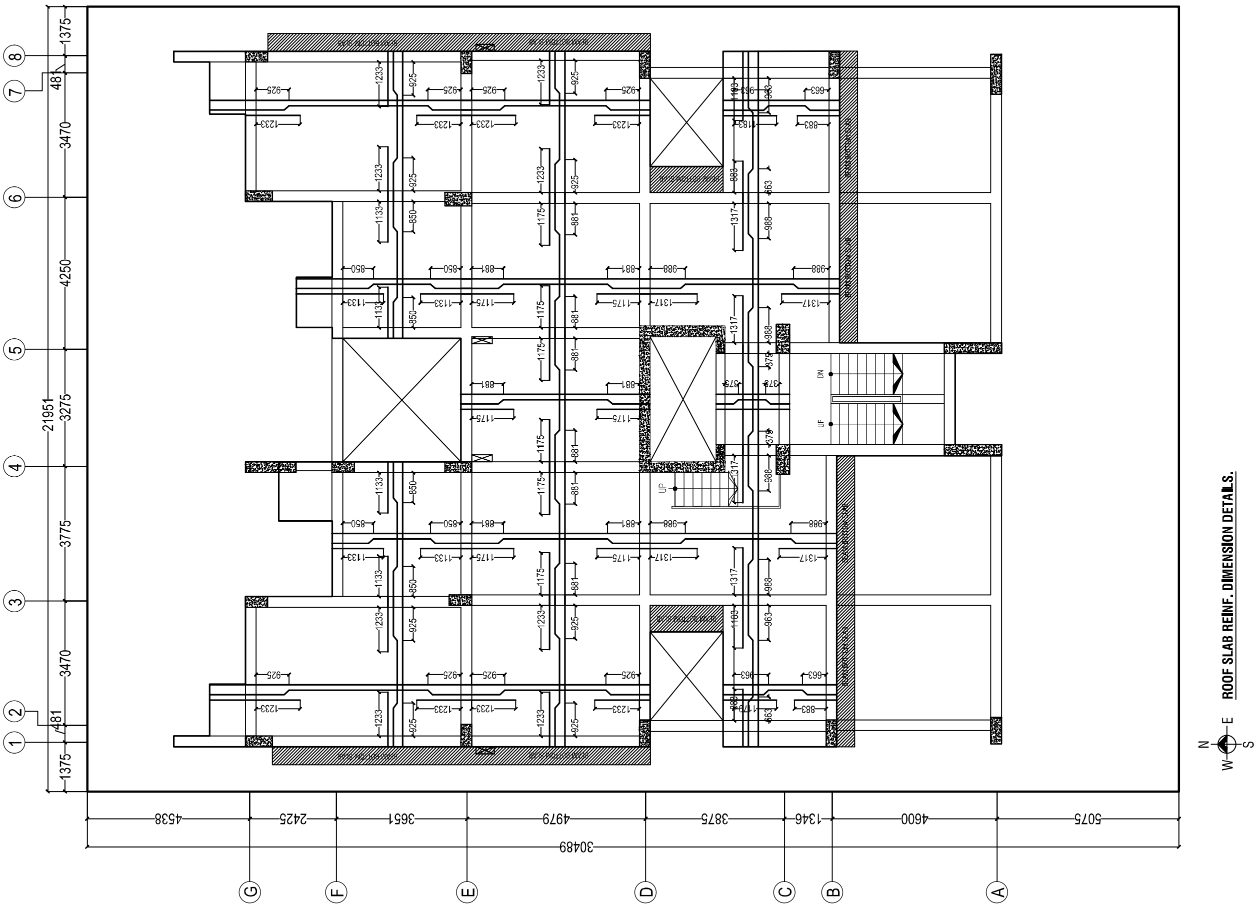
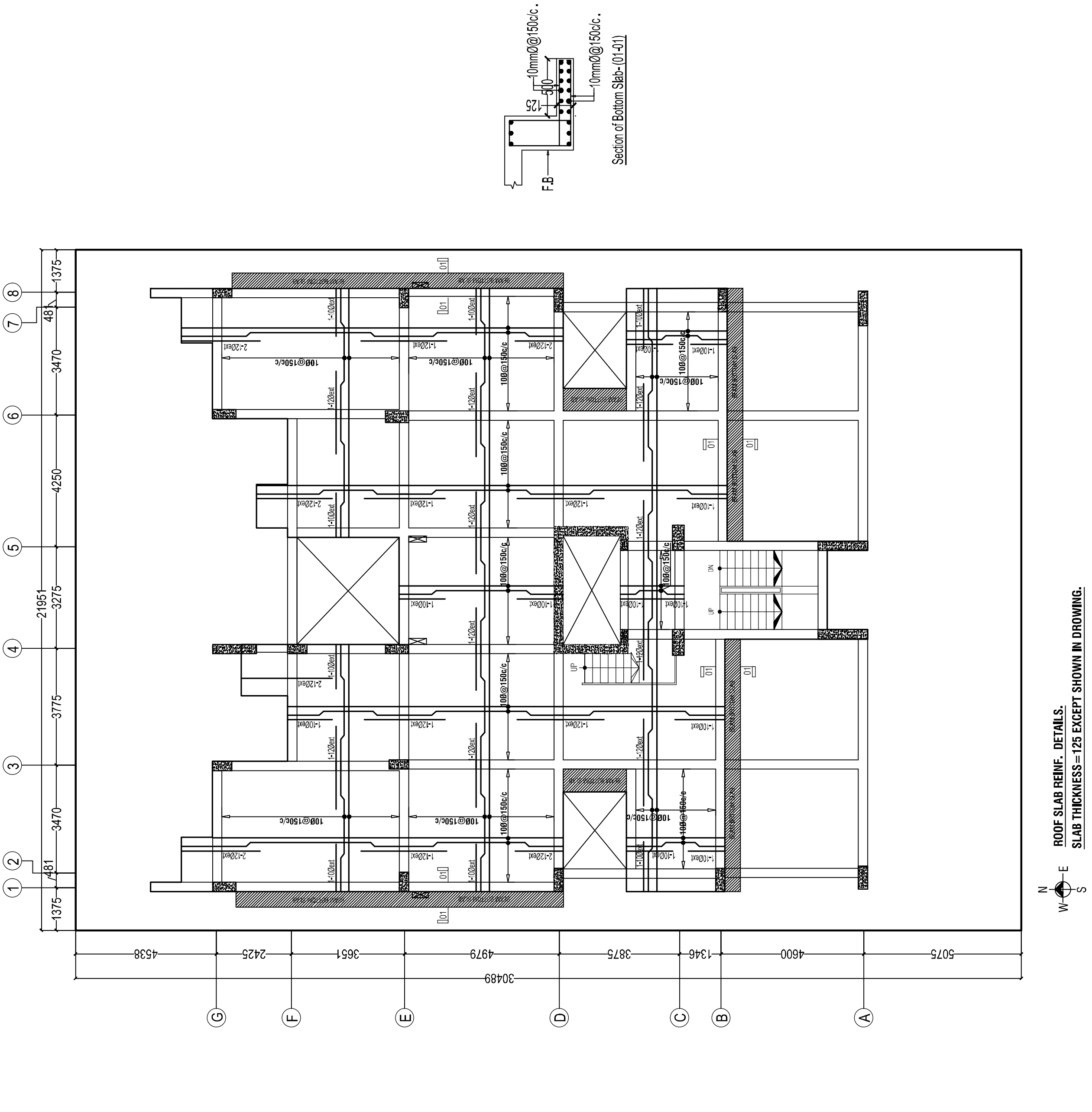


CLIENT.

PROJECT. PROPOSED B+G+9 STORIED RESIDENTIAL BUILDING

ALL DIMENSION ARE IN MILIMETER

DWG TITLE ROOF SLAB REINF. DIMENSION DETAILS.



ROOF SLAB REINF. DETAILS.
SLAB THICKNESS=125 EXCEPT SHOWN IN DROWING.

PROJECT.	CLIENT.	
PROPOSED B+G+9 STORIED RESIDENTIAL BUILDING		ALL DIMENSION ARE IN MILIMETER
	DWG TITLE	ROOF SLAB REINF. DETAILS.

PROJECT.

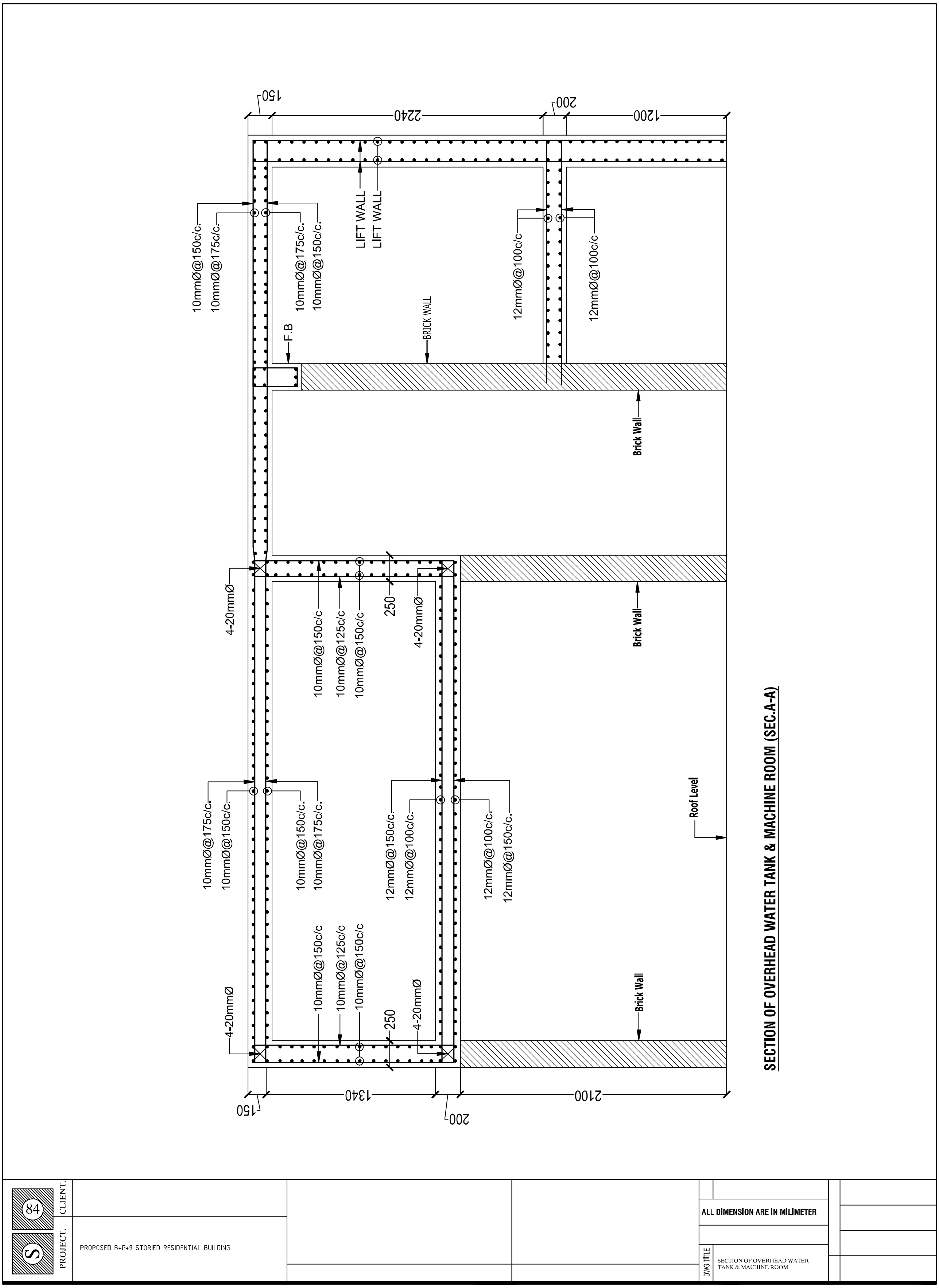
CLIENT.

PROPOSED B+G+9 STORIED RESIDENTIAL BUILDING

ALL DIMENSION ARE IN MILIMETER

DWG TITLE

8TH TO TOP FLOOR BEAM
LAY-OUT PLAN


SECTION OF OVERHEAD WATER TANK & MACHINE ROOM (SEC.A-A)
150
2240
200
1200
1340
2100
250
LIFT WALL
LIFT WALL
BRICK WALL
Brick Wall
Roof Level
F.B
10mmØ@150c/c.
10mmØ@175c/c.
10mmØ@175c/c.
10mmØ@150c/c.
12mmØ@100c/c
12mmØ@100c/c
4-20mmØ
10mmØ@150c/c
10mmØ@125c/c
10mmØ@150c/c
12mmØ@150c/c.
12mmØ@100c/c.
12mmØ@100c/c.
12mmØ@150c/c.
CLIENT.
PROJECT.
PROPOSED B+G+9 STORIED RESIDENTIAL BUILDING
ALL DIMENSION ARE IN MILIMETER
DWG TITLE
SECTION OF OVERHEAD WATER TANK & MACHINE ROOM

SUBSOIL INVESTIGATION REPORT

PROPOSED G+9 STORIED RESIDENTIAL BUILDING

BASHUNDHARA, DHAKA

JANUARY, 2023

Site Class Based on Soil Properties

Input:

Calculations:

Depth (m)	Field N Value	Shear Wave Velocity, V_{si} (m/s)	Average Shear Wave Velocity, V_s (m/s)	Average N Value
1.5	1	61.00	108.38	2.67
3	2	86.27		
4.5	3	105.66		
6	4	122.00		
7.5	1	61.00		
9	1	61.00		
10.5	2	86.27		
12	2	86.27		
13.5	3	105.66		
15	2	86.27		
16.5	3	105.66		
18	4	122.00		
19.5	8	172.53		
21	5	136.40		
22.5	4	122.00		
24	6	149.42		
25.5	14	228.24		
27	12	211.31		
28.5	12	211.31		
30	49	427.00		

PILE LOAD CAPACITY CALCULATION FROM SPT VALUE

Pile Length = 29.7 m; From Level = -2.4 m

Ultimate Capacity of Pile, $Q_u = 40NAp + (\frac{N}{2})As$ (ton)

End bearing of pile = 40NAp

Bore hole -02 at 32 m, SPT = 50+

Corrected SPT = 15+0.5(N-15)

= 15+0.5(50-15)

= 15+17.5

= 32.5

End bearing of pile = $40*32.5*(3.1416*.6^2/4)$ = 367.5 tons

Skin friction $= \frac{N}{2}\pi DL$

$N = \frac{5+5+9+6+5+5+7+8+28+32.5+32.5}{11}$

= 13

Skin friction $= \frac{13}{2}*\pi*0.6*16.5$

= 202 tons

Total = 367.5+202 = 569.5 tons

For bored pile capacity = 569.5 *0.65 = 370 tons

Considering, F.S = 3

$Q_{allowable} = \frac{370}{3}$

= 123 tons

Considered capacity = 100 tons

Contents

Appendices

Appendix No	Title
1	Bore Log
2	Laboratory Test Results
2A	Wash Sieve and Moisture Content
2B	Sieve analysis
2C	Atterberg Limit Curves
2D	Summary
3	Calculation of Bearing Capacity
3A	Axial Capacity of Single Bored Pile

CHAPTER 01

GENERAL INFORMATION

1.1 Introduction

This report manifests the geotechnical investigation of this project and provides reliable, specific and detailed information about the subsoil condition of the concerned site as well as engineering properties of soil at different layers. The investigation included 6 boreholes of highest 34.5 m depths.

The soil investigation has revealed that the boreholes in project location contain a mixture silt, clay and sand depending on location and depth. After collecting disturbed soil samples from the boreholes, geotechnical team of SDE has done several laboratory tests to determine different soil parameters.

1.2 About the Investigation:

This report gives a synopsis of the results of subsoil investigation, in-situ testing and the laboratory tests conducted by SDE in the plot in ***Plot #2731, Road #20/A, Block- M, Basundhara R/A, Dhaka.***

1.3 Purpose of the Investigation:

The purpose of this investigation is to determine the existing soil profile and engineering characteristics of the subsurface conditions at the site and to provide the designer with comments on the following:

- Soil properties such as soil composition, cohesion, angle of internal friction, water content etc.
- Geotechnical design parameters (e.g. ground bearing pressure) which will be required for a safe and economic design and excavation of the engineering works.

1.4 Scope of the Investigation:

The scope of work is given hereunder:

- To explore the subsurface condition of proposed development area and to provide general data relating to the project.
- To carry out Standard Penetration Test (SPT) to determine the natural bearing resistance of the subsoil.
- To obtain disturbed and undisturbed soil samples for carrying out the laboratory tests to determine the natural and relevant physical & engineering properties of the subsoil pertaining to the site.
- Preparing a Geotechnical Report based on engineering analysis to present the data obtained through investigation and recommendations for the foundation design and considerations.

1.5 Project Information:

SDE performed reconnaissance survey to identify the given location of boreholes by the Client for the proper knowledge of the design parameters required for the structure, keeping the design in line with the local construction practices and the availability of materials.

Moreover, the Client provided a layout plan of boring locations, which was assisted, to the Consultant during the soil boring work.

Table 1: Project information

Project	**Standard Penetration Test (SPT) of Fortress Holdings Limited at Basundhara R/A**
Location	**Block # M; Plot: 2731, Road #20/A, Basundhara R/A**
Developer	**Fortress Holdings Limited**
Topography	Low land and open area
GPS Coordinate	23°49'18.0"N 90°27'26.2"E

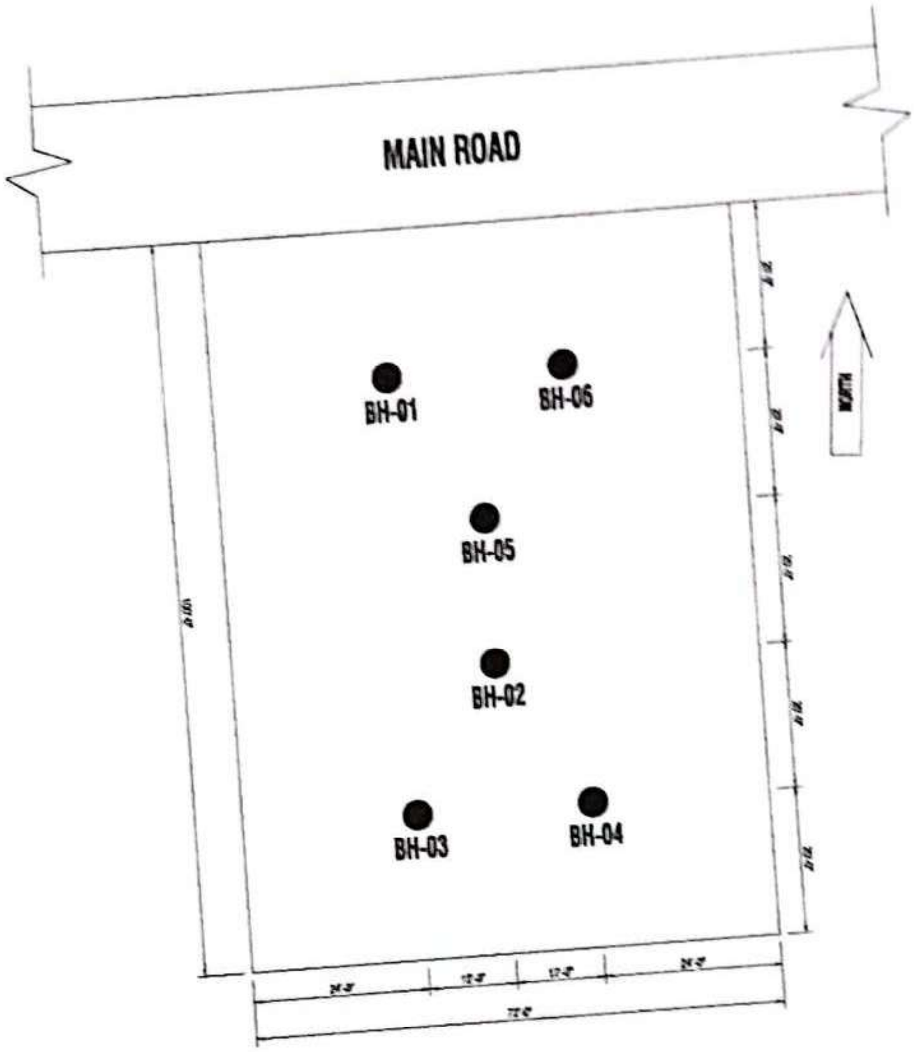


Figure 1: Borehole location layout plan given by the Client

1.6 Reporting:

The report consists of the following:

- Sub-soil condition of project area
- Detailed description of every soil layer presented in Borelogs
- Foundation bearing capacity
- Final Borelog including Summary of Test Results
- Laboratory Test Results.

CHAPTER 02

SITE EXPLORATION

2.1 Description of Site:

The proposed recidential building plot is located at Block # M; Plot: 2731, Road #20/A, Basundhara R/A, Dhaka. The project site is located at Seismic Zone 02. The sub soil deposits are found to be consistent in formation, that comprises of the cohesive layer of fine soils mostly consisting of inorganic SILT and CLAY (ML/CH/ CL) and layers of cohesionless soil consisting of various types of sand (SM) layers are encountered in different layers.

Figure 2: Site Location

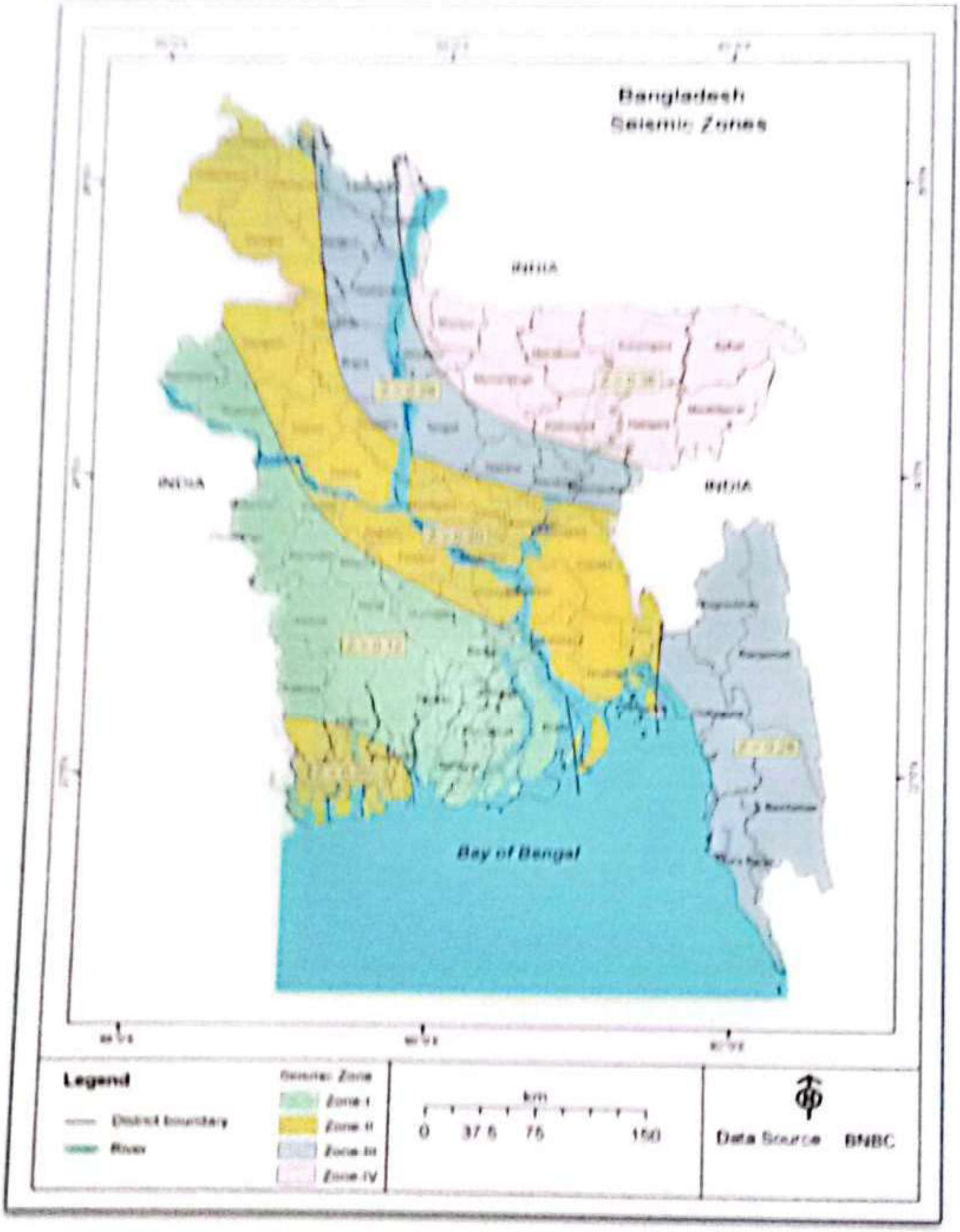


Figure 3: Seismic Zoning of Bangladesh

2.2 Details of Boreholes:

Borehole No.	Depth of Borehole (m)	Ground Water Level (m)	Date of Drilling
01	34.5	-1.0	04/09/2021
02	33.0	-1.0	05/09/2021
03	34.5	-1.0	06/09/2021
04	33.0	-1.5	07/09/2021
05	33.0	-1.5	08/09/2021
06	34.5	-1.5	09/09/2021

CHAPTER 03

SUB-SOIL INVESTIGATION

3.1 Field Investigation:

3.1.1 Drilling of Boreholes:

Boreholes were drilled by percussion-wash drilling methods using a bladed bit to produce a nominal diameter for 100mm. Flushing of the hole was achieved by the addition of bentonite to form a mud of sufficient density to lift soil cutting satisfactorily. Careful attention was given to drilling rates with alternate lifting & dropping of a bit advanced to ensure that soil particles cut by the bit was able to rise in bentonite mud column and thus ensure a 'clean' base to the bore hole. Casing of 125 mm diameter was used in the uppermost few meters of each boring.

3.1.2 Standard Penetration Test (SPT):

Standard Penetration Test (SPT) according to ASTM D 1586 has been executed at 1.5 m intervals to determine relative density/consistency and classification of soil at different elevation inclusive collection of disturbed soil samples from each interval in accordance to the requirements of the specification of the Engineer. An exploratory boring with a diameter 100mm was bored to the depth of the first test. A SPT sampler, connected with required length of BW size rod to a 63.5 kg hammer, is inserted in to the boring. SPT sampler is split-spoon sampler with a ball valve to permit exit of air or water from the top during driving and to assist in retaining sample during withdrawal; in addition, the sampler has a tapered shoe for allowing penetration in to the ground. The number of blows required to progress the sampler 450 mm was recorded in three 150 mm intervals. The SPT N-value has been calculated by summing the hammer blows required to advance the sampler during the last two intervals of the test. The blow count for the first 150 mm was recorded; however, this number is ignored during the N-value since the soil immediately below the drilling rod is generally considered to be disturbed.

3.1.3 Condition of Refusal:

According to ASTM, the boring has been continued until one of the following (refusal) occurs:

- A total of 50 blows during any one of the three 6.0 inches (150mm) increments described earlier.
- A total of 100 blows have been applied.
- There is no observed advance of the sampler during the application of 10 successive blows of the hammer. For automated systems an advance of less than 0.1 inch (2mm) per blow can be considered refusal.

3.1.4 Sample Collection:

Disturbed and undisturbed sampling both were carried out in the field according to proper ASTM standard. Disturbed or split-barrel sampling was done according to ASTM D1586 – 18 (Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils) and sample was collected in zipper bag and stored in air tight plastic box. For undisturbed sampling, Shelby tube was used according to ASTM D1587/D1587M – 15 (Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes). After collection, all samples were sent to the Laboratory with proper precaution.

3.1.5 SPT Corrections:

The standard practice now to express the N-value to an average energy ratio of 60%. In addition, the borehole diameter, rod length (i.e., borehole depth), and whether a liner is used within the sampler can contribute to the energy loss and hence influence the N-value. Therefore, the N-value corrected to account for these factors can be written as,

$N_{60} = (N \times \eta_H \times \eta_B \times \eta_S \times \eta_R)/60$

Where,

N_{60} = standard penetration number, corrected for field condition
N = measured penetration number from field
η_H = hammer efficiency (%)
η_B = correction for borehole diameter
η_S = sampler correction
η_R = correction for rod length

Variations of η_H, η_B, η_S, and η_R, based on recommendations by Seed et al. (1985) and Skempton (1986) are summarized in the following figure:

1. Variation of η_H

Country	Hammer type	Hammer release	η_H (%)
Japan	Donut	Free fall	78
	Donut	Rope and pulley	67
United States	Safety	Rope and pulley	60
	Donut	Rope and pulley	45
Argentina	Donut	Rope and pulley	45
China	Donut	Free fall	60
	Donut	Rope and pulley	50

2. Variation of η_B

Diameter mm	η_B
60–120	1
150	1.05
200	1.15

3. Variation of η_S

Variable	η_S
Standard sampler	1.0
With liner for dense sand and clay	0.8
With liner for loose sand	0.9

4. Variation of η_R

Rod length m	η_R
>10	1.0
6–10	0.95
4–6	0.85
0–4	0.75

3.1.6 SPT CO-RELATIONS:

The SPT n-value is widely used in correlations for different soil parameters such as relative density of granular soil, consistency of cohesive soil, angle of internal friction of granular soil, undrained compressive strength of cohesive soil etc.

Terzaghi and Peck (1967) suggests the general idea of relative density of sand or granular soil with respect to SPT n-value. He also suggests the basic idea to get the stiffness of cohesive soil i.e. Clay based on the field SPT n-value.

Table 2: Correlation between SPT N-value and relative density of non-cohesive soil

SPT N-value	Relative Density for Sand
0 – 4	Very loose
4 – 10	Loose
10 – 30	Medium dense
30 – 50	Dense
Over 50	Very dense

Table 3: Correlation between SPT N-value and consistency of cohesive soil

SPT N-value	Consistency for Clay
0 – 2	Very Soft
2 – 4	Soft
4 – 8	Medium stiff
8 – 15	Stiff
15 – 30	Very stiff
Over 30	Hard

Moreover, Peck, Hanson, and Thornburn (1974) give a correlation between angle of internal friction and corrected SPT N-value, which can be approximated as (Wolff, 1989)

Angle of internal friction, $\emptyset\ (degree) = 27.1 + 0.3N_{60} - 0.00054(N_{60})^2$

Based on the study of Kulhawy and Mayne (1990), a correction can be expressed between the undrained cohesion of clay and corrected SPT N-value.

Undrained cohesion, c= $0.06P_aN_{60}$, where, $P_a = atmospheric\ pressure = 101\ kPa$

3.2 Laboratory Test:

All the collected samples were tested in the Geotechnical Laboratory of SDE with properly calibrated equipment to reveal their engineering properties which meet the requirement of this project. All the calibration certificates of pertinent laboratory equipment were issued by BRTC (Bureau of Research, Testing and Consultation), BUET. List of Laboratory Test and relevant standards are given in the following table:

Name of the Test	Methodology	Standard
Moisture Content	Moisture or Water Content (of a material)—the ratio expressed as a percent of the mass of "pore" or "free" water in a given mass of material to the mass of the solid material. A standard temperature of 110° ± 5°C is used to determine these masses. In this test, a test specimen is dried in an oven at a temperature of 110° ± 5°C to a constant mass. The loss of mass due to drying is considered to be water. The water content is calculated using the mass of water and the mass of the dry specimen. This test method covers the laboratory determination of the water (moisture) content by mass of soil, rock, and similar materials where the reduction in mass by drying is due to loss of water. Water content has several significances and use such as: (a) the water content of a material is used in expressing the phase relationships of air, water, and solids in a given volume of material, (b) In fine-grained (cohesive) soils, the consistency of a given soil type depends on its water content. The water content of a soil, along with its liquid	ASTM D 2216 – 98

Name of the Test	Methodology	Standard
	and plastic limits, is used to express its relative consistency or liquidity index.	
Wet Sieve Analysis	These test methods cover determination of the amount of material finer than a 75-µm (No. 200) sieve by washing. In this test a specimen of the soil is washed over a 75-µm (No. 200) sieve. Clay and other particles that are dispersed by the wash water, as well as water-soluble materials, are removed from the soil during the test. The loss in mass resulting from the wash treatment is calculated as mass percent of the original sample and is reported as the percentage of material finer than a 75-µm (No. 200) sieve by washing. Material finer than the 75-µm (No. 200) sieve can be separated from larger particles much more efficiently and completely by wet sieving than with dry sieving. Therefore, when accurate determinations of material finer than 75-µm sieve in soil are desired, this test method is used on the test specimen prior to dry sieving. Usually the additional amount of material finer than 75-µm sieve obtained in the dry sieving process is a small amount. If it is large, the efficiency of the washing operation should be checked, as it could be an indication of degradation of the soil.	ASTM D 1140 – 00
Sieve Analysis	Sieve analysis is a determination of the proportions of particles lying within certain size ranges in a granular material by separation on sieves of different size openings. Soils consist of particles with various shapes and sizes. This test method is used to separate particles into size ranges and to determine quantitatively the mass of particles in each range. These data are combined to determine the particle-size distribution (gradation). This test method uses a square opening sieve criterion in determining the gradation of soil between the 3-in. (75-mm) and No. 200 (75-µm) sieves. The gradation of a soil is an indicator of engineering properties. The gradation of a soil is an indicator of engineering properties. Sieve Analysis has several significances and use such as; (a) the gradation of the soil is used for classification, (b) The gradation (particle-size distribution) curve is used to calculate the coefficient of uniformity and the coefficient of curvature. (c) Selection and acceptance of fill materials are often based on gradation and many more which can be found in the respective ASTM standard.	ASTM D 422
Liquid Limit & Plastic Limit	Liquid limit (LL)—the water content, in percent, of a soil at the arbitrarily defined boundary between the semiliquid and plastic states. The specimen is processed to remove any material retained on a 425-µm (No. 40) sieve. The liquid limit is determined by performing trials in which a portion of the specimen is spread in a brass cup, divided in two by a grooving tool, and then allowed to flow together from the	ASTM D 4318 – 05

Name of the Test	Methodology	Standard
	shocks caused by repeatedly dropping the cup in a standard mechanical device. The multipoint liquid limit, Method A, requires three or more trials over a range of water contents to be performed and the data from the trials plotted or calculated to make a relationship from which the liquid limit is determined. The one-point liquid limit, Method B, uses the data from two trials at one water content multiplied by a correction factor to determine the liquid limit. Plastic limit (PL)—the water content, in percent, of a soil at the boundary between the plastic and semi-solid states. The plastic limit is determined by alternately pressing together and rolling into a 3.2-mm (1⁄8-in.) diameter thread a small portion of plastic soil until its water content is reduced to a point at which the thread crumbles and can no longer be pressed together and re-rolled. The water content of the soil at this point is reported as the plastic limit. These test methods are used as an integral part of several engineering classification systems to characterize the fine-grained fractions of soils and to specify the fine-grained fraction of construction materials. The liquid limit, plastic limit, and plasticity index of soils are also used extensively, either individually or together, with other soil properties to correlate with engineering behavior such as compressibility, hydraulic conductivity (permeability), compactibility, shrink-swell, and shear strength. The liquid and plastic limits of a soil and its water content can be used to express its relative consistency or liquidity index. In addition, the plasticity index and the percentage finer than 2-µm particle size can be used to determine its activity number. More details use can be found in respective ASTM standard.	
Specific Gravity	Specific gravity of soil solids, Gs, n - the ratio of the mass of a unit volume of a soil solids to the mass of the same volume of gas-free distilled water at 20°C. These test methods cover the determination of the specific gravity of soil solids that pass the 4.75-mm (No. 4) sieve, by means of a water pycnometer. The specific gravity of soil solids is used in calculating the phase relationships of soils, such as void ratio and degree of saturation. The specific gravity of soil solids is used to calculate the density of the soil solids. This is done by multiplying its specific gravity by the density of water (at proper temperature).	ASTM D 854 – 02
Unconfined Compression Test	Unconfined compressive strength (qu)—the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test; in this test method, unconfined compressive strength is taken as the maximum	ASTM D2166/D2166M – 13

Name of the Test	Methodology	Standard
	load attained per unit area or the load per unit area at 15 % axial strain, whichever is secured first during the performance of a test. In this test method, a cylindrical soil specimen is unconfined laterally while loaded axially at an axial strain rate between 0.5 to 2 %/min. Measurements are made of elapsed time, axial deformation, and axial load. The unconfined compressive stress, q_u, is calculated as the compressive stress at failure. The shear strength, s_u, is one half of the unconfined compressive strength. This test method covers the determination of the unconfined compressive strength of cohesive soil in the intact, remolded, or reconstituted condition, using strain-controlled application of the axial load.	
USCS Classification	This practice describes a system for classifying mineral and organo-mineral soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index and shall be used when precise classification is required. This classification system identifies three major soil divisions: coarse-grained soils, fine-grained soils, and highly organic soils. These three divisions are further subdivided into a total of 15 basic soil groups. This standard classifies soils from any geographic location into categories representing the results of prescribed laboratory tests to determine the particle-size characteristics, the liquid limit, and the plasticity index.	ASTM D 2487 – 06

3.3 Quantity of Laboratory Test:

Name of Tests	No. of Tests
Moisture Content and Wash Sieve	26
Grain Size Distribution Test	08
Atterberg Limit	15
Specific Gravity Test	02
Unconfined Compression Test	02

CHAPTER 04

DISCUSSIONS AND RECOMENDATIONS

4.1 General:

Due to some factors like restricted no of Boreholes and location, availability of soil samples and others issues there are some limitations which are presented below:

1. Amount of soil samples collected from sampler, was not efficient to perform all the necessary tests at every 1.5 m interval.
2. Where tests were not possible to conduct, soil parameters were calculated from well-established theories.
3. For the variation of temperature, the result of Specific Gravity test may vary a little.

4.2 Calculation of Bearing Capacity:

4.2.1 Shallow Foundation:

A shallow foundation is generally placed within the soil at a depth less than the width of the foundation. In this report, bearing capacity of shallow foundation has been calculated using Meyerhof, Vesic and Hansen's equation.

In order to determine the ultimate strength shear failure, it is used from the proposed general method in view of the many factors affecting the bearing capacity.

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

The calculation method of each used parameter has been mentioned in table 1.
It is worth mentioning that if the load on the foundation has eccentricity toward the center of foundation level, the amount of allowable bearing capacity is reduced, and if the eccentricity of foundation is more than one sixth the width of foundation, part of the foundation has been under tension, therefore shear failure foundation should be re-examined.

Where,

N_γ, N_c and N_q	= Bearing Capacity Factors
s_γ, s_c and s_q	= Shape Factors
d_γ, d_c and d_q	= Depth Factors
i_γ, i_c and i_q	= Inclination Factors
g_γ, g_c and g_q	= Ground Factors (Based on slope)
b_γ, b_c and b_q	= Base Factors (Tilted Base)
c	= Cohesion of soil
B	= Width of footing
γ	= Effective unit weight of soil

SDE

		Hansen	Vesic	Meyerhof
N	N_c	$(N_q - 1)\cot\varphi$	$(N_q - 1)\cot\varphi$	$(N_q - 1)\cot\varphi$
	N_q	$e^{\pi\tan\varphi}\cdot\tan^2\left(45^\circ + \frac{\varphi}{2}\right)$	$e^{\pi\tan\varphi}\cdot\tan^2\left(45^\circ + \frac{\varphi'}{2}\right)$	$e^{\pi\tan\varphi}\cdot\tan^2\left(45^\circ + \frac{\varphi'}{2}\right)$
	N_γ	$1.5(N_q - 1)\cdot\tan(\varphi)$	$2(N_q + 1)\cdot\tan(\varphi)$	$(N_q - 1)\cdot\tan(1.4\varphi)$
s	s_c	$1 + \left(\frac{N_q B'}{N_c L'}\right)$	$1 + \left(\frac{N_q B'}{N_c L'}\right)$	$1 + 0.2\tan^2\left(45^\circ + \frac{\varphi}{2}\right)\frac{B'}{L'}$
	s_q	$1 + (\sin\varphi)\frac{B'}{L'}$	$1 + (\tan\varphi)\frac{B'}{L'}$	$1 + 0.1\tan^2\left(45^\circ + \frac{\varphi}{2}\right)\frac{B'}{L'}$
	s_γ	$1 - \left(0.4\frac{B'}{L'}\right) \geq 0.6$	$1 - \left(0.4\frac{B'}{L'}\right) \geq 0.6$	$1 + 0.1\tan^2\left(45^\circ + \frac{\varphi}{2}\right)\frac{B'}{L'}$
d	d_c	$1 + 0.4k$	$1 + 0.4k$	$1 + 0.2\tan\left(45^\circ + \frac{\varphi}{2}\right)\frac{D}{B'}$
	d_q	$1 + 2\tan(\varphi)(1 - \sin\varphi)^2 k$	$1 + 2\tan(\varphi)(1 - \sin\varphi)^2 k$	$1 + 0.1\tan\left(45^\circ + \frac{\varphi}{2}\right)\frac{D}{B'}$
	d_γ	1	1	$1 + 0.1\tan\left(45^\circ + \frac{\varphi}{2}\right)\frac{D}{B'}$
i	i_c	$i_q - \left(\frac{1 - i_q}{N_q - 1}\right)$	$i_q - \left(\frac{1 - i_q}{N_q - 1}\right)$	$\left(1 - \frac{\theta}{90^\circ}\right)^2$
	i_q	$\left[1 - \frac{0.5H_i}{V + A_f C_a \cot\varphi}\right]^{[illegible]}$	$\left[1 - \frac{H_i}{V + A_f C_a \cot\varphi}\right]^{\left(\frac{2+\frac{B}{L}}{1+\frac{B}{L}}\right)}$	$\left(1 - \frac{\theta}{90^\circ}\right)^2$
	i_γ	$\left[1 - \frac{\left(0.7 - \frac{\eta}{450}\right)H_i}{V + A_f C_a \cot\varphi}\right]^{[illegible]}$	$\left[1 - \frac{H_i}{V + A_f C_a \cot\varphi}\right]^{\left(\frac{2+\frac{B}{L}}{1+\frac{B}{L}}+1\right)}$	$\left(1 - \frac{\theta^\circ}{\varphi^\circ}\right)^2$
g	g_c	$1 - \frac{\beta}{147}$	$i_q - \left(\frac{1 - i_q}{5.14\tan\varphi}\right)$	–
	g_q	$(1 - 0.5\tan(\beta))^5$	$(1 - \tan(\beta))^2$	–
	g_γ	$(1 - 0.5\tan(\beta))^5$	$(1 - \tan(\beta))^2$	–
b	b_c	$1 - \frac{\eta}{147}$	$1 - \left(\frac{2B'}{5.14\tan\varphi}\right)$	–
	b_q	$e^{-2\eta\tan\varphi}$	$(1 - \eta\tan(\varphi))^2$	–
	b_γ	$e^{-2.7\eta\tan\varphi}$	$(1 - \eta\tan(\varphi))^2$	–

Vesic and Hansen modified the equation of Meyerhof by providing Ground and Base Factors.

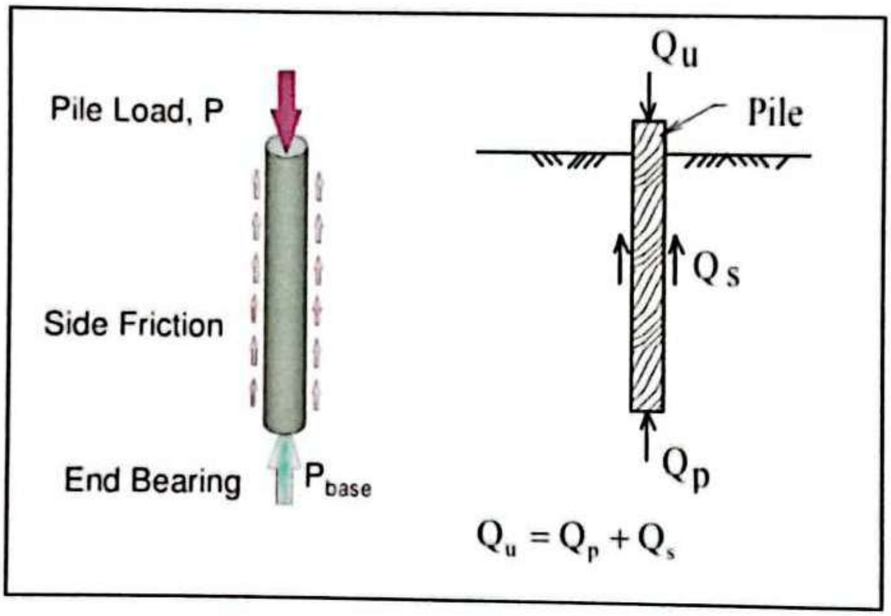
4.2.2 Deep Foundation:

The bearing capacity of Bored and Driven pile are showed at Appendix 3

The ultimate capacity (axial) of pile, $Q_{ult} = Q_s + Q_b$

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

Where,
Q_{ult} = Ultimate Geotechnical (axial) Capacity
Q_s = Skin Friction
Q_b = End Bearing
A_s = Skin friction area of pile
f_s = Unit frictional resistance
A_b = Area of pile tip
f_b = Unit tip resistance



Cohesive Soil:
The α- method is adopted for estimating short term load capacity of pile. According to this method,

$$f_s = \alpha c$$
$$Q_s = A_s f_s$$

where,
α = adhesion factor
c = cohesion of soil
The expression of End bearing, $Q_b = c_b N_c A_b$
Where,
C_b = undrained shear strength of soil at the base of pile
N_c = Bearing capacity factor

N_c is usually taken as "9". However, $N_c = 6\left[1 + 0.2\left(\frac{L}{D_b}\right)\right] \leq 9$

$$Q_{ult} = A_s \alpha c + 9\, c_b A_b$$

Cohesionless Soil:

The lateral earth pressure co-efficient, (K) is introduced in this method. According to this method,

$$f_s = K\,\sigma_z \tan\delta$$

Where,

K = Lateral earth pressure co-efficient

σ_z = effective stress at the perimeter of the pile

δ = friction angle between pile and soil

So, Skin friction of cohesion less soil is,

$$Q_s = K\,\sigma_z \tan\delta\, A_s$$

The expression of End bearing,

$$Q_b = N_q\,\sigma_t\,A_b$$

Where,

N_q = Bearing capacity factor

Σ_t = effective stress at the tip of the pile

$$Q_{ult} = K\,\sigma_z \tan\delta\, A_s + N_q\,\sigma_t\,A_b$$

4.3 Seismic Information of the Site:

Seismic Information of the Site based on BNBC-2020 are given below:

Borehole No	Site Classification	Seismic Intensity	Seismic Zone	Seismic Zone Coefficient
01	SD	Moderate	2	0.2
02	SD	Moderate	2	0.2
03	SD	Moderate	2	0.2
04	SD	Moderate	2	0.2

4.4 Conclusion:

The geotechnical parameters, information, suggestions, recommendations, computation of bearing capacity and soil resistance for piles presented herein were developed utilizing the engineering properties of the soil information obtained from Standard Penetration Test and laboratory tests conducted. This report only depicts the soil conditions at the specific boring location at the specific time the boring was made. As such, the soil conditions adjacent to the borehole locations could vary from those encountered in the borehole.

4.4 Recommendation:

The calculation of bearing capacity for shallow and deep foundation is attached in Appendix 3A and Appendix 3B respectively. It is recommended to choose the type and size of the foundation according to the requirement of the project. The ultimate bearing capacity is calculated in the report, it is suggested to use necessary factor of safety according to the type of project and re-evaluate it before final use considering the effect of settlement and liquefaction.

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