

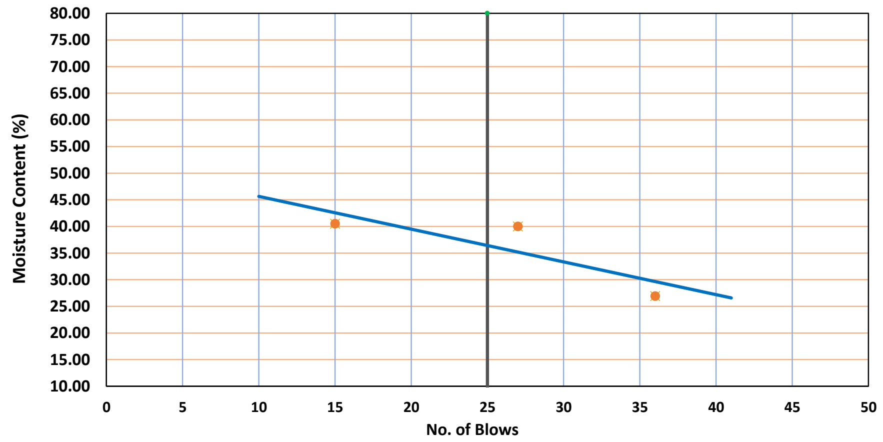
LIQUID LIMIT ANALYSIS CURVE





BH. NO:	4	DEPTH (m)	3	LIQUID LIMIT	54	PLASTIC LIMIT	35.65	BH. NO:	4	DEPTH (m)	6	LIQUID LIMIT	48	PLASTIC LIMIT	23.48





BH. NO:	4	DEPTH (m)	7.5	LIQUID LIMIT	54	PLASTIC LIMIT	23.42	BH. NO:	4	DEPTH (m)	13.5	LIQUID LIMIT	36	PLASTIC LIMIT	27.12

APPENDIX-2D

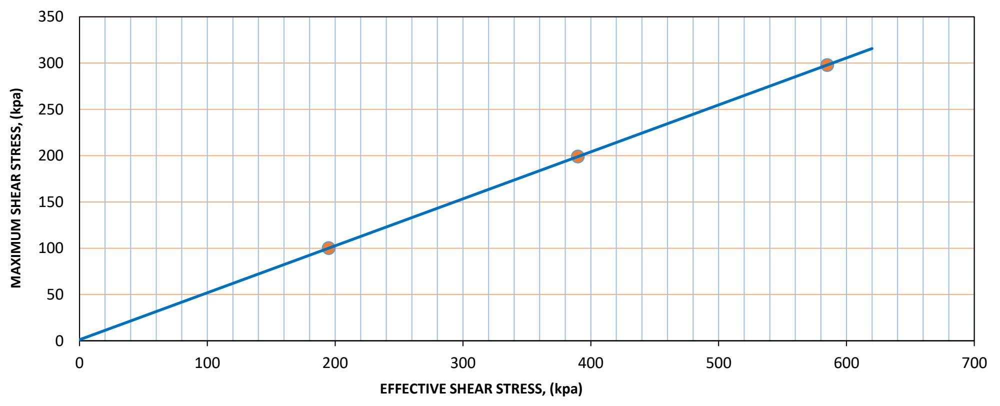
DIRECT SHEAR TEST CURVES

Geoscape Consultants Ltd.

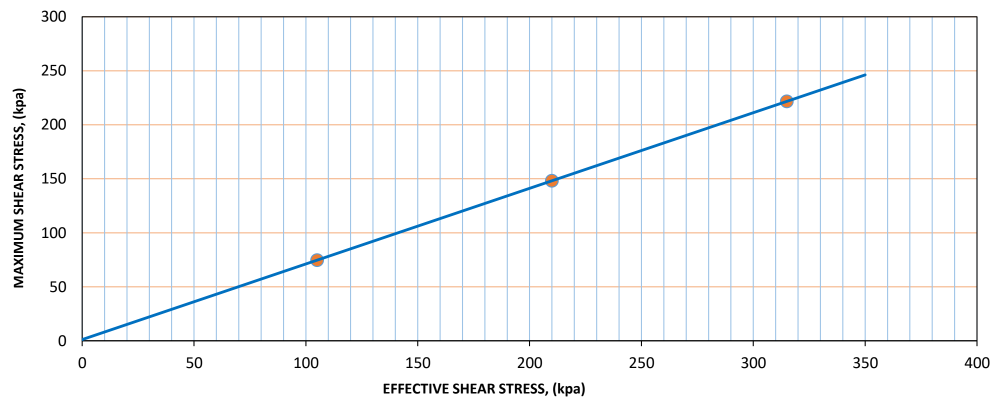
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)		**DATE OF TEST:**	**02/11/2023**
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH		**TEST METHOD:**	**ASTM D 3080**
LOCATION: KAPTAI, RANGAMATI, BANGLADESH			

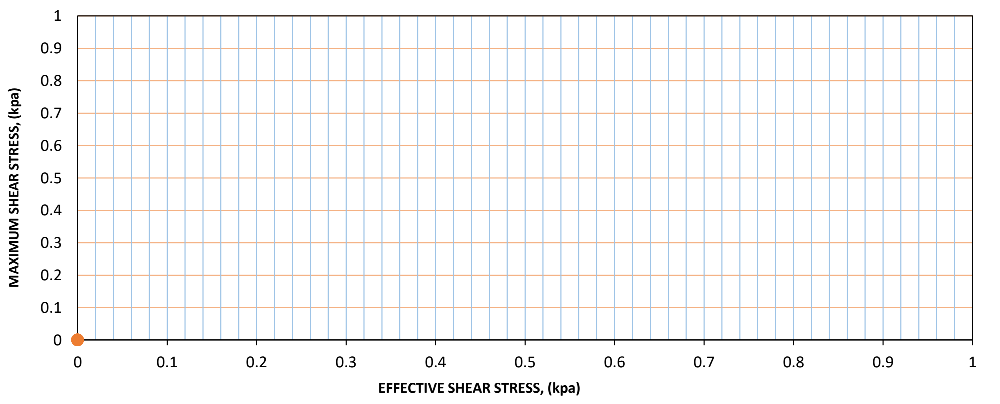
DIRECT SHEAR TEST



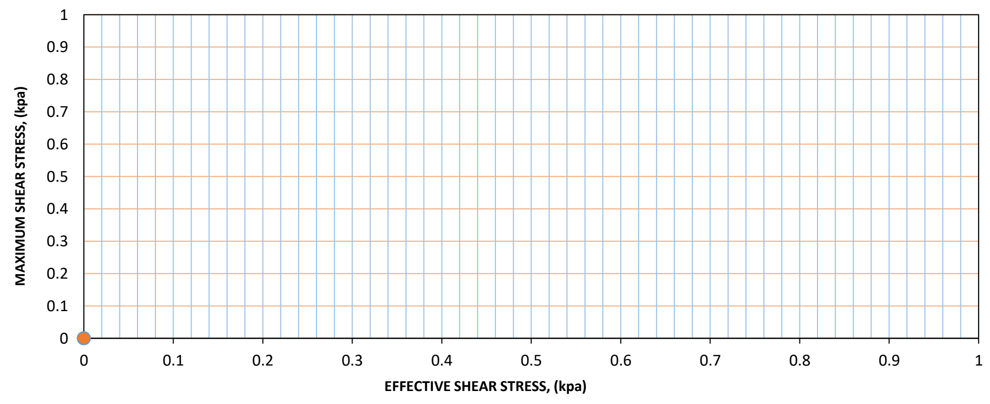
BH. NO:	1	DEPTH (m)	19.5	ANGLE OF INTERNAL FRICTION, φ (deg)	26.9	COHESION, C (kpa)	1.14



BH. NO:	3	DEPTH (m)	10.5	ANGLE OF INTERNAL FRICTION, φ (deg)	35	COHESION, C (kpa)	1.13



BH. NO:	0	DEPTH (m)	0	ANGLE OF INTERNAL FRICTION, φ (deg)		COHESION, C (kpa)	0



BH. NO:	0	DEPTH (m)	0	ANGLE OF INTERNAL FRICTION, φ (deg)		COHESION, C (kpa)	0

APPENDIX-2E

UNCONFINED COMPRESSION STRENGTH TEST CURVES

Geoscape Consultants Ltd.

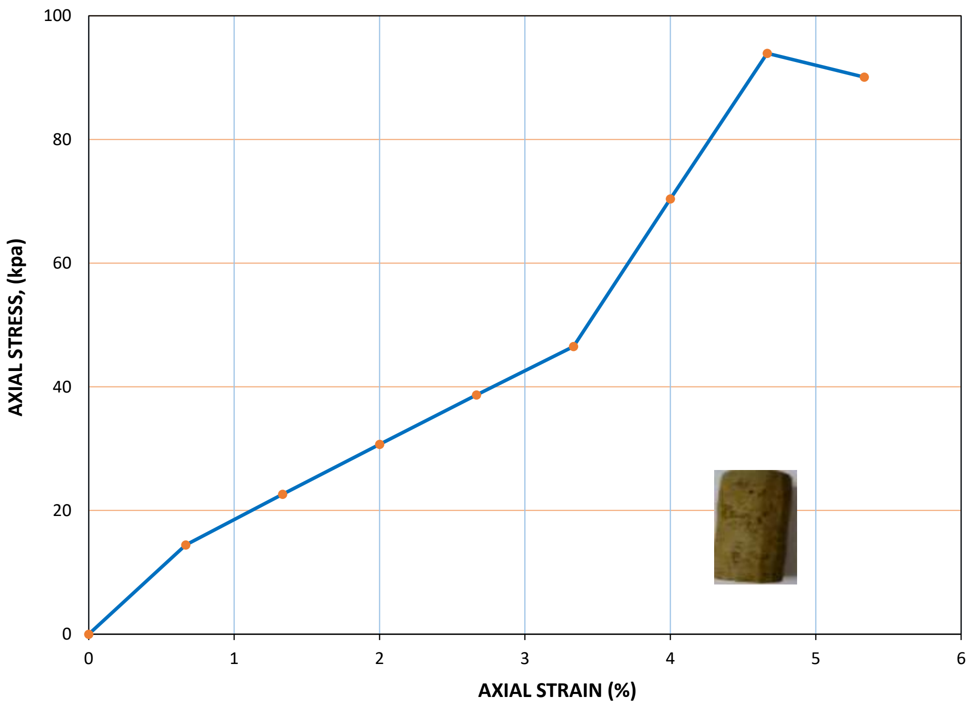
Geoscape Consultants Ltd.

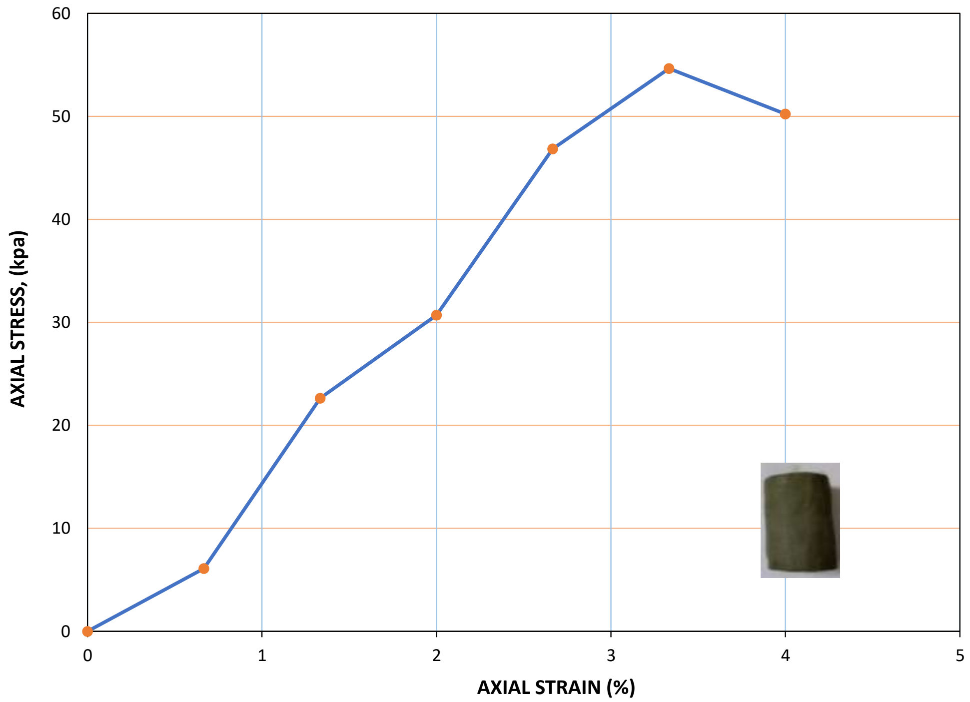
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)		DATE OF TEST:	12/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD:	ASTM D2166
LOCATION: KAPTAI, RANGAMATI, BANGLADESH			

UNCONFINED COMPRESSION STRENGTH TEST

BH NO.	1	DEPTH (m)	2.44	BH NO.	2	DEPTH (m)	3.96





UNCONFINED COMPRESSION STRENGTH, q_u (kPa)	93.94	UNCONFINED COMPRESSION STRENGTH, q_u (kPa)	54.65
UNDRAINED SHEAR STRENGTH, Su (kPa)	46.97	UNDRAINED SHEAR STRENGTH, Su (kPa)	27.33
MOISTURE CONTENT (%)	22.77	MOISTURE CONTENT (%)	23.98

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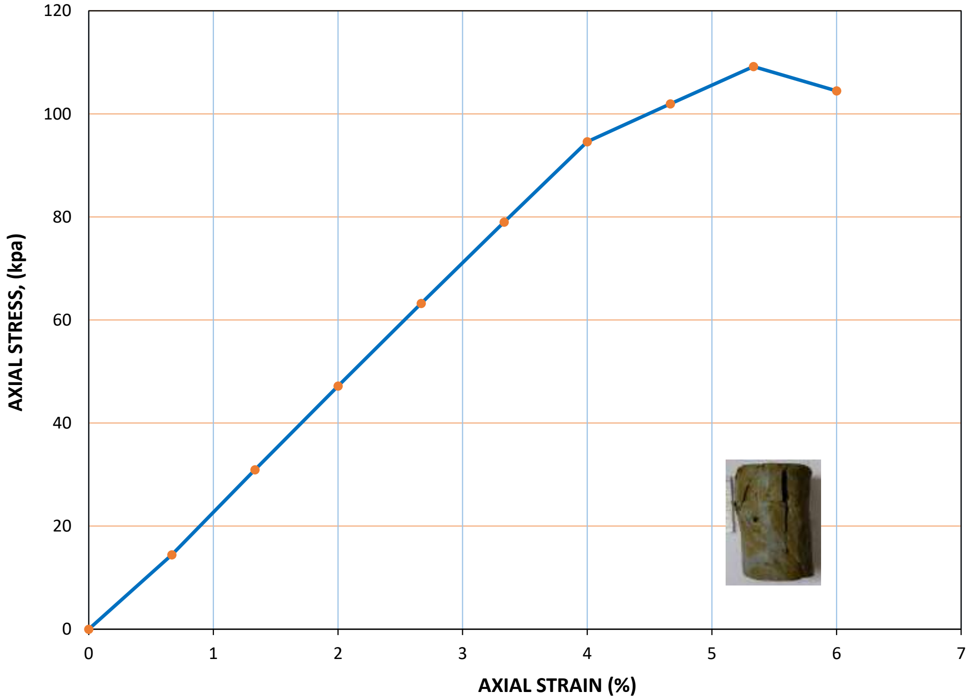
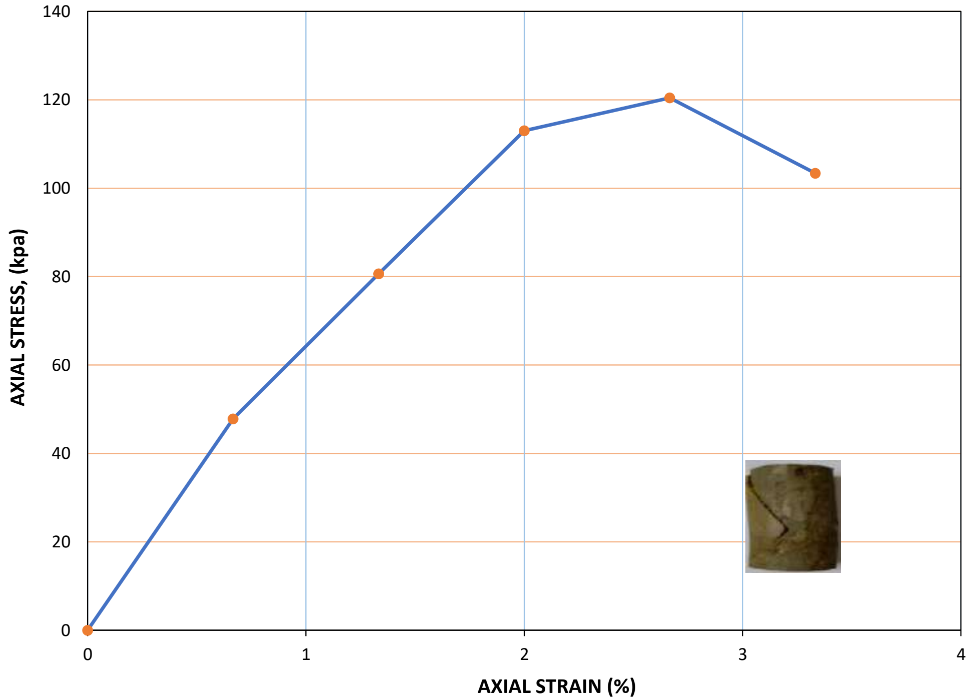
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)						DATE OF TEST:	12/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH						TEST METHOD:	ASTM D2166
LOCATION: KAPTAI, RANGAMATI, BANGLADESH							
UNCONFINED COMPRESSION STRENGTH TEST							
BH NO.	3	DEPTH (m)	8.53	BH NO.	4	DEPTH (m)	2.44





UNCONFINED COMPRESSION STRENGTH, q_u (kPa)	109.18	UNCONFINED COMPRESSION STRENGTH, q_u (kPa)	120.44
UNDRAINED SHEAR STRENGTH, Su (kPa)	54.59	UNDRAINED SHEAR STRENGTH, Su (kPa)	60.22
MOISTURE CONTENT (%)	23.31	MOISTURE CONTENT (%)	23.47

APPENDIX-2F

CONSOLIDATION TEST CURVES

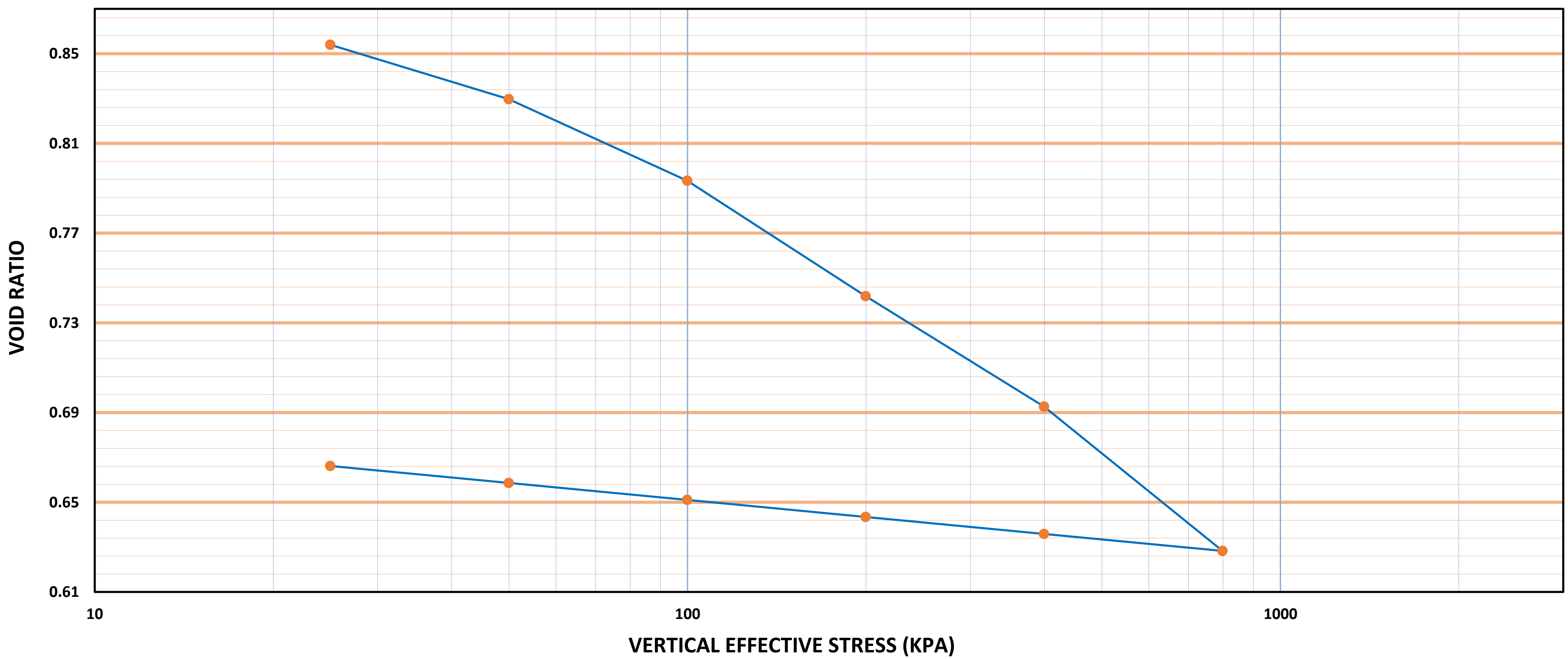
Geoscape Consultants Ltd.

House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD(BPDB)	DATE OF BORING:	03/11/2023
PROJECT NAME: CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC)	DATE OF TEST:	10/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD:	ASTM D2435

CONSOLIDATION TEST

BH NO.	1	SAMPLE NO.	U-01	DEPTH	2.44m



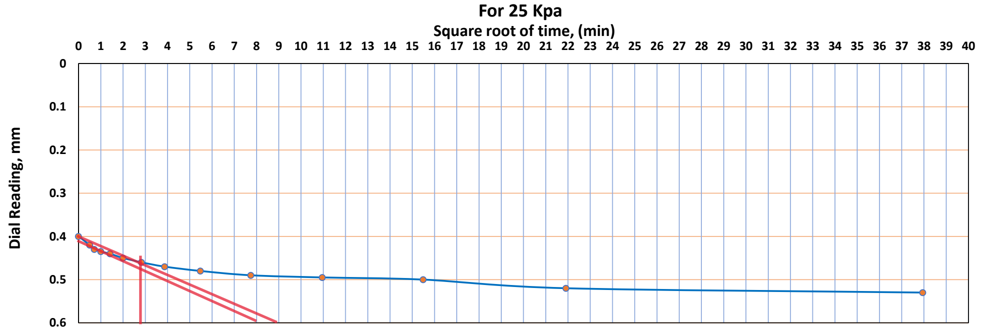
OVERBURDEN PRESSURE AT SITE, P_0	36.409 kpa	INITIAL VOID RATIO, e_0	0.854
SWELLING INDEX, Cs	0.025	COMPRESSION INDEX, C_c	0.214

Geoscape Consultants Ltd.

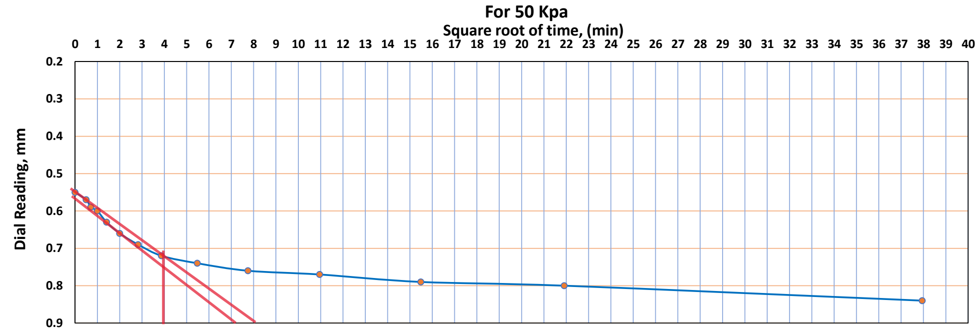
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD(BPDB)		DATE OF BORING:	03/11/2023
PROJECT NAME: CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC)		DATE OF TEST:	10/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD:	ASTM D2435

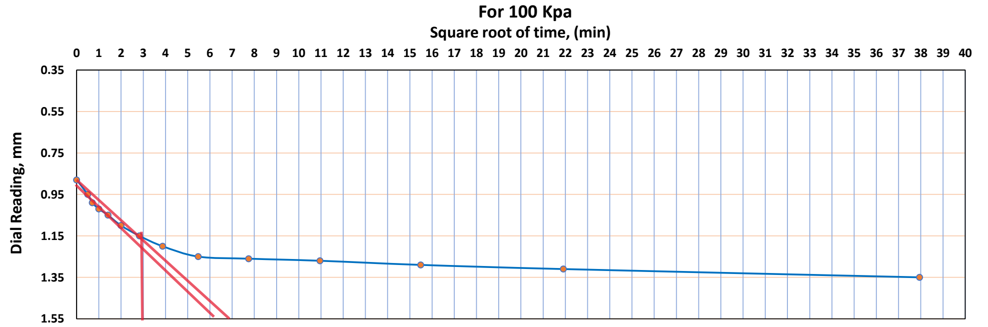
SQUARE ROOT FITTING CURVE



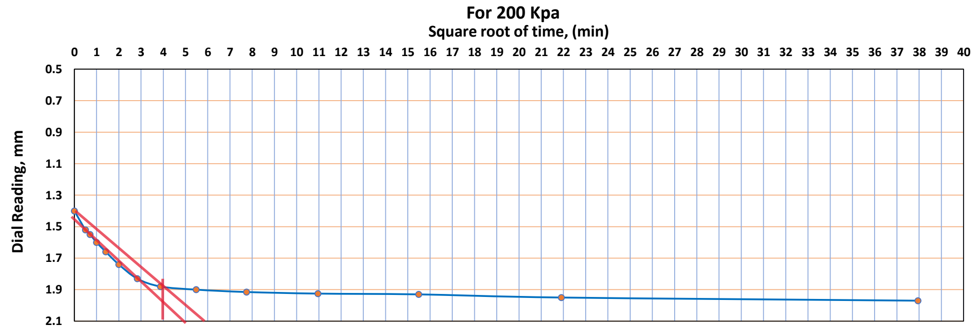
BH. NO:	1	DEPTH (m)	2.44m	t_{90}	8.41	min



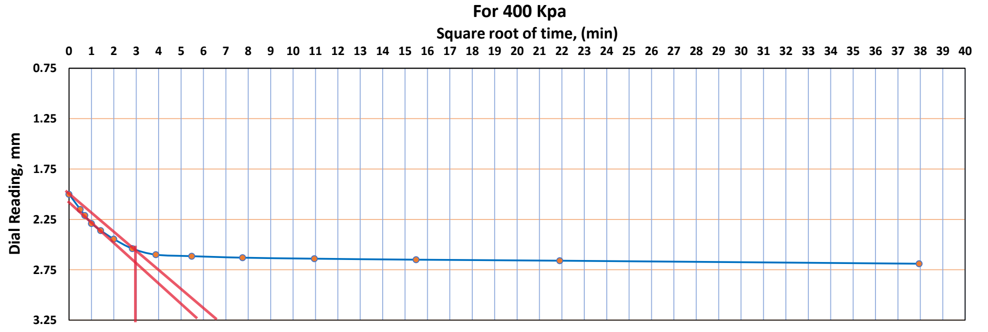
BH. NO:	1	DEPTH (m)	2.44m	t_{90}	16.00	min



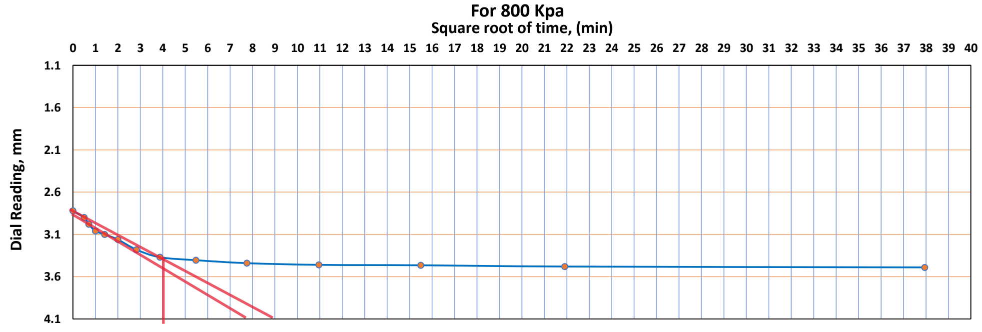
BH. NO:	1	DEPTH (m)	2.44m	t_{90}	9.00	min



BH. NO:	1	DEPTH (m)	2.44m	t_{90}	16.00	min



BH. NO:	1	DEPTH (m)	2.44m	t_{90}	9.00	min



BH. NO:	1	DEPTH (m)	2.44m	t_{90}	16.00	min

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House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

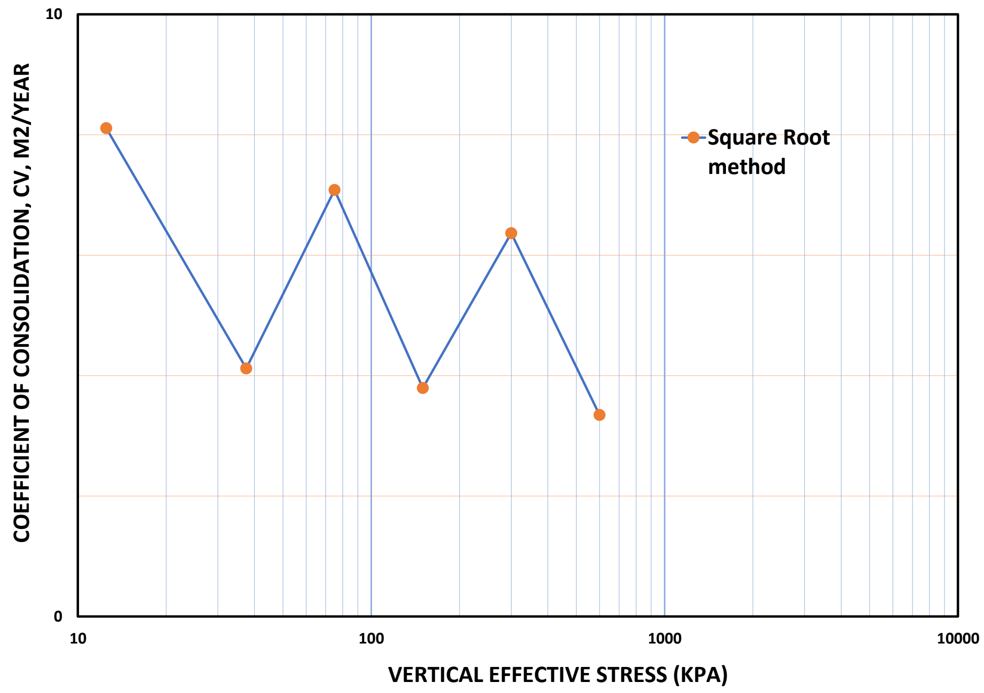
CLIENT: BANGLADESH POWER DEVELOPMENT BOARD(BPDB)	DATE OF BORING: 03/11/2023
PROJECT NAME: CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	DATE OF TEST: 10/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD: ASTM D2435

CV CURVE DATA FROM SQUARE ROOT METHOD

Pressure, (Kpa)	Average Pressure, (Kpa)	2H (mm)	H = $(H_1+H_2)/2$ (mm)	Square Root Method	
				t_{90} (min)	C_v (m^2/year)
0		25			
25	12.5	24.48	12.37	8.41	8.11
50	37.5	24.16	12.16	16.00	4.12
100	75	23.68	11.96	9.00	7.08
200	150	23.00	11.67	16.00	3.79
400	300	22.35	11.34	9.00	6.37
800	600	21.50	10.96	16.00	3.35

BH. NO:	1	DEPTH (m)	2.44m

COEFFICIENT OF CONSOLIDATION, CV VS VERTICAL EFFECTIVE STRESS CURVE



BH. NO:	1	DEPTH (m)	2.44m

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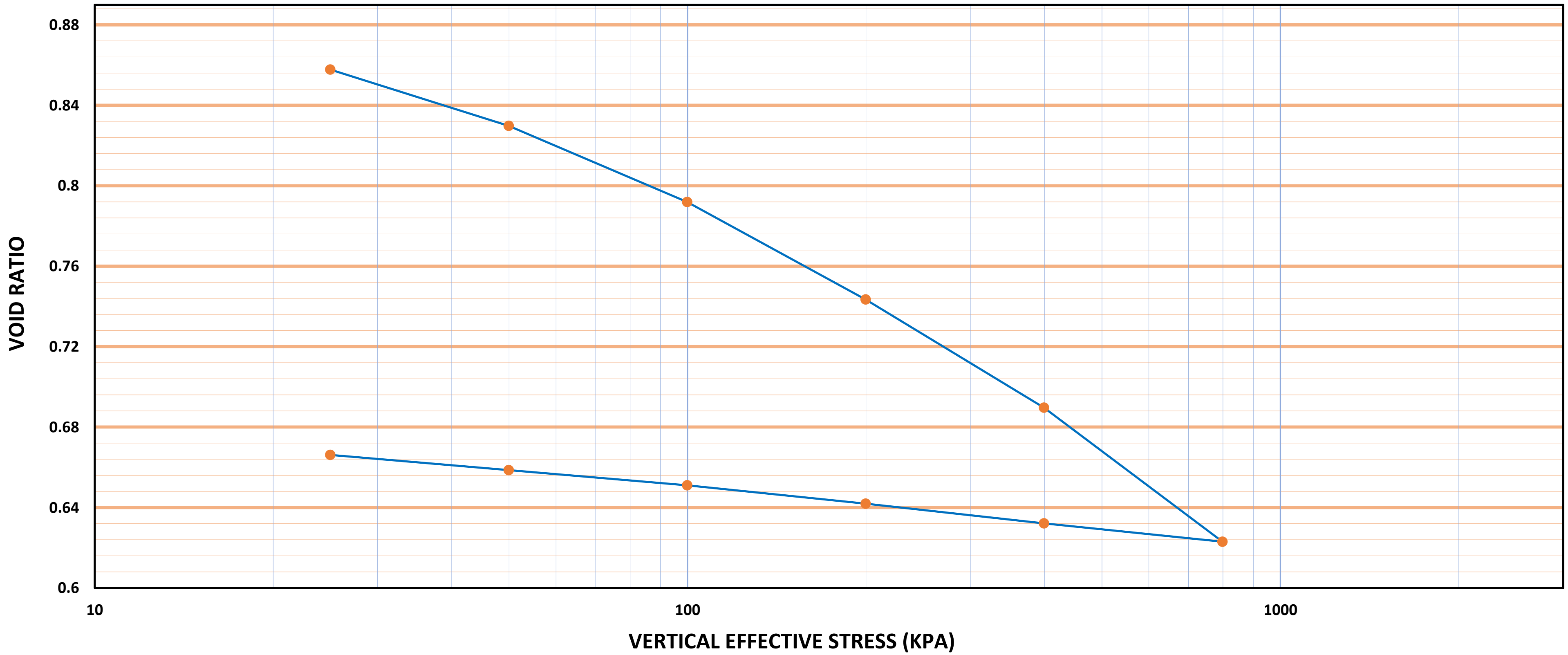
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD(BPDB)		DATE OF BORING:	05/11/2023
PROJECT NAME: CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC)		DATE OF TEST:	10/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD:	ASTM D2435

CONSOLIDATION TEST

BH NO.	2	SAMPLE NO.	U-01	DEPTH	3.96m



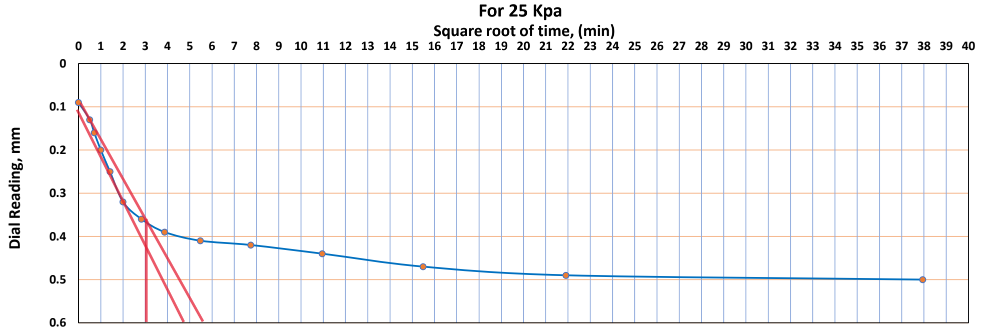
OVERBURDEN PRESSURE AT SITE, P_0	70.219 kpa	INITIAL VOID RATIO, e_0	0.858
SWELLING INDEX, Cs	0.030	COMPRESSION INDEX, C_c	0.221

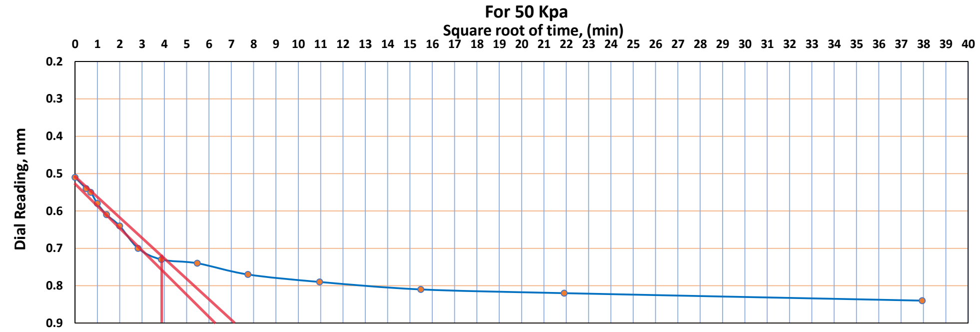
Geoscape Consultants Ltd.

House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

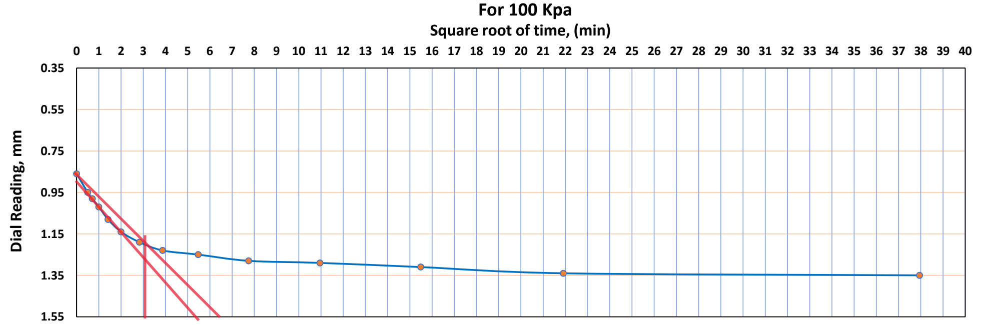
CLIENT: BANGLADESH POWER DEVELOPMENT BOARD(BPDB)		DATE OF BORING:	05/11/2023
PROJECT NAME: CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC)		DATE OF TEST:	10/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH		TEST METHOD:	ASTM D2435

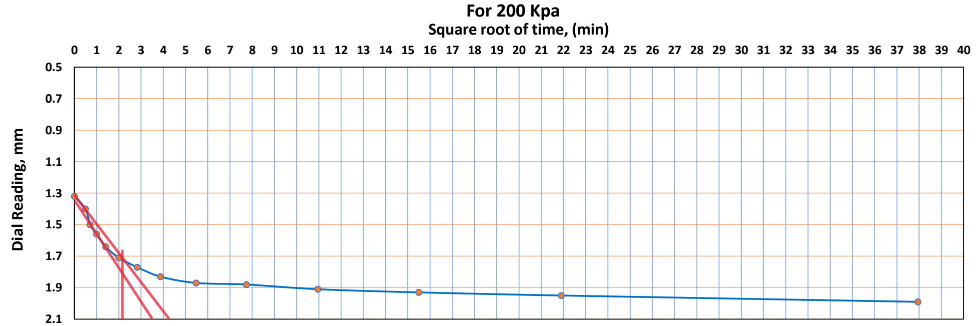
SQUARE ROOT FITTING CURVE



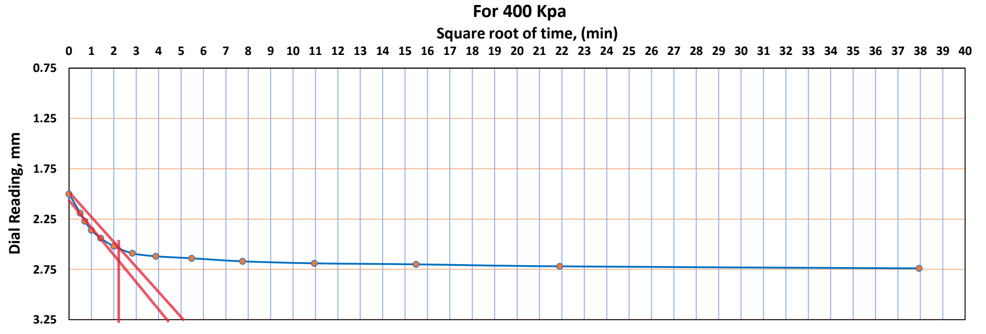


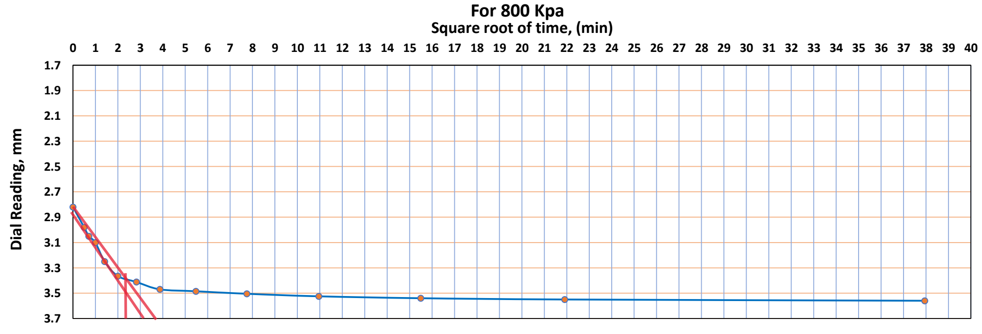
BH. NO:	2	DEPTH (m)	3.96m	t_{90}	9.00	min
BH. NO:	2	DEPTH (m)	3.96m	t_{90}	16.00	min





BH. NO:	2	DEPTH (m)	3.96m	t_{90}	9.00	min
BH. NO:	2	DEPTH (m)	3.96m	t_{90}	4.41	min





BH. NO:	2	DEPTH (m)	3.96m	t_{90}	4.41	min
BH. NO:	2	DEPTH (m)	3.96m	t_{90}	4.84	min

Geoscape Consultants Ltd.

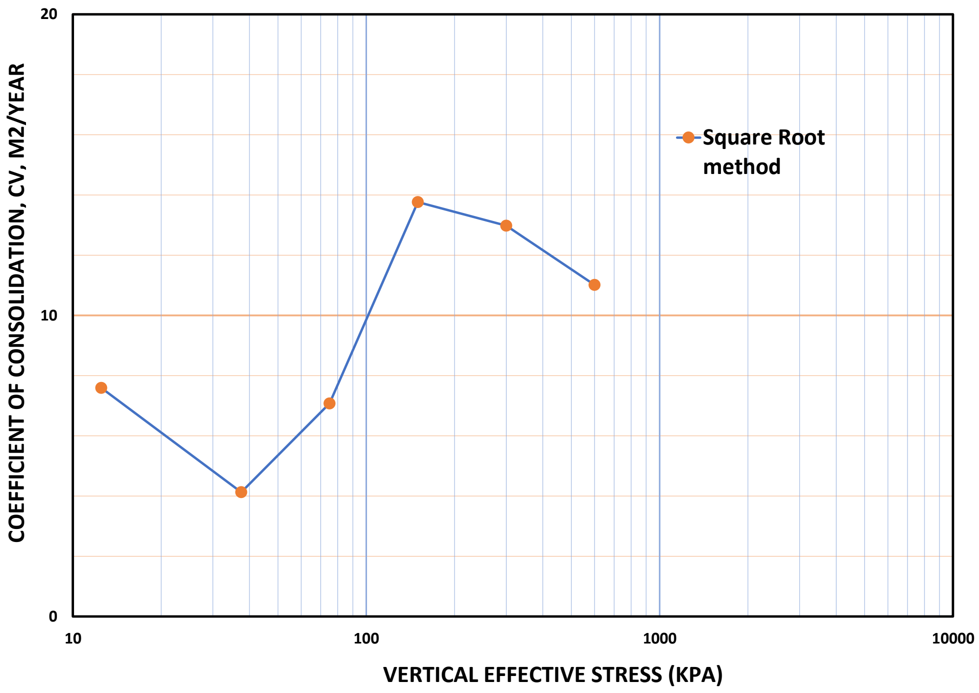
House No. #135, Road No. #05, Mohakhali DOHS, Dhaka

CLIENT: BANGLADESH POWER DEVELOPMENT BOARD(BPDB)	DATE OF BORING:	05/11/2023
PROJECT NAME: CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH	DATE OF TEST:	10/11/2023
LOCATION: KAPTAI, RANGAMATI, BANGLADESH	TEST METHOD:	ASTM D2435

CV CURVE DATA FROM SQUARE ROOT METHOD

Pressure, (Kpa)	Average Pressure, (Kpa)	2H (mm)	H = $(H_1+H_2)/2$ (mm)	Square Root Method	
				t_{90} (min)	C_v (m^2/year)
0		25			
25	12.5	24.53	12.38	9.00	7.59
50	37.5	24.16	12.17	16.00	4.13
100	75	23.66	11.96	9.00	7.08
200	150	23.02	11.67	4.41	13.76
400	300	22.31	11.33	4.41	12.98
800	600	21.43	10.94	4.84	11.01
BH. NO:		2	DEPTH (m)		3.96m

COEFFICIENT OF CONSOLIDATION, CV VS VERTICAL EFFECTIVE STRESS CURVE



BH. NO:	2	DEPTH (m)	3.96m

APPENDIX-2G

SPECIFIC GRAVITY TEST

GEOSCAPE Consultants Ltd.

Geoscape Consultants Ltd. House No. #135, Road No. #05, Mohakhali DOHS, Dhaka							
CLIENT: BANGLADESH POWER DEVELOPMENT BOARD (BPDB)						DATE OF TEST:	12/11/2023
PROJECT : CONSULTANCY SERVICES FOR DETAILED FEASIBILITY STUDY FOR CONSTRUCTION OF 7.6 MWP (DC) SOLAR PHOTOVOLTAIC GRID-CONNECTED POWER PLANT AT KAPTAI, RANGAMATI, BANGLADESH						TEST METHOD:	ASTM D 854
LOCATION: KAPTAI, RANGAMATI, BANGLADESH							
SPECIFIC GRAVITY CALCULATION							
BH. NO:	**1**	**DEPTH (m)**	**1.5**	**BH. NO:**	**2**	**DEPTH (m)**	**6**
Weight of pycnometer, (gm)		M_1 (gm)=	162.5	Weight of pycnometer, (gm)		M_1 (gm)=	162.5
Weight of pycnometer+Soil, (gm)		M_2 (gm)=	247.9	Weight of pycnometer+Soil, (gm)		M_2 (gm)=	248.1
Weight of pycnometer+Soil+water, (gm)		M_3(gm)=	709.4	Weight of pycnometer+Soil+water, (gm)		M_3(gm)=	709.4
Weight of pycnometer+Water, (gm)		M_4 (gm)=	655.5	Weight of pycnometer+Water, (gm)		M_4 (gm)=	655.5
Specific Gravity of Soil, (at 20 Deg)		G_{20} =	**2.71**	Specific Gravity of Soil, (at 20 Deg)		G_{20} =	**2.70**
BH. NO:	**3**	**DEPTH (m)**	**3**	**BH. NO:**	**4**	**DEPTH (m)**	**4.5**
Weight of pycnometer, (gm)		M_1 (gm)=	162.5	Weight of pycnometer, (gm)		M_1 (gm)=	162.5
Weight of pycnometer+Soil, (gm)		M_2 (gm)=	248.6	Weight of pycnometer+Soil, (gm)		M_2 (gm)=	247.8
Weight of pycnometer+Soil+water, (gm)		M_3(gm)=	709.4	Weight of pycnometer+Soil+water, (gm)		M_3(gm)=	709.4
Weight of pycnometer+Water, (gm)		M_4 (gm)=	655.5	Weight of pycnometer+Water, (gm)		M_4 (gm)=	655.5
Specific Gravity of Soil, (at 20 Deg)		G_{20} =	**2.67**	Specific Gravity of Soil, (at 20 Deg)		G_{20} =	**2.72**

APPENDIX-2H

TEST SUMMARY

TEST SUMMARY FOR BH 01

DEPTH (m)	SPT	USCS	UD DEPTH	MOISTURE CONTENT (%)	ATTERBERG LIMIT (%)			GRAIN SIZE ANALYSIS			SPECIFIC GRAVITY (FOR 20° C)	DIRECT SHEAR TEST		UNCONFINED COMPRESSION STRENGTH	CONSOLIDATION TEST	
					LL	PL	PI	% SAND	% SILT	% CLAY		Ø (deg)	C (Kpa)	q (kpa)	C_c	e_0
1.5	8	CL	Undisturbed samples collected at: 2.44 m	20.96							2.71			93.94	0.214	0.854
3	2	CL		19.27												
4.5	5	CL		29.34	36	22.39	13.61	0	68	32						
6	4	CL		27.62												
7.5	5	CL		28.49												
9	5	CL		28.72												
10.5	8	CL		21.71												
12	9	ML		18.92				70	21	9						
13.5	21	ML		22.88												
15	34	ML		17.46												
16.5	24	ML		19.50				80	15	5						
18	38	ML		21.05												
19.5	23	ML		22.52								26.9	1.14			
21	23	ML		20.09												
22.5	50	CL-ML		17.61												
24	50	CL-ML		21.95	28	22.32	5.68	0	81	19						
25.5	50	CL-ML		22.17												

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DEPTH (m)	SPT	USCS	UD DEPTH	MOISTURE CONTENT (%)	ATTERBERG LIMIT (%)			GRAIN SIZE ANALYSIS			SPECIFIC GRAVITY (FOR 20° C)	DIRECT SHEAR TEST		UNCONFINED COMPRESSION STRENGTH	CONSOLIDATION TEST	
					LL	PL	PI	% SAND	% SILT	% CLAY		Ø (deg)	C (Kpa)	q (kpa)	C_c	e_0
1.5	6	CL	Undisturbed samples collected at: 3.96 m	23.04												
3	1	CL		22.70										54.65	0.221	0.858
4.5	3	CL		21.65												
6	4	CL		23.90							2.7					
7.5	4	CL		27.71	35	21.26	13.74	0	64	36						
9	6	CL		32.26												
10.5	5	CL		33.33												
12	7	ML		29.68												
13.5	7	ML		35.46												
15	9	ML		35.19	39	26.96	12.04	0	76	24						

TEST SUMMARY FOR BH 03

DEPTH (m)	SPT	USCS	UD DEPTH	MOISTURE CONTENT (%)	ATTERBERG LIMIT (%)			GRAIN SIZE ANALYSIS			SPECIFIC GRAVITY (FOR 20° C)	DIRECT SHEAR TEST		UNCONFINED COMPRESSION STRENGTH	CONSOLIDATION TEST	
					LL	PL	PI	% SAND	% SILT	% CLAY		Ø (deg)	C (Kpa)	q (kpa)	C_c	e_0
1.5	2	SM	Undisturbed samples collected at: 8.53 m	21.18				78	15	7						
3	2	SM		22.79							2.67					
4.5	3	CL-ML		23.00												
6	6	CL-ML		20.78	21	14.62	6.38	0	71	29						
7.5	8	CL		24.02	45	25.69	19.31	0	55	45				109.18		
9	46	CL		20.71	55	31.58	23.42	0	49	51						
10.5	50	SM		17.06								35	1.13			
12	50	SM		16.25				76	19	5						
13.5	50	ML		15.94												
15	50	ML		16.78	33	25.17	7.83	0	71	29						

TEST SUMMARY FOR BH 04																
DEPTH (m)	SPT	USCS	UD DEPTH	MOISTURE CONTENT (%)	ATTERBERG LIMIT (%)			GRAIN SIZE ANALYSIS			SPECIFIC GRAVITY (FOR 20° C)	DIRECT SHEAR TEST		UNCONFINED COMPRESSION STRENGTH	CONSOLIDATION TEST	
					LL	PL	PI	% SAND	% SILT	% CLAY		Ø (deg)	C (Kpa)	q (kpa)	C_c	e_0
1.5	6	CH	Undisturbed samples collected at: 2.44 m	21.17										120.44		
3	8	CH		22.36	54	35.65	18.35	0	62	38						
4.5	7	CH		27.18							2.72					
6	10	CH		23.73	48	23.48	24.52	0	67	33						
7.5	7	CH		31.94	54	23.42	30.58	0	66	34						
9	8	CH		32.91												
10.5	8	ML		34.39												
12	4	ML		26.00												
13.5	5	ML		32.48	36	27.12	8.88	0	79	21						
15	6	ML		34.74												

APPENDIX-03

CALCULATION OF SOIL BEARING CAPACITY

APPENDIX-3A

BEARING CAPACITY FOR SHALLOW FOUNDATION

BEARING CAPACITY BASED ON SPT

BEARING CAPACITY FOR ISOLATED FOOTING BASED ON SPT VALUES FOR BH NO 01																
FOUNDATION AND SOIL PARAMETERS													SAFE BEARING CAPACITY FOR COHESIVE SOIL (FOR FOS 2.5)			ALLOWABLE BEARING CAPACITY FOR GRANULAR SOIL
													ACCORDING TO MEYERHOF	ACCORDING TO HANSEN	ACCORDING TO VESIC	ACCORDING TO MEYERHOF (1974)
L (m)	B (m)	D_f (m)	D_f+B (m)	Field SPT-N value	Effective overburden pressure at (D_f+B) depth, σ_0' (kN/m^2)	SPT- N_{60}	Unit weight, γ (kN/m^3)	Water level (m)	Rw_1	Rw_2	k_d	q (kN/m^2)	q_{safe} (ksf)	q_{safe} (ksf)	q_{safe} (ksf)	$q_{allowable}$ (ksf)
3	3	1.5	4.5	8	11.535	7	17.5	0	0.5	0.5	1.2	11.54	2.431	2.6441	2.6441	-
3	3	3	6	2	15.57	2	15	0	0.5	0.5	1.3	15.57	0.7077	0.8324	0.8324	-
3	3	4.5	7.5	5	27.855	5	16	0	0.5	0.5	1.3	27.86	1.9765	2.1162	2.1162	-
The safe bearing capacity (for cohesive soil) values mentioned above have been calculated for a 3m x 3m footing of various depth. The allowable bearing capacity of foundation depends on its settlement criteria, size and shape. It is highly recommended to estimate the settlement value to re-evaluate the allowable bearing capacity value. It is also suggested to mitigate the liquefaction issues (if the soil is liquefiable).																

BEARING CAPACITY FOR ISOLATED FOOTING
BASED ON SPT VALUES FOR BH NO 02

FOUNDATION AND SOIL PARAMETERS													SAFE BEARING CAPACITY FOR COHESIVE SOIL (FOR FOS 2.5)			ALLOWABLE BEARING CAPACITY FOR GRANULAR SOIL
													ACCORDING TO MEYERHOF	ACCORDING TO HANSEN	ACCORDING TO VESIC	ACCORDING TO MEYERHOF (1974)
L (m)	B (m)	D_f (m)	D_f+B (m)	Field SPT-N value	Effective overburden pressure at (D_f+B) depth, σ_0' (kN/m^2)	SPT- N_{60}	Unit weight, γ (kN/m^3)	Water level (m)	Rw_1	Rw_2	k_d	q (kN/m^2)	q_{safe} (ksf)	q_{safe} (ksf)	q_{safe} (ksf)	$q_{allowable}$ (ksf)
3	3	1.5	4.5	6	10.035	5	16.5	0	0.5	0.5	1.2	10.04	1.7289	1.8811	1.8811	-
3	3	3	6	1	15.57	1	15	0	0.5	0.5	1.3	15.57	0.3213	0.3837	0.3837	-
3	3	4.5	7.5	3	25.605	3	15.5	0	0.5	0.5	1.3	25.61	1.1488	1.2326	1.2326	-

The safe bearing capacity (for cohesive soil) and allowable bearing capacity (for granular soil) values mentioned above have been calculated for a 3m x 3m footing of various depth. The allowable bearing capacity of foundation depends on its settlement criteria, size and shape. It is highly recommended to estimate the settlement value to re-evaluate the allowable bearing capacity value. It is also suggested to mitigate the liquefaction issues (if the soil is liquefiable).

BEARING CAPACITY FOR ISOLATED FOOTING BASED ON SPT VALUES FOR BH NO 03																
FOUNDATION AND SOIL PARAMETERS													SAFE BEARING CAPACITY FOR COHESIVE SOIL (FOR FOS 2.5)			ALLOWABLE BEARING CAPACITY FOR GRANULAR SOIL
													ACCORDING TO MEYERHOF	ACCORDING TO HANSEN	ACCORDING TO VESIC	ACCORDING TO MEYERHOF (1974)
L (m)	B (m)	D_f (m)	D_f+B (m)	Field SPT-N value	Effective overburden pressure at (D_f+B) depth, σ_0' (kN/m^2)	SPT- N_{60}	Unit weight, γ (kN/m^3)	Water level (m)	Rw_1	Rw_2	k_d	q (kN/m^2)	q_{safe} (ksf)	q_{safe} (ksf)	q_{safe} (ksf)	$q_{allowable}$ (ksf)
3	3	1.5	4.5	2	7.785	2	15	0	0.5	0.5	1.2	7.785	-	-	-	0.7358373
3	3	3	6	2	15.57	2	15	0	0.5	0.5	1.3	15.57	-	-	-	0.8400546
3	3	4.5	7.5	3	25.605	3	15.5	0	0.5	0.5	1.3	25.61	1.1488	1.2326	1.2326	-

The safe bearing capacity (for cohesive soil) and allowable bearing capacity (for granular soil) values mentioned above have been calculated for a 3m x 3m footing of various depth. The allowable bearing capacity of foundation depends on its settlement criteria, size and shape. It is highly recommended to estimate the settlement value to re-evaluate the allowable bearing capacity value. It is also suggested to mitigate the liquefaction issues (if the soil is liquefiable).

BEARING CAPACITY FOR ISOLATED FOOTING
BASED ON SPT VALUES FOR BH NO 04

FOUNDATION AND SOIL PARAMETERS													SAFE BEARING CAPACITY FOR COHESIVE SOIL (FOR FOS 2.5)			ALLOWABLE BEARING CAPACITY FOR GRANULAR SOIL
													ACCORDING TO MEYERHOF	ACCORDING TO HANSEN	ACCORDING TO VESIC	ACCORDING TO MEYERHOF (1974)
L (m)	B (m)	D_f (m)	D_f+B (m)	Field SPT-N value	Effective overburden pressure at (D_f+B) depth, σ_0' (kN/m^2)	SPT- N_{60}	Unit weight, γ (kN/m^3)	Water level (m)	Rw_1	Rw_2	k_d	q (kN/m^2)	q_{safe} (ksf)	q_{safe} (ksf)	q_{safe} (ksf)	$q_{allowable}$ (ksf)
3	3	1.5	4.5	6	10.035	5	16.5	0	0.5	0.5	1.2	10.04	1.7289	1.8811	1.8811	-
3	3	3	6	8	23.07	8	17.5	0	0.5	0.5	1.3	23.07	2.9946	3.4933	3.4933	-
3	3	4.5	7.5	7	32.355	7	17	0	0.5	0.5	1.3	32.36	2.7948	2.9904	2.9904	-

The safe bearing capacity (for cohesive soil) values mentioned above have been calculated for a 3m x 3m footing of various depth. The allowable bearing capacity of foundation depends on its settlement criteria, size and shape. It is highly recommended to estimate the settlement value to re-evaluate the allowable bearing capacity value. It is also suggested to mitigate the liquefaction issues (if the soil is liquefiable).

BEARING CAPACITY BASED ON UNCONFINED COMPRESSION STRENGTH

GEOSCAPE Consultants Ltd.

BEARING CAPACITY FOR ISOLATED FOOTING BASED ON UNCONFINED COMPRESSIVE STRENGTH TEST FOR BH NO 01											
FOUNDATION AND SOIL PARAMETERS									SAFE BEARING CAPACITY FOR COHESIVE SOIL (FOR FOS 2.5)		
									ACCORDING TO MEYERHOF	ACCORDING TO HANSEN	ACCORDING TO VESIC
L (m)	B (m)	D_f (m)	Effective overburden pressure at (D_f+B) depth, σ_0' (kN/m^2)	Water level (m)	Rw_1	Rw_2	k_d	Cohesion, C_u (kN/m^2)	q_{safe} (ksf)	q_{safe} (ksf)	q_{safe} (ksf)
3	3	2.44	24.8636	0	0.5	0.5	1.268	46.97	2.70943454	3.08847578	3.08847578
The safe bearing capacity (for cohesive soil) values mentioned above have been calculated for a 3m x 3m footing of 2.44m depth.The allowable bearing capacity of foundation depends on its settlement criteria, size and shape. It is highly recommended to estimate the settlement value to re-evaluate the allowable bearing capacity value. It is also suggested to mitigate the liquefaction issues (if the soil is liquefiable).											

GEOSCAPE Consultants Ltd.

BEARING CAPACITY FOR ISOLATED FOOTING BASED ON UNCONFINED COMPRESSIVE STRENGTH TEST FOR BH NO 02											
FOUNDATION AND SOIL PARAMETERS									SAFE BEARING CAPACITY FOR COHESIVE SOIL (FOR FOS 2.5)		
									ACCORDING TO MEYERHOF	ACCORDING TO HANSEN	ACCORDING TO VESIC
L (m)	B (m)	D_f (m)	Effective overburden pressure at (D_f+B) depth, σ_0' (kN/m^2)	Water level (m)	Rw_1	Rw_2	k_d	Cohesion, C_u (kN/m^2)	q_{safe} (ksf)	q_{safe} (ksf)	q_{safe} (ksf)
3	3	3.96	40.3524	0	0.5	0.5	1.33	27.33	1.61108493	1.75014568	1.75014568

The safe bearing capacity (for cohesive soil) values mentioned above have been calculated for a 3m x 3m footing of 3.96m depth.The allowable bearing capacity of foundation depends on its settlement criteria, size and shape. It is highly recommended to estimate the settlement value to re-evaluate the allowable bearing capacity value. It is also suggested to mitigate the liquefaction issues (if the soil is liquefiable).

GEOSCAPE Consultants Ltd.

BEARING CAPACITY FOR ISOLATED FOOTING BASED ON UNCONFINED COMPRESSIVE STRENGTH TEST FOR BH NO 04											
FOUNDATION AND SOIL PARAMETERS									SAFE BEARING CAPACITY FOR COHESIVE SOIL (FOR FOS 2.5)		
									ACCORDING TO MEYERHOF	ACCORDING TO HANSEN	ACCORDING TO VESIC
L (m)	B (m)	D_f (m)	Effective overburden pressure at (D_f+B) depth, σ_0' (kN/m^2)	Water level (m)	Rw_1	Rw_2	k_d	Cohesion, C_u (kN/m^2)	q_{safe} (ksf)	q_{safe} (ksf)	q_{safe} (ksf)
3	3	2.44	24.8636	0	0.5	0.5	1.268	60.22	3.50304238	3.98900925	3.98900925

The safe bearing capacity (for cohesive soil) values mentioned above have been calculated for a 3m x 3m footing of 2.44m depth.The allowable bearing capacity of foundation depends on its settlement criteria, size and shape. It is highly recommended to estimate the settlement value to re-evaluate the allowable bearing capacity value. It is also suggested to mitigate the liquefaction issues (if the soil is liquefiable).

APPENDIX-3B

BEARING CAPACITY FOR DEEP FOUNDATION

PRECAST PILE CAPACITY

PRECAST PILE CAPACITY, SIZE = 16''×16'' FOR BH 01

PILE DETAILS INPUTS: Pile depth (m)	SPT-N_{field} Per layer	A_s (m^2)	A_{bottom} (m^2)	ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020): COHESIVE SOIL SKIN FRICTION Q_S (kN)	COHESIVE SOIL END BEARING Q_B (kN)	NON-COHESIVE SOIL SKIN FRICTION Q_s (kN)	NON-COHESIVE SOIL END BEARING Q_B (kN)	TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-1: Q_{ult} kN	Method-1: Q_{ult} Ton	Method-1: Q_{ult} kip	ALPHA METHOD FOR COHESIVE SOIL AND NORDLUND METHOD FOR NON-COHESIVE SOIL (ADOPTED BY AASHTO-2014): NON-COHESIVE SOIL SKIN FRICTION Q_s (kN)	NON-COHESIVE SOIL END BEARING Q_B (kN)	TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-2: Q_{ult} kN	Method-2: Q_{ult} Ton	Method-2: Q_{ult} kip
1.5	8	2.4	0.2	78.4	66.9	0.0	0.0	78.4	66.9	145.3	14.5	**32.6**	0.0	0.0	78.4	66.9	145.3	14.5	**32.6**
3	2	2.4	0.2	105.8	16.7	0.0	0.0	105.8	16.7	122.6	12.3	**27.5**	0.0	0.0	105.8	16.7	122.6	12.3	**27.5**
4.5	5	2.4	0.2	171.4	41.8	0.0	0.0	171.4	41.8	213.2	21.3	**47.8**	0.0	0.0	171.4	41.8	213.2	21.3	**47.8**
6	4	2.4	0.2	226.2	33.5	0.0	0.0	226.2	33.5	259.7	26.0	**58.2**	0.0	0.0	226.2	33.5	259.7	26.0	**58.2**
7.5	5	2.4	0.2	291.8	41.8	0.0	0.0	291.8	41.8	333.6	33.4	**74.7**	0.0	0.0	291.8	41.8	333.6	33.4	**74.7**
9	5	2.4	0.2	357.3	41.8	0.0	0.0	357.3	41.8	399.1	39.9	**89.4**	0.0	0.0	357.3	41.8	399.1	39.9	**89.4**
10.5	8	2.4	0.2	435.7	66.9	0.0	0.0	435.7	66.9	502.6	50.3	**112.6**	0.0	0.0	435.7	66.9	502.6	50.3	**112.6**
12	9	2.4	0.2	435.7	0.0	110.6	357.3	546.3	357.3	903.6	90.4	**202.4**	92.1	250.1	527.8	250.1	777.9	77.8	**174.3**
13.5	21	2.4	0.2	435.7	0.0	262.5	952.7	698.3	952.7	1650.9	165.1	**369.8**	290.9	666.9	726.6	666.9	1393.5	139.4	**312.1**
15	34	2.4	0.2	435.7	0.0	447.5	1515.5	883.2	1515.5	2398.7	239.9	**537.3**	559.2	972.4	994.9	972.4	1967.4	196.7	**440.7**
16.5	24	2.4	0.2	435.7	0.0	647.3	1238.0	1083.0	1238.0	2321.0	232.1	**519.9**	821.6	866.6	1257.4	866.6	2123.9	212.4	**475.8**
18	38	2.4	0.2	435.7	0.0	871.5	2273.2	1307.2	2273.2	3580.4	358.0	**802.0**	1089.9	1379.1	1525.6	1379.1	2904.7	290.5	**650.7**
19.5	23	2.4	0.2	435.7	0.0	1094.5	1223.2	1530.2	1223.2	2753.4	275.3	**616.8**	1355.0	749.2	1790.7	749.2	2539.9	254.0	**568.9**
21	23	2.4	0.2	435.7	0.0	1335.1	1317.3	1770.8	1317.3	3088.0	308.8	**691.7**	1623.3	806.8	2059.0	806.8	2865.9	286.6	**642.0**
22.5	100	2.4	0.2	1121.7	836.6	1335.1	0.0	2456.7	836.6	3293.3	329.3	**737.7**	1623.3	0.0	2745.0	836.6	3581.6	358.2	**802.3**
24	100	2.4	0.2	1807.7	836.6	1335.1	0.0	3142.7	836.6	3979.3	397.9	**891.4**	1623.3	0.0	3431.0	836.6	4267.5	426.8	**955.9**
25.5	100	2.4	0.2	2493.6	836.6	1335.1	0.0	3828.7	836.6	4665.3	466.5	**1045.0**	1623.3	0.0	4117.0	836.6	4953.5	495.4	**1109.6**

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

PRECAST PILE CAPACITY, SIZE = 16''×16'' FOR BH 02

PILE DETAILS INPUTS				ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020)									ALPHA METHOD FOR COHESIVE SOIL AND NORDLUND METHOD FOR NON-COHESIVE SOIL (ADOPTED BY AASHTO-2014)						
				COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-1: Q_{ult}			NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-2: Q_{ult}		
				SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING						SKIN FRICTION	END BEARING					
Pile depth (m)	SPT-N_{field} Per layer	A_s (m^2)	A_{bottom} (m^2)	Q_S (kN)	Q_B (kN)	Q_s (kN)	Q_B (kN)			kN	Ton	kip	Q_s (kN)	Q_B (kN)			kN	Ton	kip
1.5	6	2.4	0.2	72.0	50.2	0.0	0.0	72.0	50.2	122.2	12.2	**27.4**	0.0	0.0	72.0	50.2	122.2	12.2	**27.4**
3	1	2.4	0.2	85.7	8.4	0.0	0.0	85.7	8.4	94.1	9.4	**21.1**	0.0	0.0	85.7	8.4	94.1	9.4	**21.1**
4.5	3	2.4	0.2	126.9	25.1	0.0	0.0	126.9	25.1	152.0	15.2	**34.0**	0.0	0.0	126.9	25.1	152.0	15.2	**34.0**
6	4	2.4	0.2	181.8	33.5	0.0	0.0	181.8	33.5	215.2	21.5	**48.2**	0.0	0.0	181.8	33.5	215.2	21.5	**48.2**
7.5	4	2.4	0.2	236.7	33.5	0.0	0.0	236.7	33.5	270.1	27.0	**60.5**	0.0	0.0	236.7	33.5	270.1	27.0	**60.5**
9	6	2.4	0.2	308.7	50.2	0.0	0.0	308.7	50.2	358.9	35.9	**80.4**	0.0	0.0	308.7	50.2	358.9	35.9	**80.4**
10.5	5	2.4	0.2	374.2	41.8	0.0	0.0	374.2	41.8	416.1	41.6	**93.2**	0.0	0.0	374.2	41.8	416.1	41.6	**93.2**
12	7	2.4	0.2	450.5	58.6	0.0	0.0	450.5	58.6	509.1	50.9	**114.0**	0.0	0.0	450.5	58.6	509.1	50.9	**114.0**
13.5	7	2.4	0.2	526.9	58.6	0.0	0.0	526.9	58.6	585.4	58.5	**131.1**	0.0	0.0	526.9	58.6	585.4	58.5	**131.1**
15	9	2.4	0.2	605.1	75.3	0.0	0.0	605.1	75.3	680.4	68.0	**152.4**	0.0	0.0	605.1	75.3	680.4	68.0	**152.4**

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

GEOSCAPE Consultants Ltd.

PRECAST PILE CAPACITY, SIZE = 16''×16'' FOR BH 03

PILE DETAILS INPUTS				ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020)									ALPHA METHOD FOR COHESIVE SOIL AND NORDLUND METHOD FOR NON-COHESIVE SOIL (ADOPTED BY AASHTO-2014)						
				COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-1: Q_{ult}			NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-2: Q_{ult}		
				SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING						SKIN FRICTION	END BEARING					
Pile depth (m)	SPT-N_{field} Per layer	A_s (m^2)	A_{bottom} (m^2)	Q_S (kN)	Q_B (kN)	Q_s (kN)	Q_B (kN)			kN	Ton	kip	Q_s (kN)	Q_B (kN)			kN	Ton	kip
1.5	2	2.4	0.2	0.0	0.0	4.3	23.2	4.3	23.2	27.5	2.7	**6.2**	2.9	12.6	2.9	12.6	15.5	1.5	**3.5**
3	2	2.4	0.2	0.0	0.0	17.1	46.3	17.1	46.3	63.4	6.3	**14.2**	11.3	21.6	11.3	21.6	32.9	3.3	**7.4**
4.5	3	2.4	0.2	41.2	25.1	17.1	0.0	58.3	25.1	83.4	8.3	**18.7**	11.3	0.0	52.5	25.1	77.6	7.8	**17.4**
6	6	2.4	0.2	113.2	50.2	17.1	0.0	130.3	50.2	180.5	18.0	**40.4**	11.3	0.0	124.5	50.2	174.7	17.5	**39.1**
7.5	8	2.4	0.2	191.6	66.9	17.1	0.0	208.7	66.9	275.6	27.6	**61.7**	11.3	0.0	202.9	66.9	269.8	27.0	**60.4**
9	46	2.4	0.2	507.1	384.8	17.1	0.0	524.2	384.8	909.0	90.9	**203.6**	11.3	0.0	518.4	384.8	903.3	90.3	**202.3**
10.5	70	2.4	0.2	507.1	0.0	140.8	3536.1	648.0	3536.1	4184.1	418.4	**937.2**	279.6	1336.6	786.7	1336.6	2123.4	212.3	**475.6**
12	100	2.4	0.2	507.1	0.0	273.1	4041.2	780.3	4041.2	4821.5	482.2	**1080.0**	547.9	1527.6	1055.0	1527.6	2582.6	258.3	**578.5**
13.5	100	2.4	0.2	1193.1	836.6	273.1	0.0	1466.3	836.6	2302.8	230.3	**515.8**	547.9	0.0	1741.0	836.6	2577.6	257.8	**577.4**
15	100	2.4	0.2	1879.1	836.6	273.1	0.0	2152.2	836.6	2988.8	298.9	**669.5**	547.9	0.0	2427.0	836.6	3263.5	326.4	**731.0**

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

PRECAST PILE CAPACITY, SIZE = 16''×16'' FOR BH 04

PILE DETAILS INPUTS				ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020)									ALPHA METHOD FOR COHESIVE SOIL AND NORDLUND METHOD FOR NON-COHESIVE SOIL (ADOPTED BY AASHTO-2014)						
				COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-1: Q_{ult}			NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	Method-2: Q_{ult}		
				SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING						SKIN FRICTION	END BEARING					
Pile depth (m)	SPT-N_{field} Per layer	A_s (m^2)	A_{bottom} (m^2)	Q_S (kN)	Q_B (kN)	Q_s (kN)	Q_B (kN)			kN	Ton	kip	Q_s (kN)	Q_B (kN)			kN	Ton	kip
1.5	6	2.4	0.2	72.0	50.2	0.0	0.0	72.0	50.2	122.2	12.2	**27.4**	0.0	0.0	72.0	50.2	122.2	12.2	**27.4**
3	8	2.4	0.2	150.4	66.9	0.0	0.0	150.4	66.9	217.3	21.7	**48.7**	0.0	0.0	150.4	66.9	217.3	21.7	**48.7**
4.5	7	2.4	0.2	226.7	58.6	0.0	0.0	226.7	58.6	285.3	28.5	**63.9**	0.0	0.0	226.7	58.6	285.3	28.5	**63.9**
6	10	2.4	0.2	302.7	83.7	0.0	0.0	302.7	83.7	386.3	38.6	**86.5**	0.0	0.0	302.7	83.7	386.3	38.6	**86.5**
7.5	7	2.4	0.2	379.0	58.6	0.0	0.0	379.0	58.6	437.6	43.8	**98.0**	0.0	0.0	379.0	58.6	437.6	43.8	**98.0**
9	8	2.4	0.2	457.4	66.9	0.0	0.0	457.4	66.9	524.3	52.4	**117.4**	0.0	0.0	457.4	66.9	524.3	52.4	**117.4**
10.5	8	2.4	0.2	535.8	66.9	0.0	0.0	535.8	66.9	602.7	60.3	**135.0**	0.0	0.0	535.8	66.9	602.7	60.3	**135.0**
12	4	2.4	0.2	590.7	33.5	0.0	0.0	590.7	33.5	624.1	62.4	**139.8**	0.0	0.0	590.7	33.5	624.1	62.4	**139.8**
13.5	5	2.4	0.2	656.2	41.8	0.0	0.0	656.2	41.8	698.0	69.8	**156.4**	0.0	0.0	656.2	41.8	698.0	69.8	**156.4**
15	6	2.4	0.2	728.2	50.2	0.0	0.0	728.2	50.2	778.4	77.8	**174.4**	0.0	0.0	728.2	50.2	778.4	77.8	**174.4**

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

CAST IN SITU PILE CAPACITY

GEOSCAPE Consultants Ltd.

CAST IN SITU PILE CAPACITY, DIA = 24 INCH FOR BH 01

PILE DETAILS INPUTS				ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020)									ALPHA METHOD FOR COHESIVE SOIL AND REESE & O'-NEILL METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020 & AASHTO-2014)								
				COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-1: $Q_{ULTIMATE}$			COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-2: $Q_{ULTIMATE}$		
				SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING						SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING					
PILE DEPTH (m)	SPT-N_{field} PER LAYER	A_s (m^2)	A_{bottom} (m^2)	Q_S (kN)	Q_B (kN)	Q_s (kN)	Q_B (kN)			KN	TON	KIP	Q_S (kN)	Q_B (kN)	Q_S (kN)	Q_B (kN)			KN	TON	KIP
1.5	8	2.9	0.3	61.9	39.0	0.0	0.0	61.9	39.0	100.9	10.1	22.6	0.0	118.3	0.0	0.0	0.0	118.3	118.3	11.8	26.5
3	2	2.9	0.3	83.5	9.8	0.0	0.0	83.5	9.8	93.3	9.3	20.9	17.8	29.6	0.0	0.0	17.8	29.6	47.3	4.7	10.6
4.5	5	2.9	0.3	135.3	24.4	0.0	0.0	135.3	24.4	159.7	16.0	35.8	62.2	73.9	0.0	0.0	62.2	73.9	136.1	13.6	30.5
6	4	2.9	0.3	178.6	19.5	0.0	0.0	178.6	19.5	198.1	19.8	44.4	97.8	59.1	0.0	0.0	97.8	59.1	156.9	15.7	35.1
7.5	5	2.9	0.3	230.3	24.4	0.0	0.0	230.3	24.4	254.7	25.5	57.1	142.2	73.9	0.0	0.0	142.2	73.9	216.1	21.6	48.4
9	5	2.9	0.3	282.0	24.4	0.0	0.0	282.0	24.4	306.4	30.6	68.6	186.7	73.9	0.0	0.0	186.7	73.9	260.6	26.1	58.4
10.5	8	2.9	0.3	343.9	39.0	0.0	0.0	343.9	39.0	383.0	38.3	85.8	257.8	118.3	0.0	0.0	257.8	118.3	376.1	37.6	84.2
12	9	2.9	0.3	343.9	0.0	87.3	208.4	431.2	208.4	639.6	64.0	143.3	257.8	0.0	143.8	141.9	401.6	141.9	543.5	54.3	121.7
13.5	21	2.9	0.3	343.9	0.0	207.2	555.6	551.2	555.6	1106.7	110.7	247.9	257.8	0.0	398.5	331.1	656.3	331.1	987.5	98.7	221.2
15	34	2.9	0.3	343.9	0.0	353.2	883.8	697.2	883.8	1580.9	158.1	354.1	257.8	0.0	742.8	536.1	1000.6	536.1	1536.7	153.7	344.2
16.5	24	2.9	0.3	343.9	0.0	510.9	721.9	854.9	721.9	1576.8	157.7	353.2	257.8	0.0	1044.9	378.4	1302.7	378.4	1681.2	168.1	376.6
18	38	2.9	0.3	343.9	0.0	687.9	1325.6	1031.8	1325.6	2357.5	235.7	528.1	257.8	0.0	1429.1	599.2	1686.9	599.2	2286.1	228.6	512.1
19.5	23	2.9	0.3	343.9	0.0	863.9	713.3	1207.8	713.3	1921.1	192.1	430.3	257.8	0.0	1736.0	362.7	1993.8	362.7	2356.4	235.6	527.8
21	23	2.9	0.3	343.9	0.0	1053.8	768.2	1397.7	768.2	2165.9	216.6	485.2	257.8	0.0	2051.3	362.7	2309.1	362.7	2671.8	267.2	598.5
22.5	100	2.9	0.3	885.4	487.8	1053.8	0.0	1939.2	487.8	2427.0	242.7	543.7	493.8	1168.1	2051.3	0.0	2545.0	1168.1	3713.1	371.3	831.7
24	100	2.9	0.3	1426.8	487.8	1053.8	0.0	2480.6	487.8	2968.5	296.8	664.9	729.7	1168.1	2051.3	0.0	2781.0	1168.1	3949.1	394.9	884.6
25.5	100	2.9	0.3	1968.3	487.8	1053.8	0.0	3022.1	487.8	3509.9	351.0	786.2	965.7	1168.1	2051.3	0.0	3017.0	1168.1	4185.0	418.5	937.4

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

CAST IN SITU PILE CAPACITY, DIA = 24 INCH FOR BH 02

PILE DETAILS INPUTS				ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020)									ALPHA METHOD FOR COHESIVE SOIL AND REESE & O'-NEILL METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020 & AASHTO-2014)								
				COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-1: $Q_{ULTIMATE}$			COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-2: $Q_{ULTIMATE}$		
				SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING						SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING					
PILE DEPTH (m)	SPT-N_{field} PER LAYER	A_s (m^2)	A_{bottom} (m^2)	Q_S (kN)	Q_B (kN)	Q_s (kN)	Q_B (kN)			KN	TON	KIP	Q_S (kN)	Q_B (kN)	Q_S (kN)	Q_B (kN)			KN	TON	KIP
1.5	6	2.9	0.3	56.9	29.3	0.0	0.0	56.9	29.3	86.1	8.6	19.3	0.0	88.7	0.0	0.0	0.0	88.7	88.7	8.9	19.9
3	1	2.9	0.3	67.7	4.9	0.0	0.0	67.7	4.9	72.6	7.3	16.3	8.9	14.8	0.0	0.0	8.9	14.8	23.7	2.4	5.3
4.5	3	2.9	0.3	100.2	14.6	0.0	0.0	100.2	14.6	114.8	11.5	25.7	35.6	44.3	0.0	0.0	35.6	44.3	79.9	8.0	17.9
6	4	2.9	0.3	143.5	19.5	0.0	0.0	143.5	19.5	163.0	16.3	36.5	71.1	59.1	0.0	0.0	71.1	59.1	130.2	13.0	29.2
7.5	4	2.9	0.3	186.8	19.5	0.0	0.0	186.8	19.5	206.3	20.6	46.2	106.7	59.1	0.0	0.0	106.7	59.1	165.8	16.6	37.1
9	6	2.9	0.3	243.7	29.3	0.0	0.0	243.7	29.3	272.9	27.3	61.1	160.0	88.7	0.0	0.0	160.0	88.7	248.7	24.9	55.7
10.5	5	2.9	0.3	295.4	24.4	0.0	0.0	295.4	24.4	319.8	32.0	71.6	204.5	73.9	0.0	0.0	204.5	73.9	278.4	27.8	62.4
12	7	2.9	0.3	355.6	34.1	0.0	0.0	355.6	34.1	389.8	39.0	87.3	266.7	103.5	0.0	0.0	266.7	103.5	370.2	37.0	82.9
13.5	7	2.9	0.3	415.9	34.1	0.0	0.0	415.9	34.1	450.0	45.0	100.8	328.9	103.5	0.0	0.0	328.9	103.5	432.4	43.2	96.9
15	9	2.9	0.3	477.6	43.9	0.0	0.0	477.6	43.9	521.6	52.2	116.8	408.9	133.0	0.0	0.0	408.9	133.0	542.0	54.2	121.4

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

GEOSCAPE Consultants Ltd.

CAST IN SITU PILE CAPACITY, DIA = 24 INCH FOR BH 03

PILE DETAILS INPUTS				ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020)									ALPHA METHOD FOR COHESIVE SOIL AND REESE & O'-NEILL METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020 & AASHTO-2014)								
				COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-1: $Q_{ULTIMATE}$			COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-2: $Q_{ULTIMATE}$		
				SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING						SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING					
PILE DEPTH (m)	SPT-N_{field} PER LAYER	A_s (m^2)	A_{bottom} (m^2)	Q_S (kN)	Q_B (kN)	Q_s (kN)	Q_B (kN)			KN	TON	KIP	Q_S (kN)	Q_B (kN)	Q_S (kN)	Q_B (kN)			KN	TON	KIP
1.5	2	2.9	0.3	0.0	0.0	3.4	13.5	3.4	13.5	16.9	1.7	**3.8**	0.0	0.0	0.0	31.5	0.0	31.5	31.5	3.2	**7.1**
3	2	2.9	0.3	0.0	0.0	13.5	27.0	13.5	27.0	40.5	4.1	**9.1**	0.0	0.0	26.5	31.5	26.5	31.5	58.0	5.8	**13.0**
4.5	3	2.9	0.3	32.5	14.6	13.5	0.0	46.0	14.6	60.6	6.1	**13.6**	26.7	44.3	26.5	0.0	53.1	44.3	97.5	9.7	**21.8**
6	6	2.9	0.3	89.3	29.3	13.5	0.0	102.8	29.3	132.1	13.2	**29.6**	80.0	88.7	26.5	0.0	106.5	88.7	195.2	19.5	**43.7**
7.5	8	2.9	0.3	151.2	39.0	13.5	0.0	164.7	39.0	203.7	20.4	**45.6**	151.1	118.3	26.5	0.0	177.6	118.3	295.8	29.6	**66.3**
9	46	2.9	0.3	400.3	224.4	13.5	0.0	413.8	224.4	638.2	63.8	**143.0**	482.1	680.0	26.5	0.0	508.6	680.0	1188.6	118.9	**266.2**
10.5	70	2.9	0.3	400.3	0.0	111.2	1031.1	511.5	1031.1	1542.5	154.3	**345.5**	482.1	0.0	293.5	1103.8	775.6	1103.8	1879.4	187.9	**421.0**
12	100	2.9	0.3	400.3	0.0	215.6	1178.3	615.9	1178.3	1794.2	179.4	**401.9**	482.1	0.0	588.7	1314.1	1070.8	1314.1	2384.8	238.5	**534.2**
13.5	100	2.9	0.3	941.8	487.8	215.6	0.0	1157.4	487.8	1645.2	164.5	**368.5**	718.1	1168.1	588.7	0.0	1306.7	1168.1	2474.8	247.5	**554.4**
15	100	2.9	0.3	1483.2	487.8	215.6	0.0	1698.8	487.8	2186.7	218.7	**489.8**	954.0	1168.1	588.7	0.0	1542.7	1168.1	2710.8	271.1	**607.2**

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

GEOSCAPE Consultants Ltd.

CAST IN SITU PILE CAPACITY, DIA = 24 INCH FOR BH 04

PILE DETAILS INPUTS				ALPHA METHOD FOR COHESIVE SOIL AND TOMLINSON METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020)									ALPHA METHOD FOR COHESIVE SOIL AND REESE & O'-NEILL METHOD FOR NON-COHESIVE SOIL (ADOPTED BY BNBC-2020 & AASHTO-2014)								
				COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-1: $Q_{ULTIMATE}$			COHESIVE SOIL		NON-COHESIVE SOIL		TOTAL SKIN FRICTION (KN)	END BEARING (KN)	METHOD-2: $Q_{ULTIMATE}$		
				SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING						SKIN FRICTION	END BEARING	SKIN FRICTION	END BEARING					
PILE DEPTH (m)	SPT-N_{field} PER LAYER	A_s (m^2)	A_{bottom} (m^2)	Q_S (kN)	Q_B (kN)	Q_s (kN)	Q_B (kN)			KN	TON	KIP	Q_S (kN)	Q_B (kN)	Q_S (kN)	Q_B (kN)			KN	TON	KIP
1.5	6	2.9	0.3	56.9	29.3	0.0	0.0	56.9	29.3	86.1	8.6	19.3	0.0	88.7	0.0	0.0	0.0	88.7	88.7	8.9	19.9
3	8	2.9	0.3	118.7	39.0	0.0	0.0	118.7	39.0	157.8	15.8	35.3	71.1	118.3	0.0	0.0	71.1	118.3	189.4	18.9	42.4
4.5	7	2.9	0.3	179.0	34.1	0.0	0.0	179.0	34.1	213.1	21.3	47.7	133.3	103.5	0.0	0.0	133.3	103.5	236.8	23.7	53.0
6	10	2.9	0.3	238.9	48.8	0.0	0.0	238.9	48.8	287.7	28.8	64.4	222.2	147.8	0.0	0.0	222.2	147.8	370.1	37.0	82.9
7.5	7	2.9	0.3	299.2	34.1	0.0	0.0	299.2	34.1	333.3	33.3	74.7	284.5	103.5	0.0	0.0	284.5	103.5	388.0	38.8	86.9
9	8	2.9	0.3	361.0	39.0	0.0	0.0	361.0	39.0	400.1	40.0	89.6	355.6	118.3	0.0	0.0	355.6	118.3	473.9	47.4	106.1
10.5	8	2.9	0.3	422.9	39.0	0.0	0.0	422.9	39.0	461.9	46.2	103.5	426.7	118.3	0.0	0.0	426.7	118.3	545.0	54.5	122.1
12	4	2.9	0.3	466.2	19.5	0.0	0.0	466.2	19.5	485.7	48.6	108.8	462.3	59.1	0.0	0.0	462.3	59.1	521.4	52.1	116.8
13.5	5	2.9	0.3	518.0	24.4	0.0	0.0	518.0	24.4	542.4	54.2	121.5	506.7	73.9	0.0	0.0	506.7	73.9	580.6	58.1	130.1
15	6	2.9	0.3	574.8	29.3	0.0	0.0	574.8	29.3	604.1	60.4	135.3	560.0	88.7	0.0	0.0	560.0	88.7	648.7	64.9	145.3

The Consultant is requested to give further attention while using this Pile capacity. The capacity mentioned above is a generalized capacity and it is highly recommended to re-evaluate the pile capacity. The consultant should choose the Pile geotechnical capacity based on his engineering judgment, knowledge and experience. It is suggested to evaluate the Structural capacity of pile and consider other factors that affect the Pile capacity (e.g: Negative skin friction, Liquefaction, Scouring, etc.) before using the allowable Geotechnical capacity of pile.

APPENDIX-04

SOIL LIQUEFACTION ANALYSIS

LIQUEFACTION POTENTIAL EVALUATION BASED ON IDRISS & BOULANGER (2014) (FOR BH 01)

Depth (m)	SUSCEPTIBILITY TO LIQUEFACTION	FC (%)	WATER LEVEL (m)	N_{field}	$(N_1)_{60}$	$\Delta(N_1)_{60}$	$(N_1)_{60CS}$	σ_v (Kpa)	σ_v' (Kpa)	M_w	MSF	K_σ	r_d AS PER IB 2014	PGA (BED-ROCK)	F(pga)	a_{max}/g	CRR AS PER IB 2014	CSR AS PER IB 2014	FACTOR OF SAFETY AS PER IB 2014	REMARKS AS PER IB 2014
1.5	Non-susceptible																			**Non-Liquefiable**
3	Non-susceptible																			**Non-Liquefiable**
4.5	Non-susceptible																			**Non-Liquefiable**
6	Non-susceptible																			**Non-Liquefiable**
7.5	Non-susceptible																			**Non-Liquefiable**
9	Non-susceptible																			**Non-Liquefiable**
10.5	Non-susceptible																			**Non-Liquefiable**
12	Susceptible	30	1.55	9	7.7014	5.36662	13.07	197.25	94.245	7.5	1	1.01	0.78	0.28	1.35	0.378	0.14056	0.40069	0.3507895	**Liquefiable**
13.5	Susceptible	20	1.55	21	18.045	4.49281	22.54	226.5	108.78	7.5	1	0.99	0.78	0.28	1.35	0.378	0.24155	0.40567	0.595442	**Liquefiable**
15	Susceptible	20	1.55	34	31.028	4.49281	35.52	256.5	124.065	7.5	1	0.95	0.78	0.28	1.35	0.378	1.23834	0.42148	2.9380953	**Non-Liquefiable**
16.5	Susceptible	20	1.55	24	18.755	4.49281	23.25	286.13	138.975	7.5	1	0.95	0.78	0.28	1.35	0.378	0.25377	0.41649	0.6093112	**Liquefiable**
18	Susceptible	20	1.55	38	32.42	4.49281	36.91	316.13	154.26	7.5	1	0.88	0.78	0.28	1.35	0.378	1.71225	0.45009	3.8042154	**Non-Liquefiable**
19.5	Susceptible	20	1.55	23	15.998	4.49281	20.49	345.75	169.17	7.5	1	0.93	0.78	0.28	1.35	0.378	0.21197	0.42276	0.5013824	**Liquefiable**
21	Susceptible	20	1.55	23	15.264	4.49281	19.76	375.38	184.08	7.5	1	0.92	0.78	0.28	1.35	0.378	0.20293	0.42585	0.4765333	**Liquefiable**
22.5	Non-susceptible																			**Non-Liquefiable**

APPENDIX-05

FIELD PHOTOGRAPHS

Geoscape Consultants Ltd.

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