GEOTECNICAL STATISTICAL EVALAUATION OF SITE PROPOSED FOR COURTYARD BY MARRIOT, G-11 ISLAMABAD

By

Ali Shahriyar

Muhammad Ali

Department of Earth and Environmental Sciences Bahria University, Islamabad, Pakistan

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A thesis submitted to Bahria University, Islamabad in partial fulfilment of the requirements for the degree of B.S Geology

By

Ali Shahriyar (01-165191-017)
Muhammad Ali (01-165191-011)

Department of Earth and Environmental Sciences Bahria University, Islamabad, Pakistan

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ABSTRACT

The primary goal of this study is to determine the soil's bearing capacity in Evaluation of Courtyard by Marriot in G11 Islamabad. 2 boreholes up to 100ft depth have been performed. The boreholes were drilled using a percussion machine, and samples were then collected. Standard Penetration Test (SPT) and field density testing were carried out in accordance with ASTM guidelines. Tests like sieve analysis, the Unconfined Compression Strength Test (USC), the Atterberg Limits, and moisture content tests were performed to interpret the soil geotechnical behavior. Using Meyerhof's equation for deep formation, bearing capacity was calculated. On the basis of field and lab test results baring capacity was calculated and raft formations are recommended for it. The Atterberg Limits (ASTM D-4318) and grain size analysis (ASTM D-4318) were performed to interpret the sub soil geotechnical behaviors. On the basis of the field and lab test results bearing capacity and pile foundation are recommended for the site.

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ABBREVIATIONS

ASTM	The American Society of Testing Materials
SPT	Standard Penetration Tool
S	SPT Number
R	Refusal. N>50
CPT	Cone Penetration Test
C	CPT Number
UDS	Undisturbed Sample
FDT	Field Density Test
MDD	Maximum Dry Density
UCS	Unconfined Compressive Strength
DS	Disturbed Sample
GWT	Ground Water Table
BH	Bore Hole
Qa	Allowable Bearing Capacity
GM	Silty Gravel with sand and clay
SM	Silty sand with clay and gravel
NMC	Natural Moisture Content
GED	Geotechnical Engineering Department

CHAPTER 1

INTRODUCTION

1.1 Introduction

Engineering Geology is an area where assessments are focused on how well soil mechanics operate, which includes identifying the subsurface conditions, chemical, physical, and mechanical qualities that have an impact on the project, and evaluating the risks associated with technical constraints. Following the required evaluation, the seismic design is completed, and the site, foundation, and construction are then supervised.

Typically, a structure's foundation is its lowest point. The building's foundation transfers the superstructure's weight to the ground or other surface underneath it. Every building has a number of separate foundations, in general. In order to transfer the stress from each main column directly to the ground, foundations for structures typically sit immediately beneath each main column (Meyerhof, 1951).

A superstructure is an engineering component of a system that transfers weight into a foundation or substructure. A foundation can only hold machinery and sustain heavy industrial equipment like pipes, towers, and tanks. This phrase is crucial for structures and bridges. Since the load-bearing components of an engineering system are carried to the ground by a foundation, it is preferable to refer to it as such. This definition makes it obvious that the foundation is the most crucial component of an engineering system.

A solid foundation has the capacity to evenly distribute weight while minimizing stress on the soil. An excessive amount of soil stress can lead to depression in the area and harm engineering buildings. The companies researching the area must ascertain the soil's bearing capacity for this reason.

1.2 Types of foundation

The construction and consequently the soil encountered determine the type of foundation to be used. There are two categories for foundation types: shallow foundations and deep foundations. The deep foundation is constructed at a depth of roughly 20–60 meters, the shallow foundation is laid at a depth of about 9 meters.

1.2.1 Shallow foundations

The construction is built in close proximity to the earth or rock surface in shallow foundations. When compared to deep foundations, the depth is shallow and can vary by up to 9 meters. Additional forms of shallow foundation include:

i) Raft foundation

ii) Spread or isolated footing

iii) Strip footing

1.2.2 Raft Foundation

The term "Raft foundation" also refers to raft foundation. In order to support the columns and walls and distribute the weight throughout the earth, it is made of thick cemented slices of block on a sizable area of soil that has been fortified by steel

It is used in scenarios like:

I. Soil having low bearing capacity

II. When the load of engineering structure has to be divided throughout a large area.

III. When the stress on soil has to be decreased.

IV. The basement needs to be built.

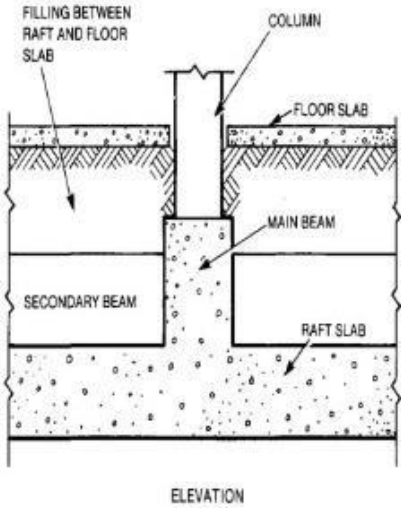


Fig.1.1. Raft foundation. (Bowels, 1996)

1.2.3 Types of raft foundation

The soil conditions and the amount of stress placed on the foundation determine the type of raft that is employed. The numerous kinds employed during building include the following:

I. Flat plate mat

II. Plate thickened under the column

III. Two-way beam and slab raft

IV. Plate raft with pedestals

V. Plied raft

Rigid frame mat

i) Flat plate mat

The simplest raft foundation is this kind. They are employed when the structure's weight is not excessive and the walls or columns are erected with equal distances between them. Economically, a thickness of up to 300 mm is desirable because anything beyond that is unprofitable.

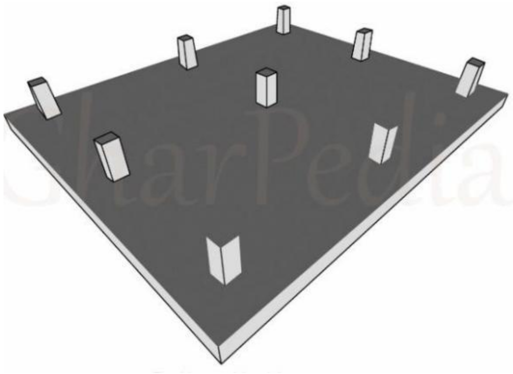


Fig.1.2. Flat plate mat. (Bowels, 1996)

ii) Plate thickened under columns

When there is a heavy load on the walls and columns, the slab's thickness is increased and more strength is provided beneath them to withstand the additional stresses.

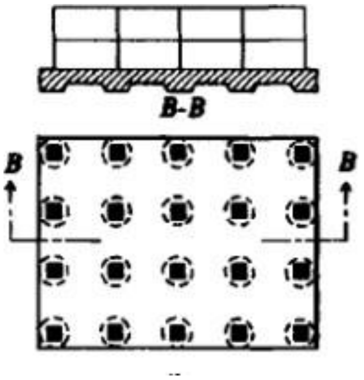


Fig.1.3. Plate thickened under columns. (Bowels, 1996)

ii) Two-way beam and slab

In this kind of raft, the walls and columns are joined together for support, and the beams are formed of a single, substantial material. This type of raft is required when building walls far apart and the strain on the columns is not constant.

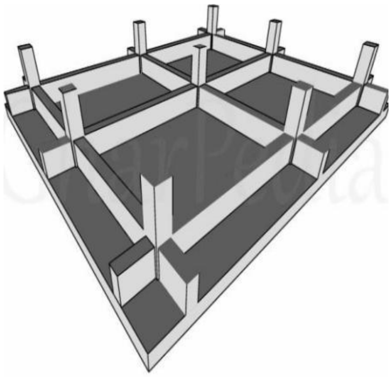


Fig.1.4. Two-way beam and slab. (Bowels, 1996)

iii) Plates with pedestals

This kind is constructed beneath the columns, serving the same function as a flat plate that has been thickened there.

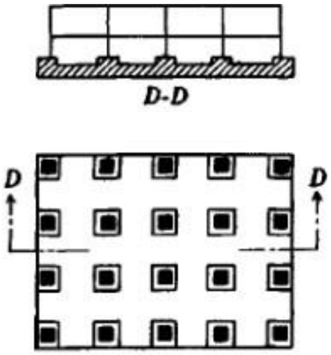


Fig.1.5. Plates with pedestals. (Bowels, 1996)

iv) Piled raft

This type of raft is supported by the piles. It is primarily required when the water table is high and the soil is easily crushed. These piles aid in reducing sinking and act as a buoyant barrier.

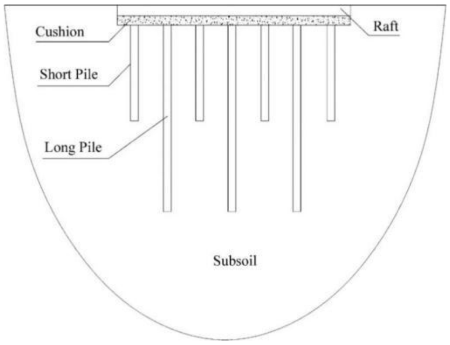


Fig.1.6. Piled raft. (Bowels, 1996)

v) Rigid frame mat

When the connecting beams are deeper than a specific point and the columns are carrying a very heavy weight, this kind of raft is necessary. When a thick slab is needed, this type is helpful.

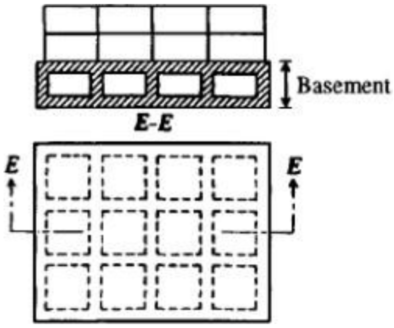


Fig.1.7. Rigid frame mat. (Bowels, 1996)

1.2.4 Isolated Footings

They are employed for shallow foundations in order to support and distribute the weight of pillar-like constructions. This type of footing can either be fortified or not. A non-strengthened footing requires a larger height to enable the necessary load division.

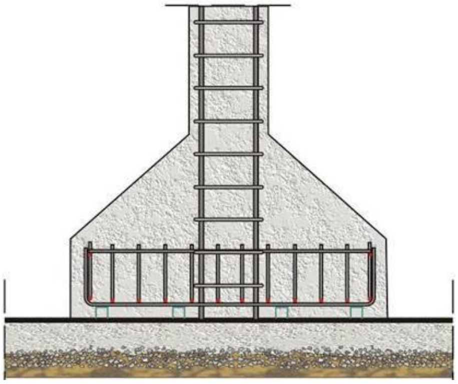


Fig.1.8. Isolated footing. (Bowels, 1996)

1.2.5 Strip footing

Most frequently, they serve as the load-bearing walls' base. Its width is often twice as wide as the wall, but it can also be wider. The width and material utilised for strengthening depend on how much weight the foundation soil can support.

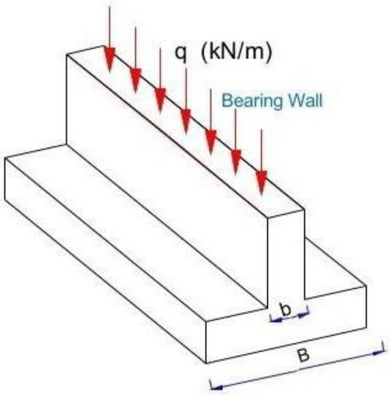


Fig.1.9. Strip footing. (Bowels, 1996)

When the surface soil can sustain the structure with firm, sturdy support, the shallow form of foundation is used. They are however frail, and if the earth is not compacted well or contains alluvial deposits, they could be squeezed by the construction.

1.3 Deep foundations

When a foundation is built with deep foundations, it is buried well beneath the surface of the earth, making it more stable and vulnerable to earthquakes and other natural disasters. The range of depth is up to 60 meters. Other varieties of deep foundations include:

I. Pile foundation
II. Drilled shafts

1.3.1 Pile foundation

It is composed of a sturdy cylindrical piece of wood or concrete. In comparison to shallow foundation, they can be utilized to lay down deep foundation and are more costly.

They are used in the scenarios like:

I. Piles are constructed to distribute the load of the structure towards the bedrock or a firmer soil if the top soil layer is more prone to compress or is too weak to support the construction.
II. The same can be done to prevent bending and simultaneously sustain the load on the structure in the event that horizontal forces are acting in that area.

1. If the structures are below the water table, it can still aid to prevent forces from acting upward.

Piles are used in construction varies which depends on the kind of load that has to be transferred. Their varieties include:

I. Wooden piles
II. Concrete piles
III. Composite piles
IV. Steel piles

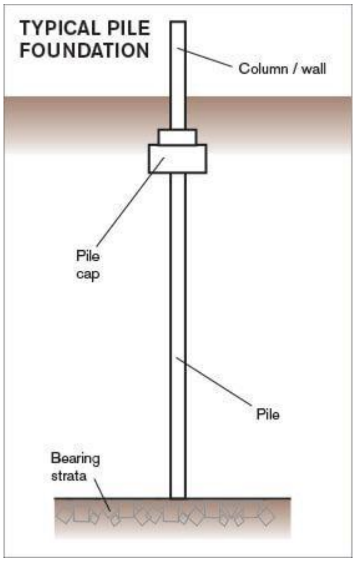


Fig.1.10. Typical pile foundation. (Bowels, 1996)

1.3.2 Drilled shafts

They are the piles with a diameter of roughly 30 inches that are buried deeply in the ground. It has many benefits, some of which are as follows:

I. Only one drilled shaft can be enough rather than using group of piles.
II. No noise pollution produced from hammering unlike the pile driving.
III. They can resist high forces coming from lateral loads.

Drilled shafts, like other items, have drawbacks such as the need for regular supervision and the operation being delayed by poor weather.

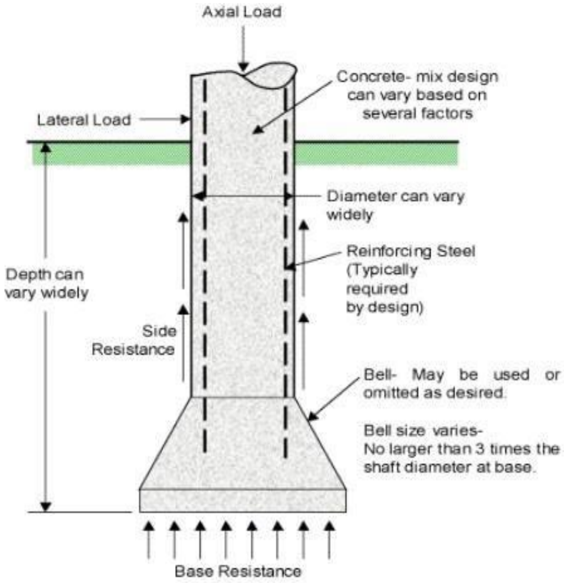


Fig.1.11. Drilled shaft. (Bowels, 1996)

1.4 Seismicity of area

According to the Pakistani seismic zones created by numerous organizations, Islamabad and Rawalpindi may experience a moderate earthquake. However, given the seismic history of the twin cities, particularly the earthquake of October 8, 2005, and the destruction of the Margalla Towers, we must treat this issue seriously in order to prevent a repeat of this type of tragedy. Aside from the events of 2005, the region where Islamabad and Rawalpindi are located has a lengthy history and has been the target of attacks of various intensities and sizes which is shown in the table 1.1.

Table 1.1. Historical database from prehistoric times until 1903 including earthquakes that caused major destruction in 20th century, (PMD, 2009).

History of earthquakes in Islamabad and Rawalpindi			
Dates	**Epicenter**	**Intensity**	**Description**
25 A. D	33.7N 72.9E	X	Taxila Earthquake It occurred at the main center of Buddhist civilization.
4/6/1669	33.4N 73.2E	VI-XI	Mandra Earthquake Max intensity was around VI
24/1/1852	34N 73.5E	VIII	Murree Hills Earthquake Murree hills was the epicente which killed 350 People.
20/12/1869	33.6N 73.1E	VII-VIII	Rawalpindi Earthquake Max intensity was around VII

Islamabad, the nation's capital, is encircled by five significant faults: the MBT, the Kalabagh fault, the Salt Range Push (SRT), the Jhelum fault, and the Himalayan frontal thrust. There is no way to completely predict when an earthquake would happen along these fault lines. The fault lines beneath Islamabad are 30 million years old, according to the officials. Even though earthquakes could happen at any time, it's crucial that we are ready for them. Seismic zoning, according to GSP authorities, was completed when Islamabad was designated as Pakistan's capital. Geologists warned the authorities that high intensity earthquakes could happen in Islamabad because it is located in an active zone and advised against building tall structures there (PMD, 2009).

1.5 Objectives

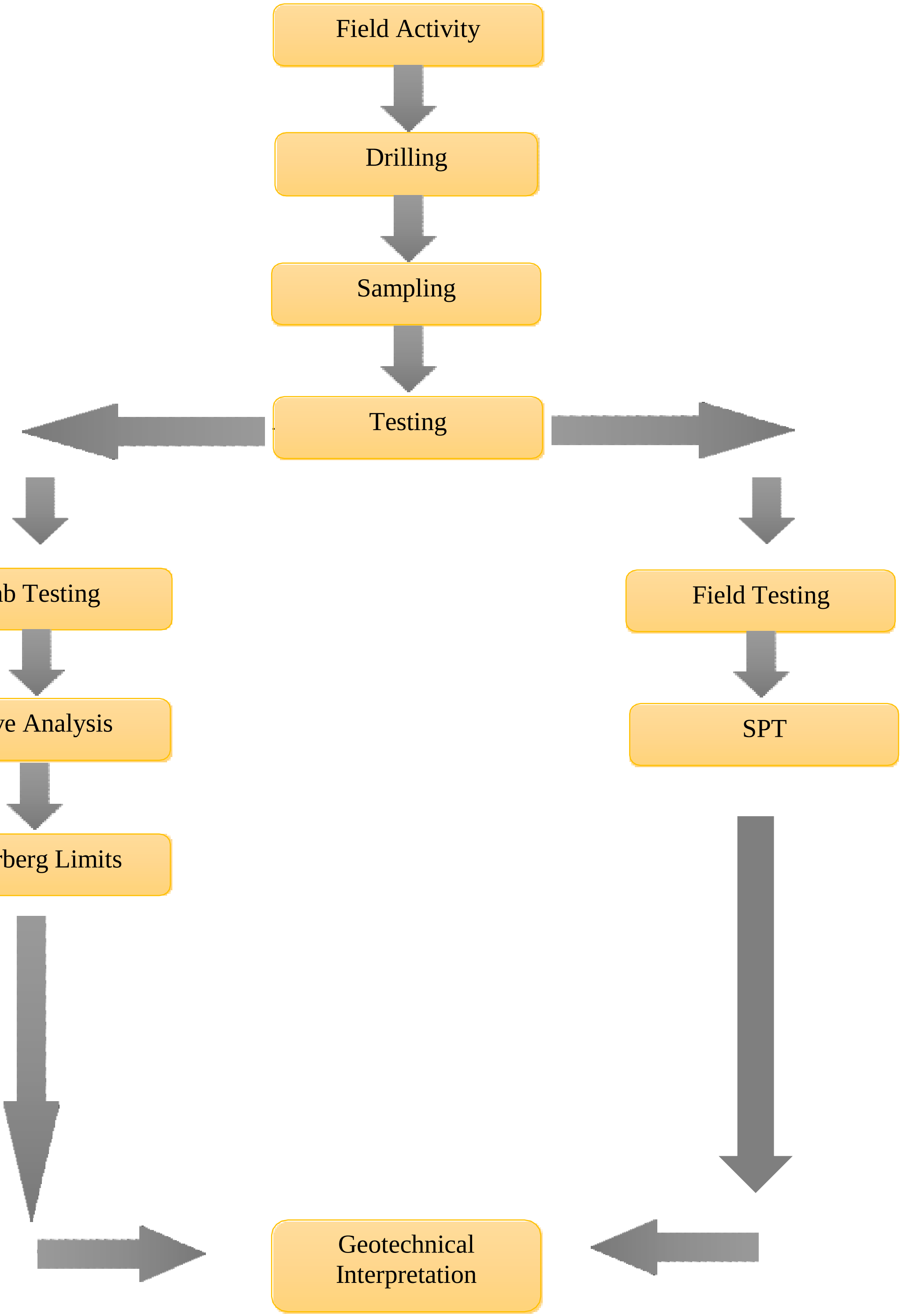
This field work's objectives are as follows:

i) To evaluate the geological conditions of the construction site.

ii) To calculate the bearing capacity of the foundation.

1.6 Methodology

The methodology of research work involves borehole drilling and excavation of disturbed and undisturbed samples. Field testing was done by Standard Penetration Test (SPT). However, where the SPT could not be done, Cone Penetration Test (CPT) was performed for sampling. On the other hand, variety of tests were performed in the laboratory which includes, Atterberg limits, Sieve analysis and Unconfined Compressive Strength test (UCS). Eventually, by observing and discussing the results of these tests, the bearing capacity of the foundation was known. The flow chart of the methodology is shown in figure.1.2.


Field Activity
Drilling
Sampling
Testing
Lab Testing
Field Testing
Sieve Analysis
SPT
Atterberg Limits
Geotechnical Interpretation

CHAPTER 2
LITERATURE REVIEW

2.1 Literature Review

Although the exact date when people began manipulating soil for development goals is unknown, work done in the archaeological field during ancient times has shown that it began long ago, as evidenced by the Indus culture, which flourished in modern-day Pakistan. There is no proof of how the weathering process affects the foundation. But it is true that work in the geological sector began in antiquity and that engineering geology advanced in the 19th century (Kerisel, 1985).

Engineering geology has been practiced for a very long time, but it has not yet reached its full potential. It began to develop from Egypt, India, and China, according to history. Ancient dam-like structures were built in the Indus basin circa 2000 B.C. to supply water to the inhabitants of Mohenjo-Daro. No proof of the work done to balance this structure's foundation exists today (Shah, S.M., 2009).

Popular figure Leonardo da Vinci contributed to geology and architecture in the late 15th century. He studied soil behavior and developed methods for assessing soil bearing capacity, measuring sand's angle of repose, and working on processes connected to ground water hydrology, but sadly, his work was only used in books and not in actual applications at the time (Shah, S.M., 2009).

It is well known that one of Italy's towers, Pisa, is tipped, and this is because there was insufficient soil analysis. The most recent examinations indicate that it is slanted as a result of the loose and compressible earth beneath the tower. These kinds of incidents are what ignite the necessity for soil testing before the construction of any project.

The states of forces in a certain quantity of soil and along the fracture plain were presented by Rankine in 1857. According to his idea, a failure is likely to occur when the maximum principle stress at a particular place becomes close to being equal to the tensile stress. The hypothesis, however, says nothing about how the other two forces produce effects. His theory only holds true for ductile materials, not breakable ones. Maximum Stress Theory is another name for this theory.

Osborne Reynolds presented the evidence for sand expansion in 1887, and around this time, John Stuart Beresford and John Clibborn promoted the utilization of sand beds and increased water pressure.

The development of engineering geology flourished during the 19th century. William Penning wrote the renowned textbook in this subject during this time. Atterberg (1911) made a significant contribution by developing the concepts of homogenous cohesive soil consistency and the limits of plasticity, shrinkage, and liquidity (Atterberg, 1911).

Terzaghi (1925) accomplished an outstanding job researching this topic and writing a book titled "Mechanics of earth construction based on soil physics." He added the stress and consolidation hypothesis as well as the requirement for various field observations.

Similar circumstances occurred at California's San Francis Dam. Engineering geologists were forced to concentrate on the big projects as well after its collapse, which resulted in the deaths of 426 people.

Casagrande (1932), who also invented the plasticity chart, conducted a thorough investigation of soil compaction, soft clays, and seepages.

The work of Terzaghi was modified by Meyerhof (1951), who also included the equation for deep and shallow foundations. Along with the depth term Nq (supercharge), Terzaghi also added the factors of depth and factors of inclination. S-q, a shape component, was also included.

Kallstenius (1963) constructed the geotechnical equipment used in 1947, including the hydraulic piezometer, SGI inclinometer, and settlement measuring instruments. Along with it, he had the idea to interpret and use several penetrometer types as well as the Iskymeter. Because of his dedication and efforts, he received praise from many people throughout the world for making advancements in soil samplers and the SGI piston.

Engineering geology is a field of study that has only recently gained recognition worldwide after beginning in the late 19th and early 20th centuries (De Mello, 1977).

Engineering geologists have made significant contributions to the study of geotechnical qualities such as slope strength, risk assessment, landslides, erosion, etc. At various building stages of public and private assignments, this field can collaborate with environmentalists, civil engineers, and many other disciplines. The primary responsibility of an engineering geologist is to fully satisfy anyone planning to develop a building or other geotechnical structure so that it will be durable and able to endure all types of natural disasters.

CHAPTER 3
GEOLOGY AND TECTONIC SETTING

3.1 Geology of Islamabad and Rawalpindi

The Eurasian and Indian plate collision, which is the primary cause of tectonism and governs the geology of this region, occurred about 20 million years ago. The Himalayas are still rising as a result of these plates' ongoing movement, which causes tectonic events. Geologists from all around the world have examined the many structures and strata that were generated as a result of this collision. The structural characteristics of the Potwar Plate and surrounding regions are represented in (Fig3.1), one of many maps that the Geological Survey of Pakistan (GSP) and other survey organizations have issued. Referencing a geologic map is the best way to discuss the geology of the Rawalpindi and Islamabad region (William et al, 1999).

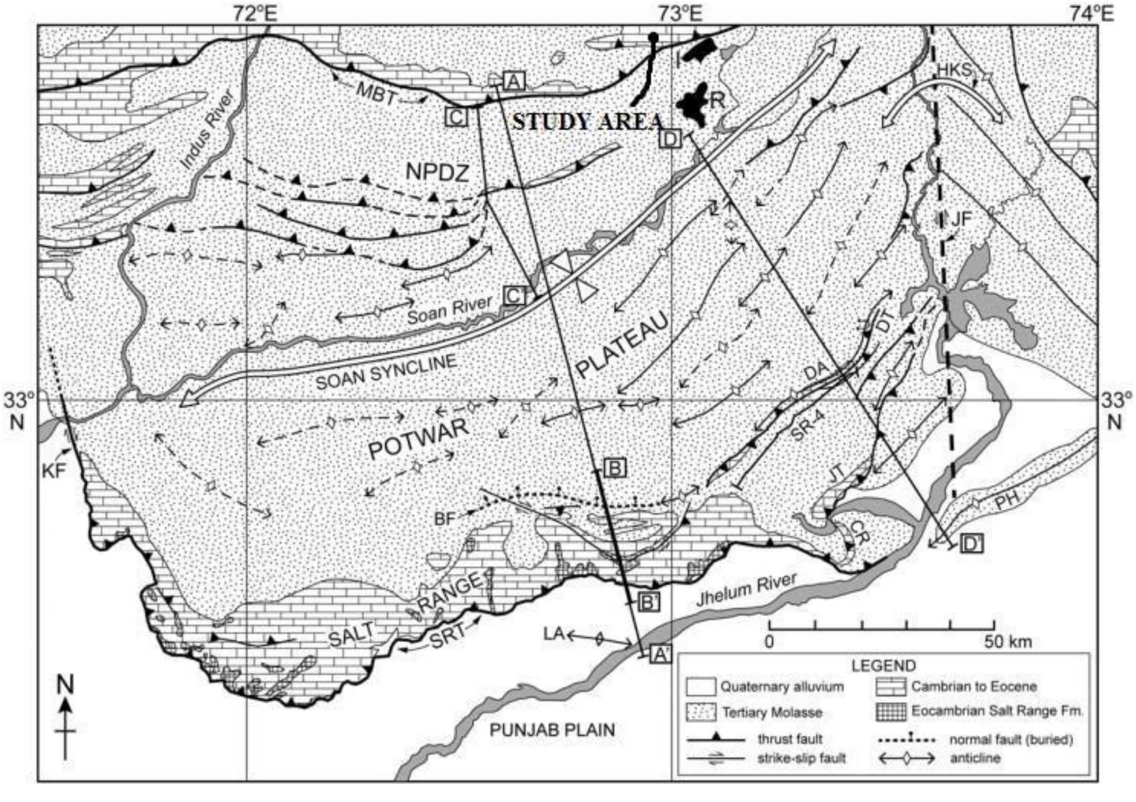


Figure 3.1. Geologic and tectonic map of study area (Jaswal et al. 1997)

3.2 Geological History

Sedimentary rocks that are 150 million years old and contain the geologic history of the mid-Jurassic to quaternary eras may be found in the Islamabad area. In this era, tectonic activity was not as prominent as marine deposition. However, continental deposition with somewhat slower subsidence occurred in the first two to four million years, and beyond two million years, there was significant erosion and intense tectonic activity with little local deposition. Dolomite and limestone from the Jurassic period were deposited on the continental side of the Indian plate prior to Eurasia and India colliding, making them the oldest rocks in the region. An unconformity is present between the Chichali and Samana suk Formation and it can be observed through the gap in the age. Furthermore, the shale with glauconite and chichali Formation's sandstone were accumulated in the environment which lacked oxygen and chemically reducing environment from late Jurassic- early cretaceous (khan et al., 2017).

3.3 Stratigraphy

3.3.1 Makarwal Group (Paleocene Age)

3.3.1.1 Hangu Formation

The lithology of the Hangu formation includes claystones, intercalated shales, and quartzose sandstone. Sandstone has brittleness and a brownish tint, and it is also quartzose since it contains more than 90% quartz. Shale and Claystone, in contrast, have a greenish hue. According to observations, the sequence in the Hangu Formation is fining upward and has a thickness of between 6 and 10 metres. It also creates a conformable contact with the Lockhart Formation (Williams et al., 1999).

3.3.1.2 Lockhart limestone

The majority of it is limestone, as the name suggests, although it also contains shale and marl. There are fossils and a range of grey hues in the limestone. The marl is grey to black in hue and also contains fossils. Aside from that, Lockhart limestone can be as thick as 280 metres and is compatible with the Patala Formation (Williams et al., 1999).

3.3.2 Surghur Group (Jurassic-Lower Cretaceous Age)

3.3.2.1 Samana Suk Formation

This formation's lithology consists of greenish grey Marl with thin beds and grey to brown limestone in other locations. At least 190 meters of thickness are possible, with a maximum thickness of 360 meters. This formation has an unknown base that is not exposed that makes an unconformable contact with the Chichali Formation above (Fatmi, 1990).

3.3.2.2 Chichali Formation

The glauconitic sandstone, shale, claystone, and milestone that make up this formation's lithology. These lithology have different colors; for example, siltstone and limestone have a greenish grey hue, whereas shale has a more greenish hue or a dark grey hue. Glauconitic sandstone, on the other hand, has a range of grain sizes, from fine to coarse. It can be up to 50 meters thick before grading into touch with the Lumshiwal Formation above (Williams et al., 1999).

3.3.2.3 Lumshiwal Formation

Sandstone, limestone, shale, and milestone make up the majority of this formation. These lithology, such as limestone, have thin beds with a yellowish tint and a lot of sand. There are some fossilised ammonoids and brachiopods scattered throughout the somewhat thick sandstone, which also contains glauconites. Unconformable contact is made between this structure and the Hangu above (Williams et al., 1999).

3.3.3 Cherat Group (Lower Eocene Age)

3.3.3.1 Margalla Hill Limestone

As the name implies, marl, shale, and limestone also make up the majority of this formation. The limestone in this area is nodular, thick-bedded, and dark grey. The marl in this instance is tougher and has a grey tone. The Margalla Hill Limestone is made up of splintery, greenish-gray shale. This formation's overall thickness ranges from 60 to 90 m, making a conformable contact with the Chorgali Formation (Williams et al., 1999).

3.3.3.2 Chorgali Formation

Upper and lower portions make to the Chorgali Formation. This formation comprises fossils and yellowish limestone in the upper part that may perhaps contain some chert. The Marl is a light grey tint, and its total thickness in this region can reach 120 meters.

On the other side, the lower half is primarily composed of shale with a greenish grey tint, with limestone interspersed as well. Coquina beds also contain several foraminifera fossils. Kuldana Formation is in conformable touch with the entire unit (Williams et al., 1999).

3.3.3.3 Kuldana Formation

Claystone, marl, limestone, and a small number of sandstones make up the majority of the lithology in this region, both marine and non-marine. Currently, marl is a light greyish tint with only a trace of gypsum. Limestone's color is described as white-light brown. This formation's total thickness can reach 120 meters, and its unconformable boundary with the Murree Formation of the Rawalpindi Group (Williams et al., 1999).

3.3.4 Rawalpindi Group (Miocene Age)

3.3.4.1 Murree Formation

Wynne (1874) called them the "Mari Group," but the Stratigraphic Committee of Pakistan altered their name to the Murree Formation (Fatmi, 1972).

Sandstone and siltstone are predominant in this formation, with conglomerates being less common. Sandstone is reddish grey in color and varies in thickness across this formation, but it can reach heights of up to 2900 meters (Williams et al., 1999).

This formation is conformably overlain by Kamlial Formation but uncomfortably overlies Kohat Formation (Amjad Ali, 1997).

3.3.4.2 Kamlial Formation

The Stratigraphic Committee of Pakistan was the one who came up with the name Kamlial Formation (Fatmi, 1973).

Shale, siltstone, conglomerate, and sandstone are all components of the lithology. The contrast between the Kamlial Formation and the Murree Formation can be seen in the fact that the Kamlial Formation also exhibits rich tourmaline and spheroidal weathering. Chinji Formation is conformably overlain by the thickness, which is between 1500 and 1600 meters thick and 115 kilometers from Islamabad (Johnson et al., 1985).

3.3.5 Siwalik Group (Neogene to Pleistocene Age)

3.3.5.1 Chinji Formation

The name was termed to “Chinji Stage” which was then agreed by the Stratigraphic Committee of Pakistan (Shah, 1977).

The majority of the rock in this Formation is breakable siltstone, with sandstone interspersed. The sandstone is a smokey colour with a brown undertone. Because of its brick-red hue, it is well-known. Chinji is between 850 and 1170 meters thick, and the Nagri formation is uniformly overlaid (Johnson et al., 1985).

3.3.5.2 Nagri Formation

Nagri Formation was named as Nagri stage of Pilgrims before but it was renamed by Lewis which was then agreed by the Stratigraphic Committee of Pakistan (Shah, 1977).

Here, greenish grey sandstone predominates with clay in between. Conglomerates are another component of the formation that are found in various places. Ilmenite and magnetite combine to generate the distinctive pattern of salt and pepper. Between 500 and 900 meters thick, Dhok Pathan conformably covers this formation (Johnson et al**.)**

3.3.5.3 Dhok Pathan Formation

The pilgrim came up with the name “Dhok Pathan” which was changed to “Dhok Pathan Formation” by cotter in 1933, and now, this name has been formalized (Fatmi, 1973).

In this region, hard claystone and orange-colored siltstone predominate alongside greyish-colored sandstone. Its thickness is estimated to be 500 to 820 meters. Soan Formation conformably overlies it (Johnson et al., 1985).

3.3.5.4 Soan Formation

The name was accepted by the Stratigraphic Committee of Pakistan after it was given by Kravtchenko in 1964 (Rahman, 1968).

This formation's lithology includes conglomerates, siltstone, claystone, and sandstone with clays. In contrast to claystone, which is pale pink and brownish, sandstone has visible grains and a greenish grey tint. Lei conglomerates are uniformly layered on top of the thickness, which ranges from 200 to 300 meters

(Johnson et al., 1982).

3.3.6 Units on Surface (Pleistocene-Holocene)

3.3.6.1 Lei Conglomerate

In 1910, Pilgrim proposed the name "Boulder Conglomerate," but Gill later proposed the name "Lei Conglomerates." This region is more level and closer to sea level. However, there are folds and faults in the immediate vicinity. Overlaying Rawalpindi and the Siwalik group are Lei Conglomerates. The age of the Lei Conglomerates may be roughly approximated by the fission track method, which shows that the maximum age of the Lei Conglomerates is around 1.6-18 million years because the volcanic ash is younger than Soan Formation and older than the Lei Conglomerates (Johnson et al., 1982)**.**

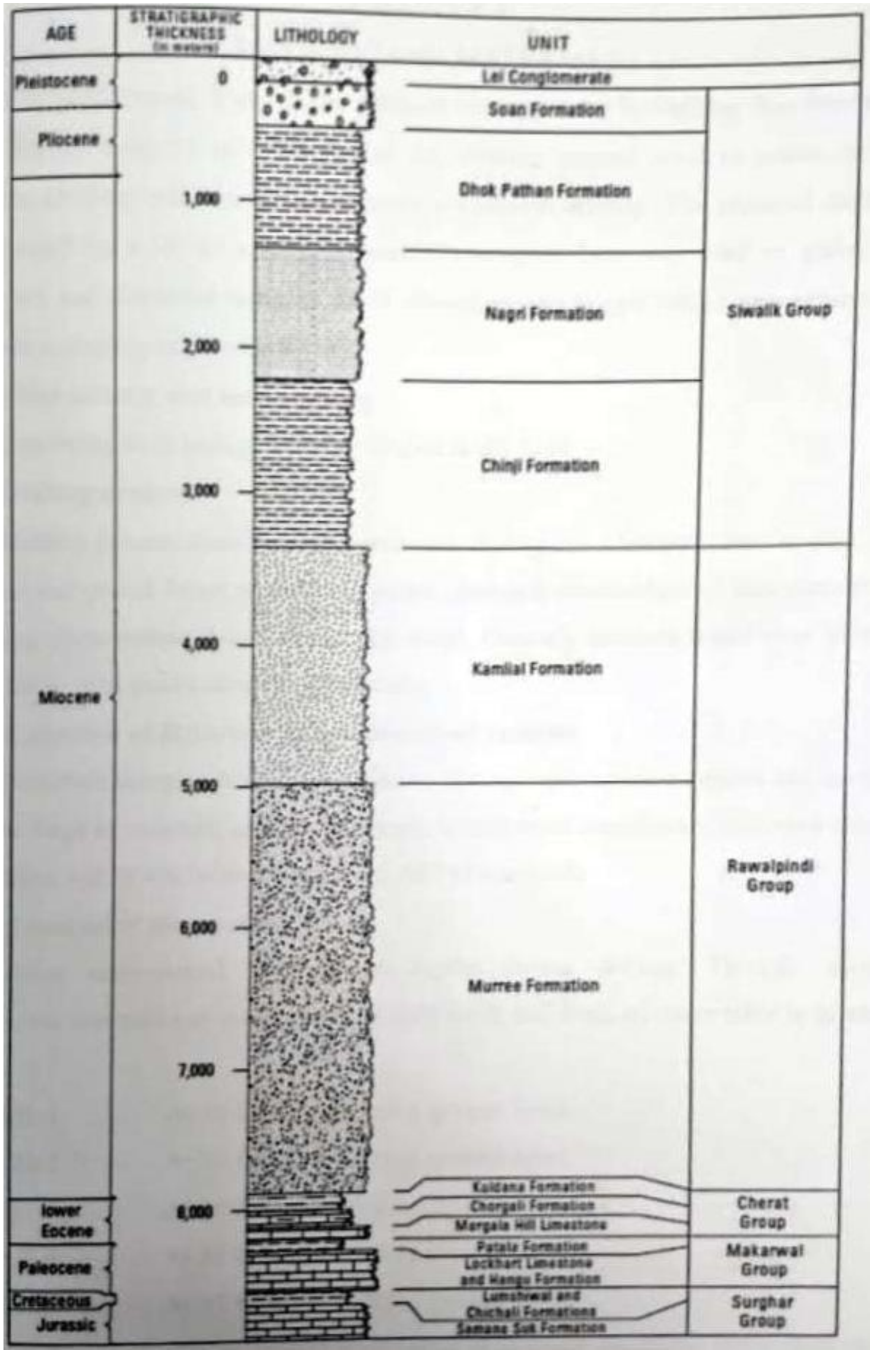


Figure 3.2. Generalized Stratigraphic column of Rawalpindi and Islamabad (Litsey, 1958).

CHAPTER 4

METHODS AND MATERIALS

4.1 Field investigation

The study area was gone through geotechnical investigation by drilling 2 boreholes in which the depth was around 100 feet to observe the soil properties within the ground. The method chose for drilling was percussion drilling. Standard Penetration tests (SPT) were performed at depth interval of 3.0ft, undisturbed samples could not be retrieved due to high moisture and loose strata at 10-20ft.

Sequence of the investigation

Following sequence was adopted for the investigation:

I. Drilling of 02 boreholes

II. Conduct of standard penetration tests

III. Collection of soil samples (disturbed / undisturbed)

IV. Performing UCS test

V. Determination of allowable bearing pressure.

4.1.1 Standard Penetration Test (ASTM D1586)

Standard penetration test (SPT) is a variety of in-situ soil test. SPT is an important test to identify the soil properties.

4.1.2 Equipment Used

I. Hammer of 63.5kg

II. Split Spoon Sampler

III. Shelby tube

IV. Guiding rod

V. Drilling rig

VI. Driving head also known as the Anvil.

4.1.3 Methodology

When the bore is drilled, the sampler is put into the borehole with the help of hammer (63.5 kg) falling from the height of 750mm and number of blows required to affect a penetration of 150mm are recorded. The recorded blow count (N Values) are then correlated to different soil properties. The condition indicated very stiff soil. SPT's were conducted in all boreholes. SPT values are taken as refusal when the values are greater than 50.

Bearing capacity of the foundation can be calculated with Standard penetration test (SPT) and with unconfined compression strength test (UCS). Un drained shear strength of clay from SPT can be calculated with the equation:

Unconfined Compressive Strength $qu(tsf) = 0.125*Ncor; su = qu/2$

For sandy, silty sand and fine gravely soil Meyerhof's equation is more useful to calculate the bearing capacity

$qa(ksf) = (Ncor/4)(1+1/B)2*Kd$ for $B>4ft$ where $Kd = 1+0.33(D/B)$ should be (=, <) 1.33

In case where gravely soil strata is encountered the bearing capacity is calculated by using the number of blows by using formula

$Qa(Ksf) = 0.116(Ncor)$ for $B = 3$ and 4ft and $D/B = 1$

If the no. of blows are more than 50 the result is taken as refusal and the test is stopped.

4.1.4 Safety measures

I. The sampler has to be in a proper working condition.
II. The cutting shoe must not be broken.
III. The height of hammer from where it needs to be dropped has to be 76cm or else the values of N will not be accurate.
IV. The drill rods must be in normal shape, if for some reason they are bent, the results will not be accurate.
V. The bottom potion of borehole has to be clean before performing the test.

Figure 4.1. Drilling rig used for performing SPT test.

Figure 4.2. Samples taken out from SPT test at BH-4 depth 6m.

Figure 4.3. Samples from BH-4 being put in the polythene bag to maintain moisture.

Figure 4.4. Drop hammer of 63.5kg used for SPT.

4.2 Lab testing

Samples retrieved from boreholes were examined in the field and then transported to testing laboratory for relevant laboratory testing. The laboratory tests were performed on selected soil samples to determine the engineering characteristics of the subsurface strata.

I. Sieve Analysis.
II. Atterburg limits.

4.3 Subsurface strata

General stratigraphy of the project area, as deduced from the site investigations duly corrected in the light of laboratory test results are as following.

a) Soft to stiff to dense soil profile was observed and subsoil consists of Lean Clay with Sand and Gravel.

b) Fill material was not encountered up to 6-10ft at the time of investigation

c) Natural moisture content varies from 13% to 31%.

d) Liquid limit varies from 30% to 33% and plasticity index varies from 13% to 15%.

4.4 Ground water table

Seepage water was encountered at 11ft at time of investigation.

4.5 Sieve analysis (ASTM D-6913)

For the categorization of soil, Sieve analysis was performed. To conduct this test, the soil samples are put in the binder for drying. After this they are gone through the sieves which are stacked in the decreasing order from top to bottom.

The smallest number of sieves that was used was of 200 whereas the wider sieve consisted of no.4. The weight of the samples in every sieve was calculated and the results were plotted on the graph which showed the arrangement of soil samples.

4.5.1 Equipment

I. Stacked sieves with pan and cover
II. Electronic weighting machine having accuracy of 0.01 grams

III. Ceramic mortar and pestle to crush the lumped soil
IV. Sieve shaker
V. Binder

4.5.2 Methodology

100 grams of sample was taken and dried in the binder for 24 hours. After taking the samples out, if the soil has combined together then pestle and mortar are used to crush them to powdered form. The sieves are then stacked above each other with larger hole sizes above the smaller ones. The sieve no. that is placed in the bottom most portion is 200. A pan is put in the bottom most portion to collect the remaining soil. The sieves must be properly cleaned before they are used. In the case where the soil particles gets stuck in the holes, the brush is used to clear the path. The soil sample is poured at the top most sieve and then it is shacked. After all the samples have passed, the weight of the soil is measured that is retained on the sieve.

4.5.3 Calculation

To find out the overall percentage of passing of soil from each sieve, the soil percentage that is left on the sieve is calculated. This is calculated by using the overall weight of sample of soil that is used.

%Soil retained= weight of soil on sieve/weight of total soil * 100

4.5.4 Safety measures

I. Appropriate care needs to be taken for accurate results.
II. If the holes of sieves are soldered the results from large breaks or a lot of small breaks should be avoided.
III. The warm samples must not be used for sieving because it changes the mesh of sieve no. 100 and 200.
IV. If the sieve has a break in the main body, it should be ignored.
V. It should be taken care that no material is lost in the process of rinsing.
VI. Do not put too much weight on the sieves.

VII. Care must be taken not to waste any material during washing of sieve. 200 due to water pressure.

4.5.5 Constraints

The sieve analysis is not a good method for the samples that have flat or elongated shape, only round and spherical shaped grains can be sieved through this method. An error is likely to occur in the case of 100 no. sieve because the sample needs to be shaken more to pass them out. If the liquid does not affect the sample, it can be used for the sieve analysis.

Figure.4.5. Showing stacked sieves.

Figure.4.6. Using pestle and mortar to crush the lumped soil.

Atterburg Limits and Plasticity Charts

4.6 Atterburg limits

The limits of soil properties for defining the characteristics of fine-grained soil was introduced by a scientist from Sweden whose name was Albert Atterburg. Ever since then, his methods are still being used to find out Plastic limit, Liquid limit and shrinkage limit of the soil. The soil can be of 4 types that depends on the quantity of water present in it which are liquid, plastic, semi-solid and solid. The characteristics of soil varies in each state due to which the characteristic regarding the engineering perspectives also vary. By definition, the liquid limit is the one in which the soil has maximum moisture content in which the soil is in liquid phase whereas the plastic limit is the one in which the soil acts in a plastic manner under which its shape can be deformed into any other shape without producing cracks.

These tests are applied on the soil which are clayey or silty because these are the ones which expand or shrink because of change in amount of moisture. The limits can be identified by using:

I. Liquid limit test
II. Plastic limit test

4.6.1 Liquid limit test

i) Instruments

a. Electronic weighting machine
b. Containers
c. Grooving tool
d. Spatula
e. Sieve no. 40

ii) Methodology

The soil sample is first gone through the sieve no. 40 then some distilled water is added to it to produce a smooth paste like substance. This paste is then put into the Casagrande cup up to 10mm and a groove is marked by using the grooving tool. The thickness of this groove was around 12mm. The crank of the device is then rotated and the blows produced are counted until the groove is closed. Right when the groove is closed, the soil sample is weighed on the electronic weighting device and then put it into the binder for around 17 hours afterwards. More water is added to the remaining sample and the process is repeated. The results are plotted on the graph in which the N value against the amount of blows shows the liquid limit of soil.

iii) Safety measures

a. The apparatus needs to be cleaned after every test.
b. Counting of blows has to be counted only till the grooves are closed.
c. Average amount of blows has to range around 10-40.

Figure 4.25. Carrying out the liquid limit test by Casagrande's method.

4.6.2 Plastic limit test

i) Instruments

a. Dish for mixing

b. Spatula

c. Glass plate

d. Sieve no. 40 with pan

ii) Methodology

After taking the required sample of soil, water is added into it so that the soil does not stick to the hands while rolling. After molding it into an ellipse shape, it is further rolled between fingers or palms in 90 strokes within minute. It is rolled until the cracks start to form and it does not further roll. Afterwards, the sample is weighed to find the moisture content in the soil and then the can is put into the binder for around 17 hours and the amount of water is calculated in every trial.

iii) Constraints

For the finding out the liquid limits the test is performed, and that test can eliminate the natural residual bonds present in the soil. Due to this method, those are not possible to identify.

CHAPTER 5

RESULTS AND DISCUSSION

5.1 Results

The investigation of the study area was done by following the methods:

I. Drilling of 2 boreholes up to 100ft depth below existing ground level (EGL) by using percussion drilling machine.
II. Performance of in-situ testing.
III. Collection of disturbed samples.
IV. Collection of rock samples.
V. Lab testing.
VI. Interpretation of Geotechnical investigation report.

The strata were observed by going through the samples collected on the field and the borehole logs. It can be seen from the summary table and the borehole logs mentioned below. Other than that, the moisture content tests were also performed and their results can be seen again in the summarized table below.

The bearing capacity of the soil can be acquired by using SPT blows and also Unconfined Compression Strength test UCS. In our case, almost all bearing capacity values were determined by using only the SPT test.

The calculation of grain sizes at various stages were applied and their results from every BH are shown in the tables and in figures below.

Sieve passing percentage for BH-1

Table 5.1.BH-1 details at 3-m depth.

Borehole	**BH-01**
Depth (ft.)	**3**
Natural Moisture Content	**14.3%**
Sieve Number	**Passing Percentage**
No. 4	**94.1**
No. 10	**92.1**
No. 40	**89.1**
No. 200	**86.3**
Description	**Clay**

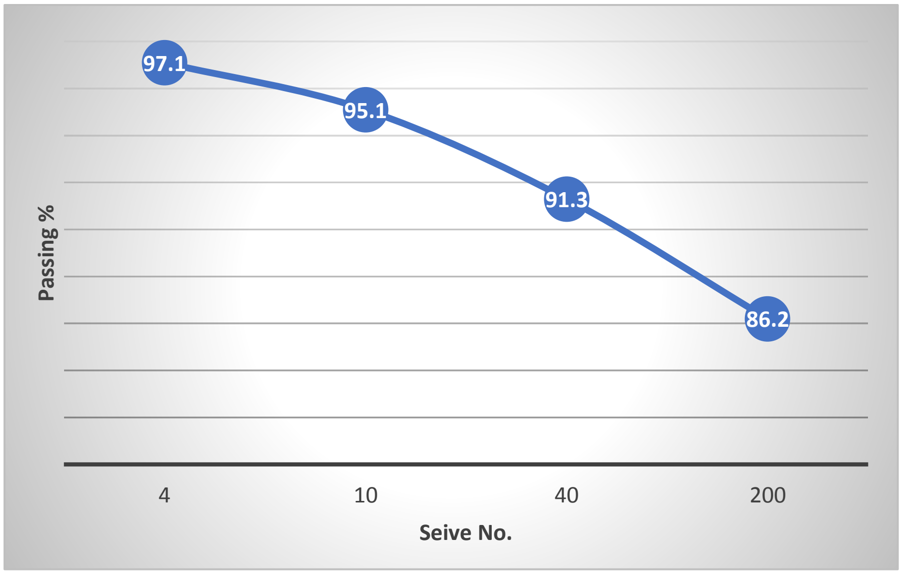


Figure 5.1.BH-1 Sieve Analysis at 3-m depth.

Table 5.2 BH-1 details at 6-m depth.

Borehole	**BH-01**
Depth (ft.)	**6**
Natural Moisture Content	**13.9%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.4**
No. 40	**96.1**
No. 200	**90.2**
Description	**Clay**

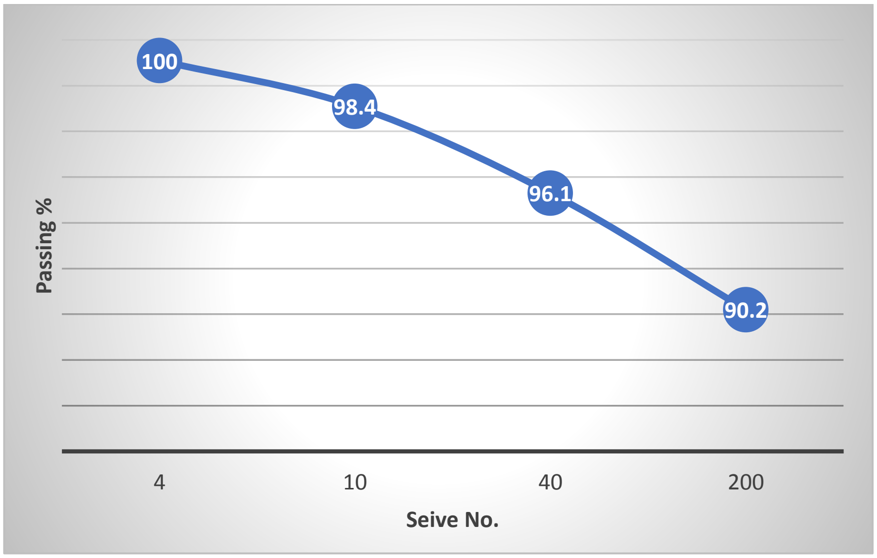


Figure 5.2.BH-1 Sieve Analysis at 6-m depth.

Table 5.3.BH-1 details at 9-m depth.

Borehole	**BH-01**
Depth (ft.)	**9**
Natural Moisture Content	**15.2%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.7**
No. 40	**97.1**
No. 200	**94.2**
Description	**Clay**

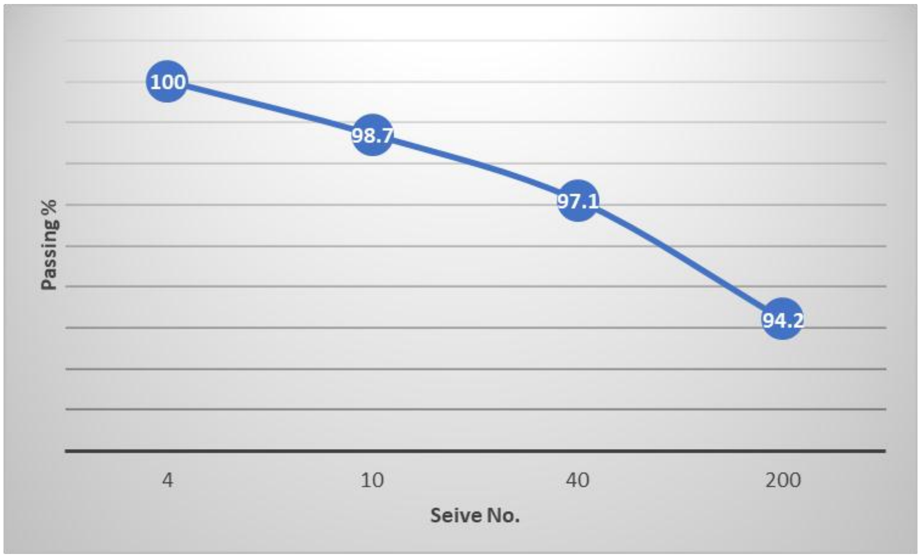


Figure 5.3.BH-1 Sieve Analysis at 9-m depth.

Table 5.4.BH-1 details at 12-m depth.

Borehole	**BH-01**
Depth (ft.)	**12**
Natural Moisture Content	**15.2%**
Sieve Number	**Passing Percentage**
No. 4	**98.1**
No. 10	**96.3**
No. 40	**94.1**
No. 200	**89.3**
Description	**Clay**

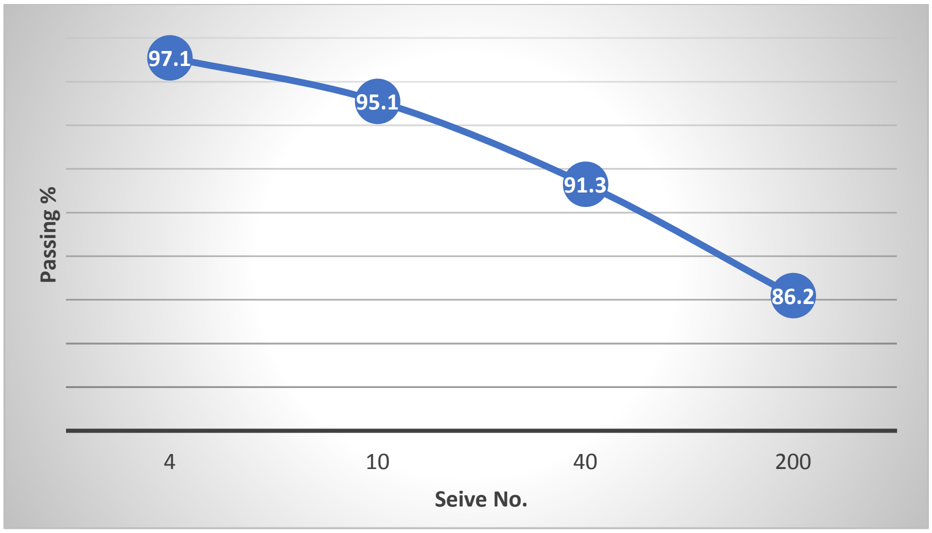


Figure 5.4.BH-1 Sieve Analysis at 12-m depth.

Table 5.5.BH-1 details at 15-m depth.

Borehole	**BH-01**
Depth (ft.)	**15**
Natural Moisture Content	**14.9%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**96.1**
No. 40	**94.7**
No. 200	**88.1**
Description	**Clay**

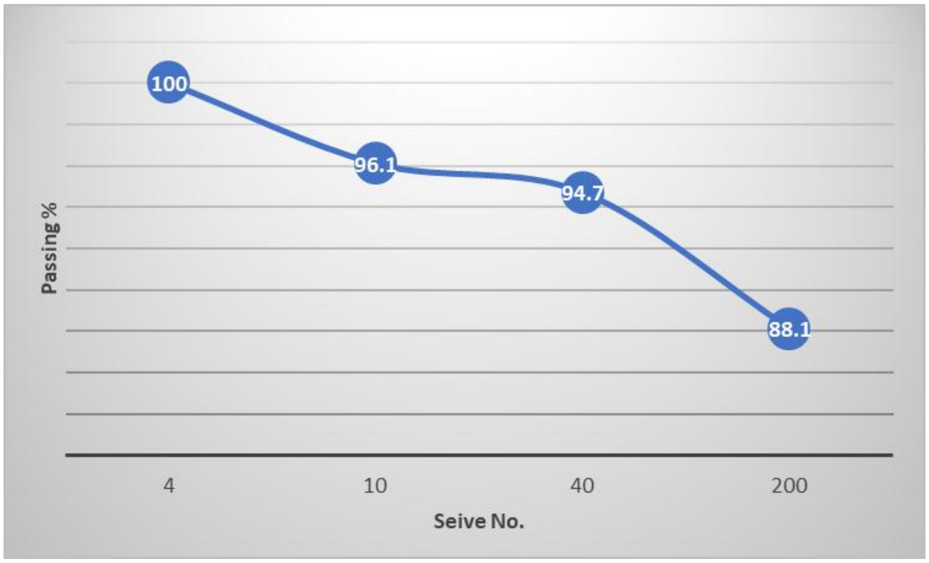


Figure 5.5.BH-1 Sieve Analysis at 15-m depth.

Table 5.6.BH-1 details at 18-m depth.

Borehole	**BH-01**
Depth (ft.)	**18**
Natural Moisture Content	**15.2%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.1**
No. 40	**96.1**
No. 200	**92.7**
Description	**Clay**

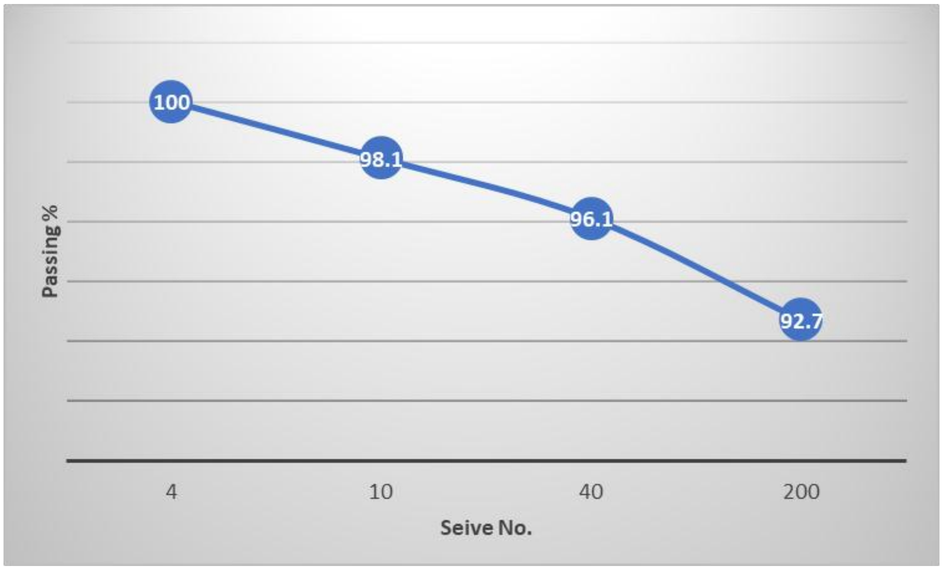


Figure 5.6.BH-1 Sieve Analysis at 18-m depth.

Table 5.7.BH-1 details at 21-m depth.

Borehole	**BH-01**
Depth (ft.)	**21**
Natural Moisture Content	**16.7%**
Sieve Number	**Passing Percentage**
No. 4	**98.1**
No. 10	**96.3**
No. 40	**92.1**
No. 200	**87.4**
Description	**Clay**

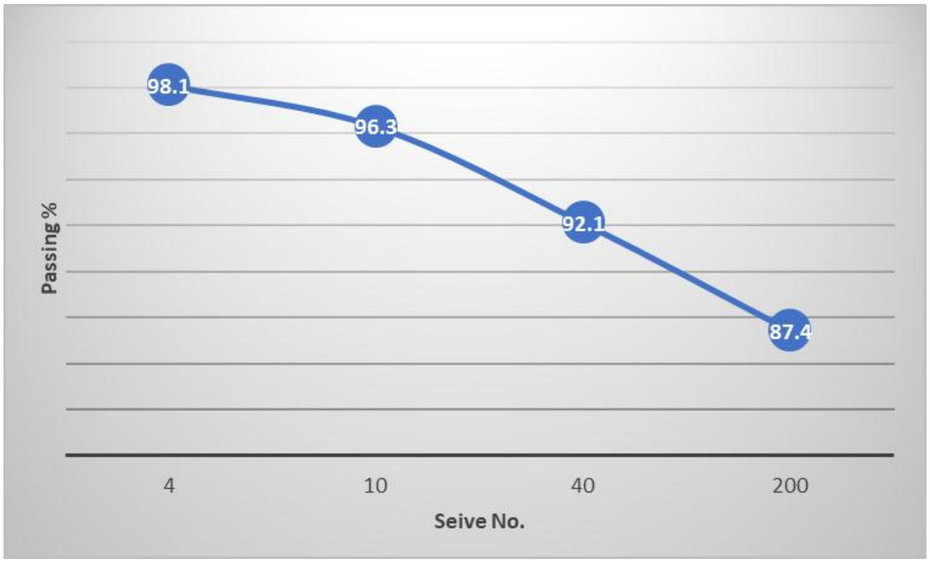


Figure 5.7.BH-1 Sieve Analysis at 21-m depth.

Table 5.8.BH-1 details at 24-m depth.

Borehole	**BH-01**
Depth (ft.)	**24**
Natural Moisture Content	**16.1%**
Sieve Number	**Passing Percentage**
No. 4	**97.1**
No. 10	**95.1**
No. 40	**91.3**
No. 200	**86.2**
Description	**Clay**

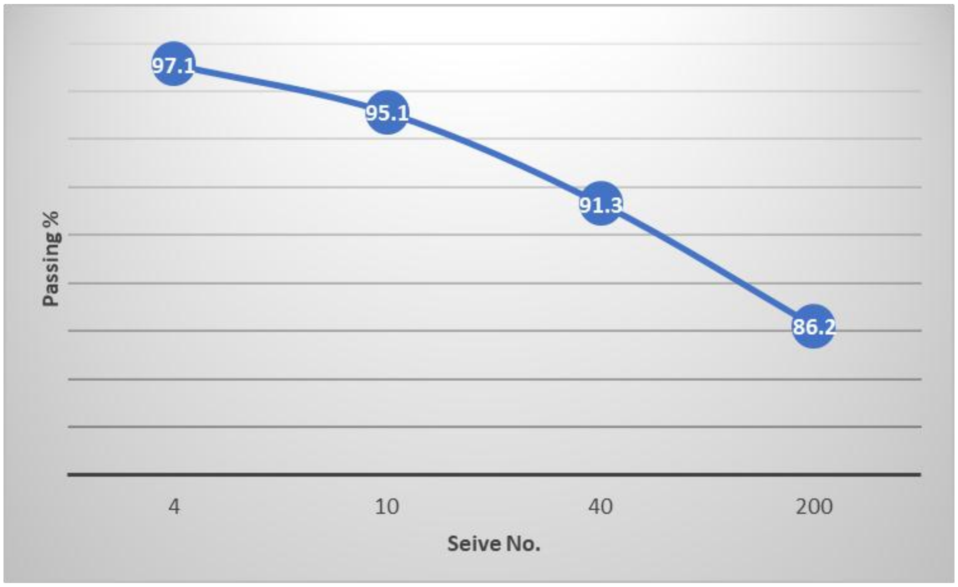


Figure 5.8.BH-1 Sieve Analysis at 24-m depth.

Table 5.9.BH-1 details at 27-m depth.

Borehole	**BH-01**
Depth (ft.)	**27**
Natural Moisture Content	**15.7%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.8**
No. 40	**97.6**
No. 200	**95.2**
Description	**Clay**

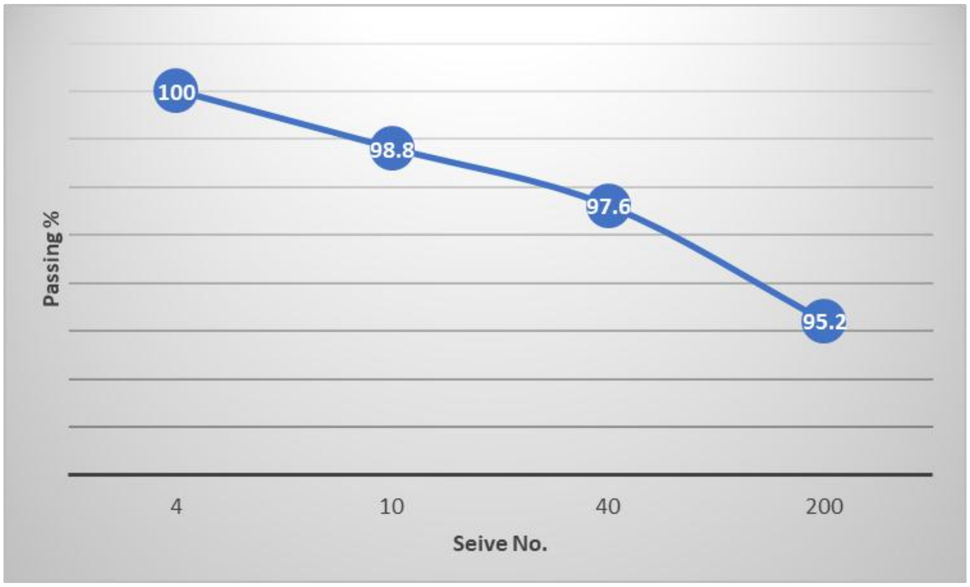


Figure 5.9.BH-1 Sieve Analysis at 27-m depth.

Table 5.10.BH-1 details at 30-m depth.

Borehole	**BH-01**
Depth (ft.)	**30**
Natural Moisture Content	**16.1%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**97.1**
No. 40	**95.2**
No. 200	**92.3**
Description	**Clay**

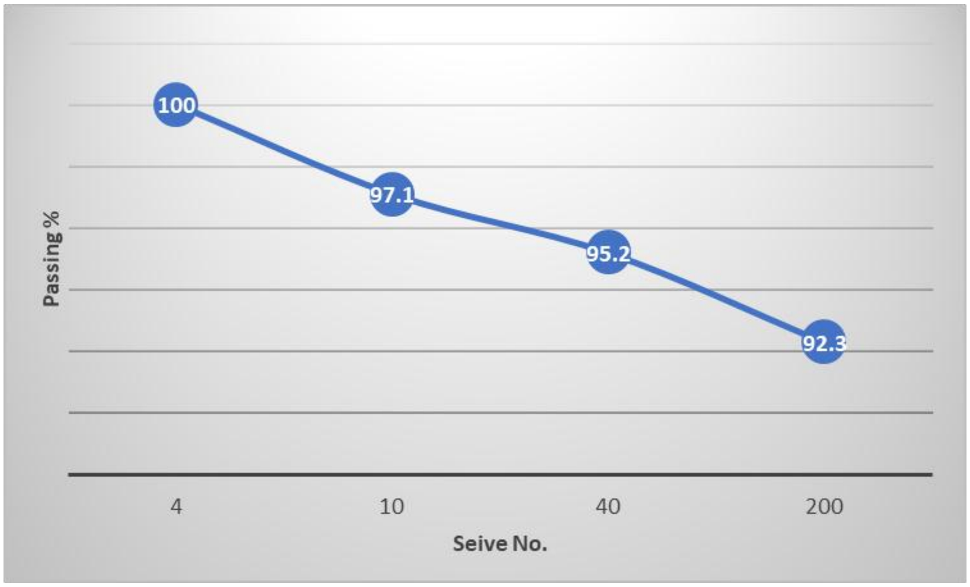


Figure 5.10.BH-1 Sieve Analysis at 30-m depth.

Table 5.11.BH-1 details at 35-m depth.

Borehole	**BH-01**
Depth (ft.)	**35**
Natural Moisture Content	**14.1%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.7**
No. 40	**97.5**
No. 200	**94.3**
Description	**Clay**

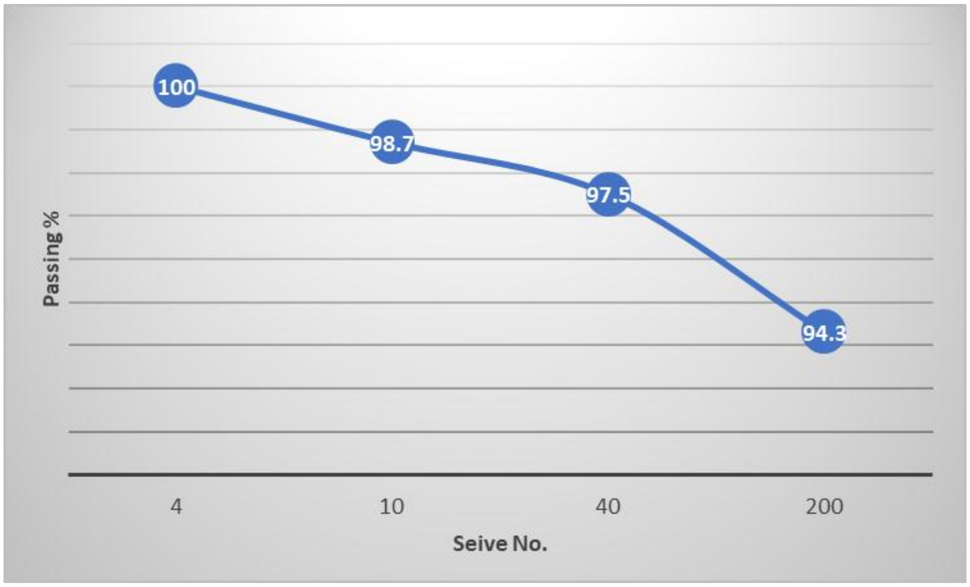


Figure 5.11.BH-1 Sieve Analysis at 35-m depth.

Table 5.12.BH-1 details at 40-m depth.

Borehole	**BH-01**
Depth (ft.)	**40**
Natural Moisture Content	**18.1%**
Sieve Number	**Passing Percentage**
No. 4	**98.1**
No. 10	**96.3**
No. 40	**94.1**
No. 200	**90.2**
Description	**Clay**

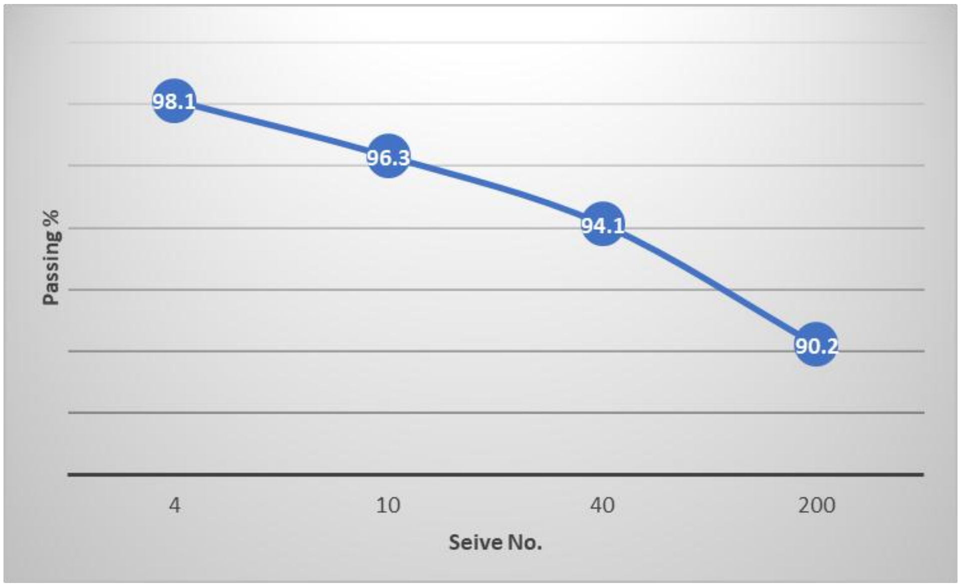


Figure 5.12.BH-1 Sieve Analysis at 40-m depth.

Table 5.13.BH-1 details at 45-m depth.

Borehole	**BH-01**
Depth (ft.)	**45**
Natural Moisture Content	**19.1%**
Sieve Number	**Passing Percentage**
No. 4	**94.1**
No. 10	**96.1**
No. 40	**94.1**
No. 200	**92.3**
Description	**Clay**

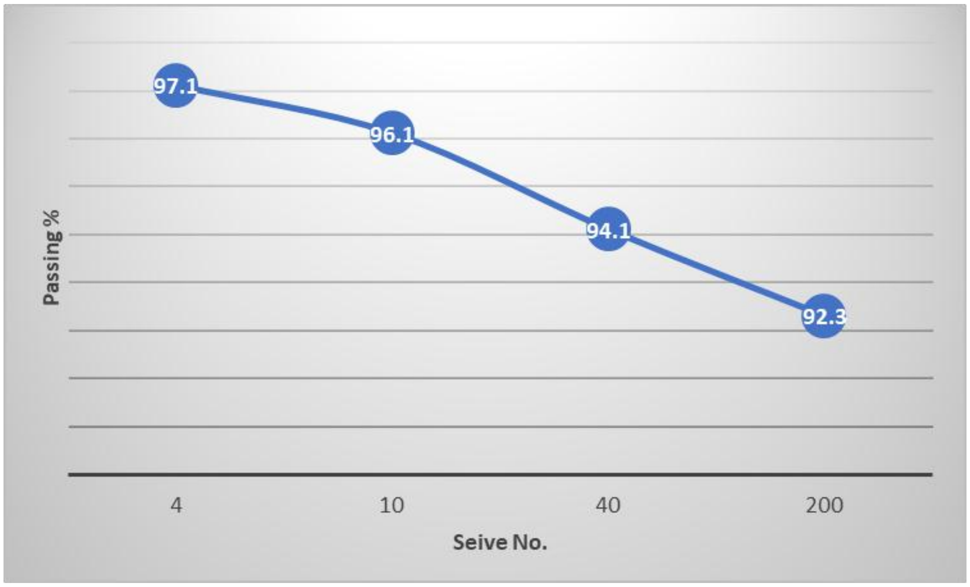


Figure 5.13.BH-1 Sieve Analysis at 45-m depth.

Table 5.14.BH-1 details at 50-m depth.

Borehole	**BH-01**
Depth (ft.)	**50**
Natural Moisture Content	**18.4%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**97.1**
No. 40	**96.3**
No. 200	**94.7**
Description	**Clay**

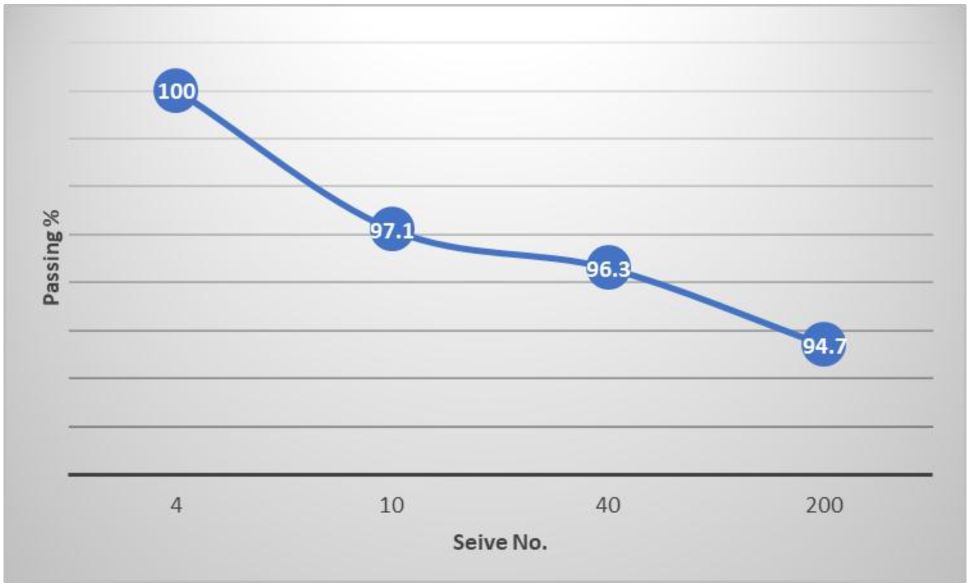


Figure 5.14.BH-1 Sieve Analysis at 50-m depth.

Table 5.15.BH-1 details at 55-m depth.

Borehole	**BH-01**
Depth (ft.)	**55**
Natural Moisture Content	**18.8%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.2**
No. 40	**96.1**
No. 200	**94.7**
Description	**Clay**

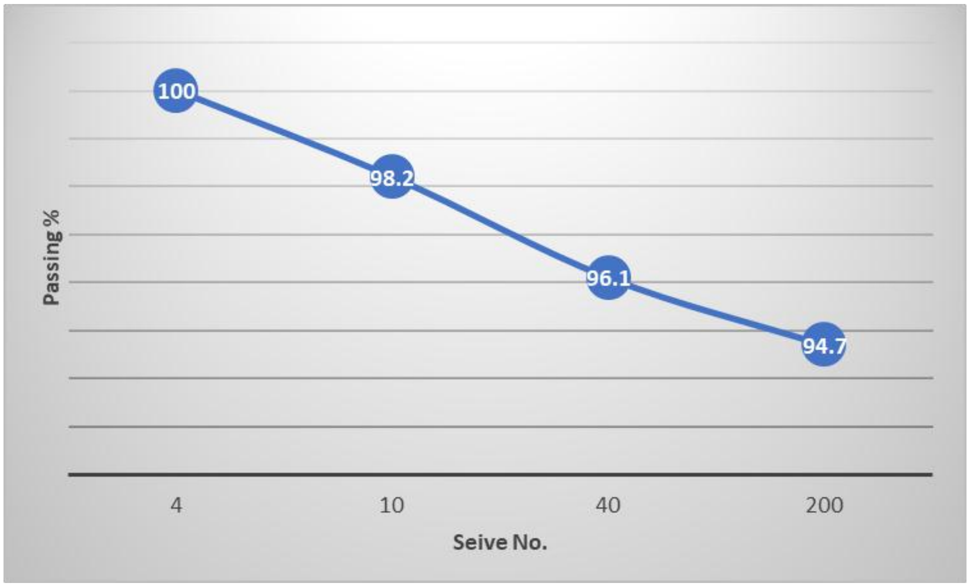


Figure 5.15.BH-1 Sieve Analysis at 55-m depth.

Table 5.16.BH-1 details at 60-m depth.

Borehole	**BH-01**
Depth (ft.)	**60**
Natural Moisture Content	**19.1%**
Sieve Number	**Passing Percentage**
No. 4	**94.3**
No. 10	**90.2**
No. 40	**89.1**
No. 200	**81.3**
Description	**Clay**

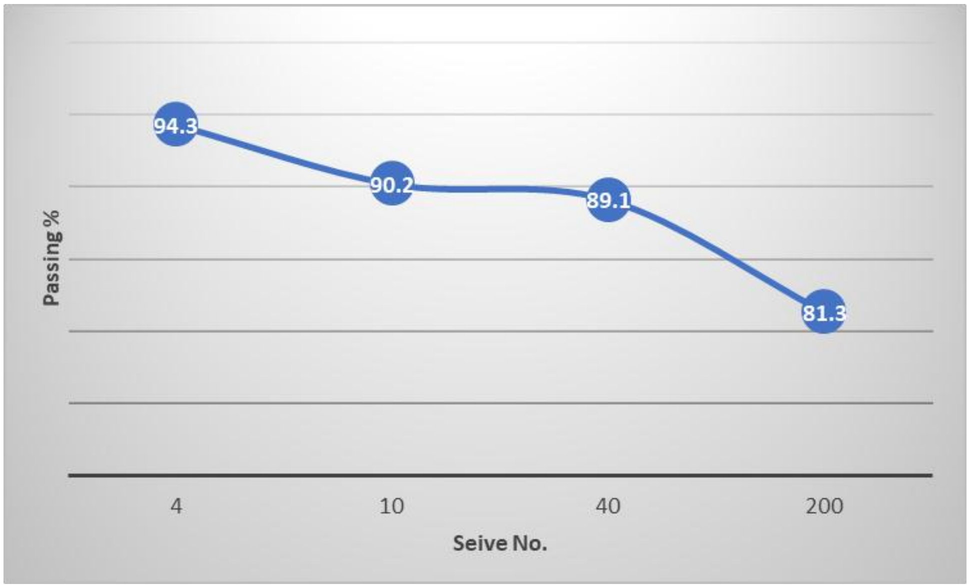


Figure 5.16.BH-1 Sieve Analysis at 60-m depth.

Table 5.17.BH-1 details at 65-m depth.

Borehole	**BH-01**
Depth (ft.)	**65**
Natural Moisture Content	**18.2%**
Sieve Number	**Passing Percentage**
No. 4	**91.7**
No. 10	**87.8**
No. 40	**83.4**
No. 200	**79.1**
Description	**Clay**

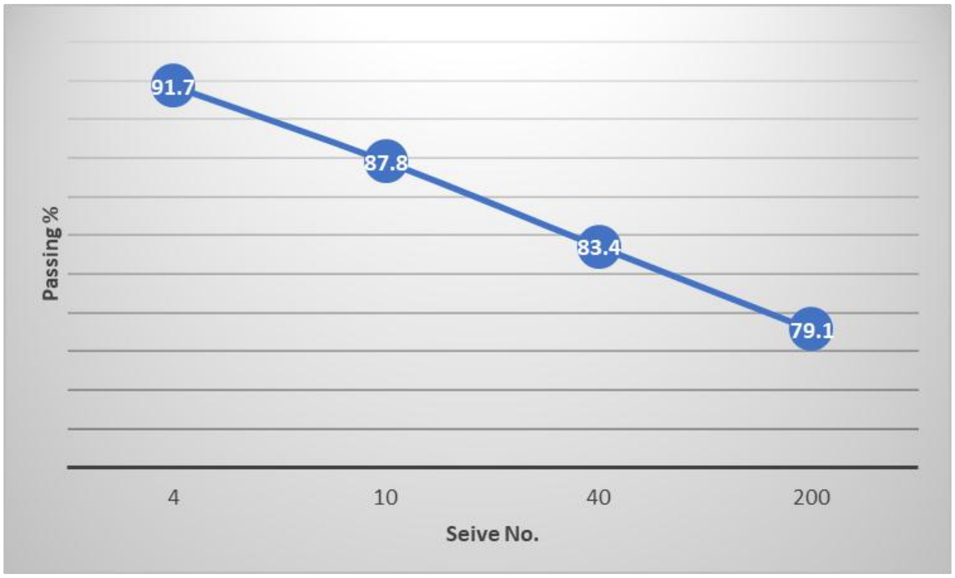


Figure 5.17.BH-1 Sieve Analysis at 65-m depth.

Table 5.18.BH-1 details at 70-m depth.

Borehole	**BH-01**
Depth (ft.)	**70**
Natural Moisture Content	**17.4%**
Sieve Number	**Passing Percentage**
No. 4	**89.1**
No. 10	**84.2**
No. 40	**76.1**
No. 200	**70.3**
Description	**Clay**

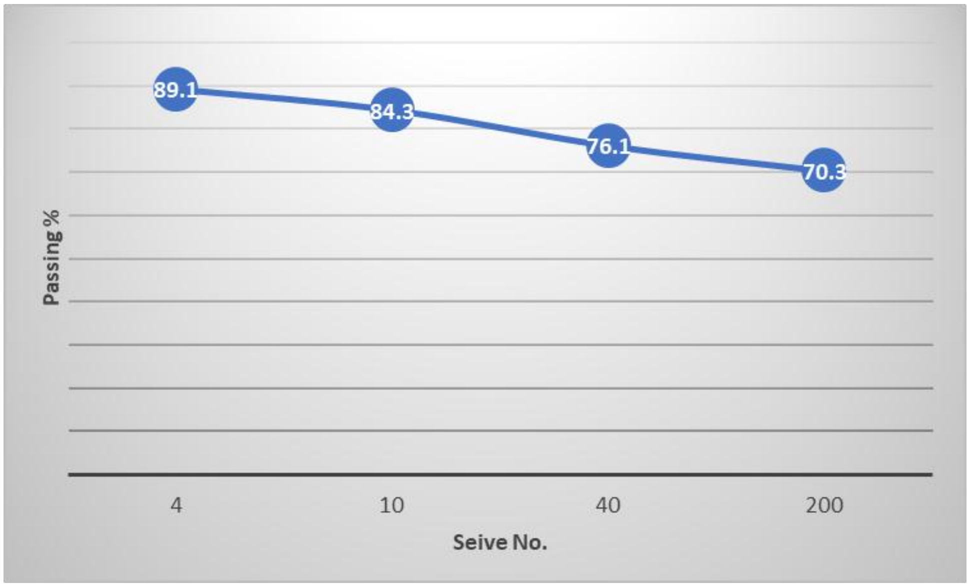


Figure 5.18.BH-1 Sieve Analysis at 70-m depth.

Table 5.19.BH-1 details at 75-m depth.

Borehole	**BH-01**
Depth (ft.)	**75**
Natural Moisture Content	**16.8%**
Sieve Number	**Passing Percentage**
No. 4	**79.8**
No. 10	**74.4**
No. 40	**70.2**
No. 200	**68.2**
Description	**Clay**

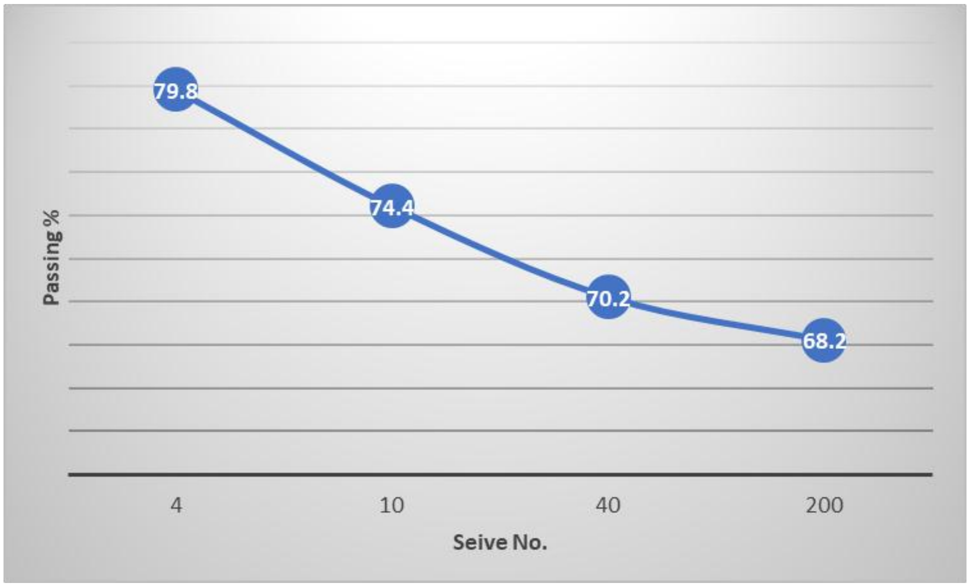


Figure 5.19.BH-1 Sieve Analysis at 75-m depth.

Table 5.20.BH-1 details at 80-m depth.

Borehole	**BH-01**
Depth (ft.)	**80**
Natural Moisture Content	**18.4%**
Sieve Number	**Passing Percentage**
No. 4	**84.1**
No. 10	**80.2**
No. 40	**77.1**
No. 200	**64.2**
Description	**Clay**

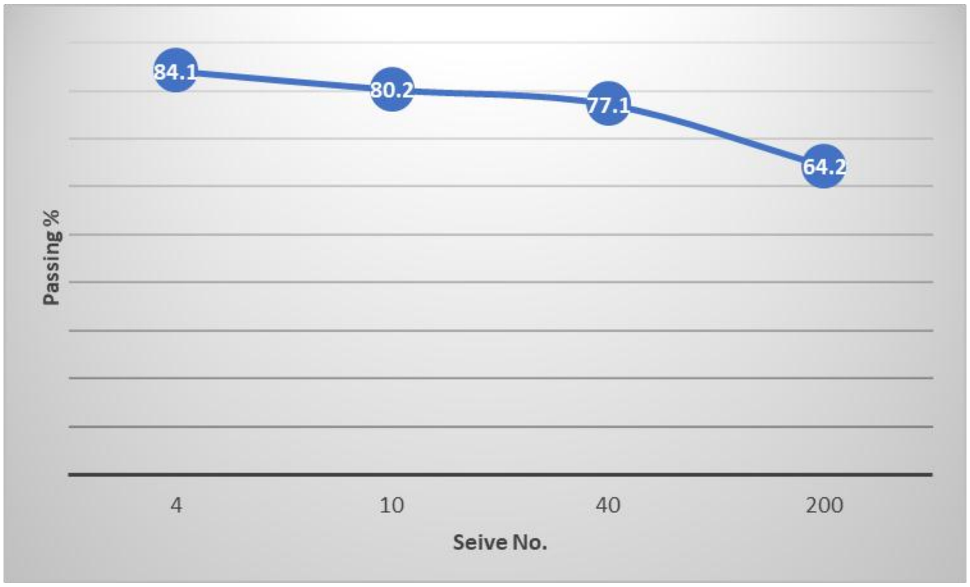


Figure 5.20.BH-1 Sieve Analysis at 80-m depth.

Table 5.21.BH-1 details at 85-m depth.

Borehole	**BH-01**
Depth (ft.)	**85**
Natural Moisture Content	**19.2%**
Sieve Number	**Passing Percentage**
No. 4	**97.1**
No. 10	**94.3**
No. 40	**90.2**
No. 200	**82.3**
Description	**Clay**

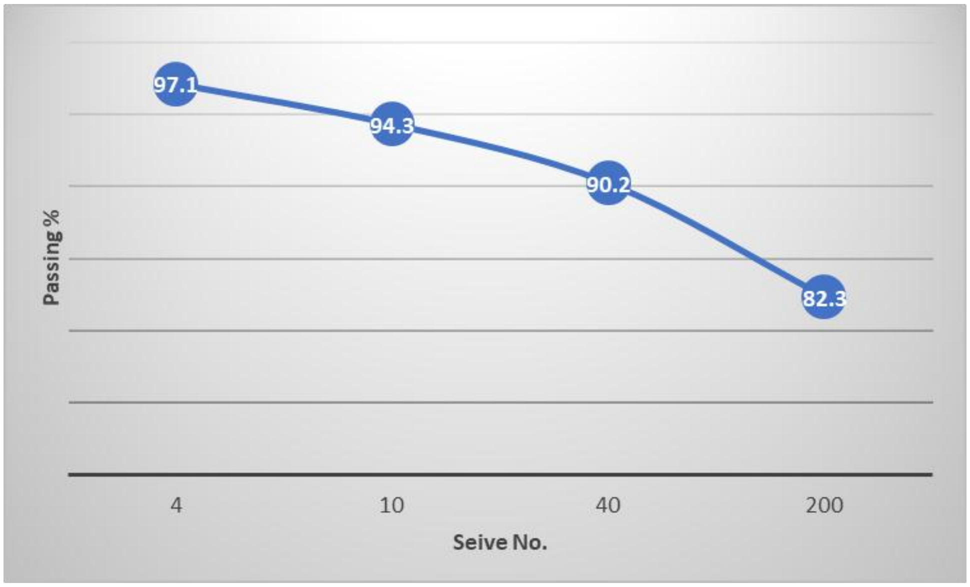


Figure 5.21.BH-1 Sieve Analysis at 85-m depth.

Table 5.22.BH-1 details at 90-m depth.

Borehole	**BH-01**
Depth (ft.)	**90**
Natural Moisture Content	**18.4%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**94.7**
No. 40	**89.1**
No. 200	**83.7**
Description	**Clay**

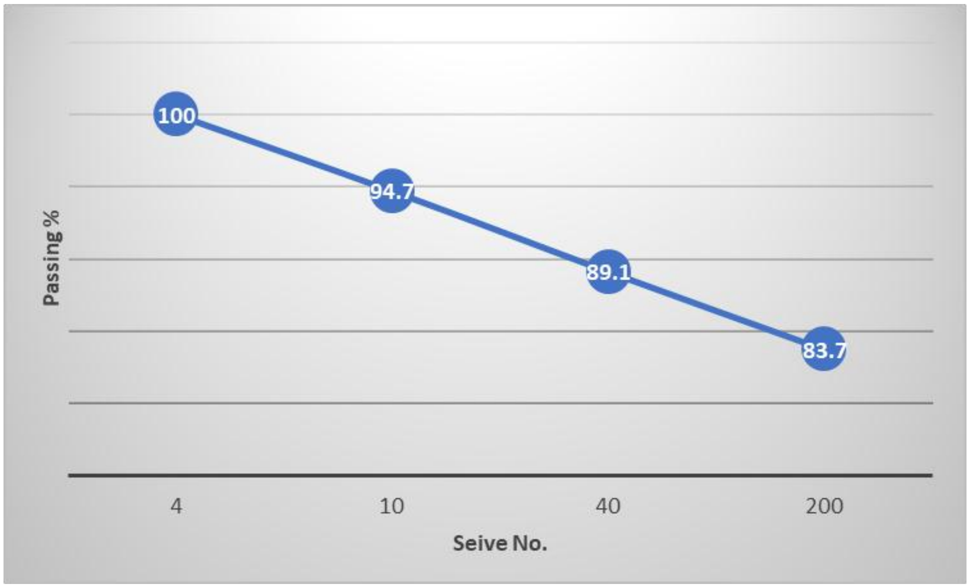


Figure 5.22.BH-1 Sieve Analysis at 90-m depth.

Table 5.23.BH-1 details at 95-m depth.

Borehole	**BH-01**
Depth (ft.)	**95**
Natural Moisture Content	**17.2%**
Sieve Number	**Passing Percentage**
No. 4	**84.1**
No. 10	**80.2**
No. 40	**77.4**
No. 200	**69.2**
Description	**Clay**

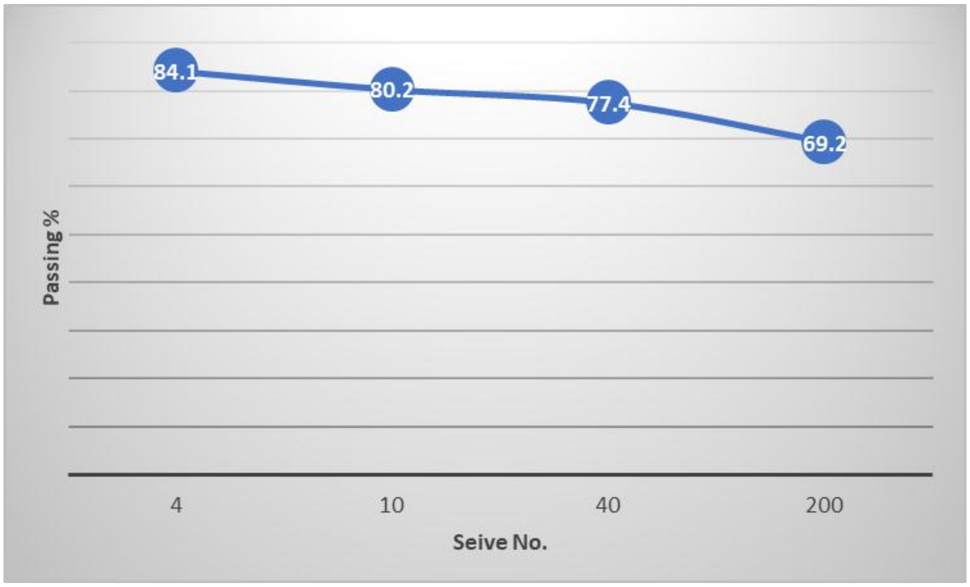


Figure 5.23.BH-1 Sieve Analysis at 95-m depth.

Table 5.24.BH-1 details at 100-m depth.

Borehole	**BH-01**
Depth (ft.)	**100**
Natural Moisture Content	**18.4%**
Sieve Number	**Passing Percentage**
No. 4	**94.7**
No. 10	**90.2**
No. 40	**87.4**
No. 200	**79.4**
Description	**Clay**

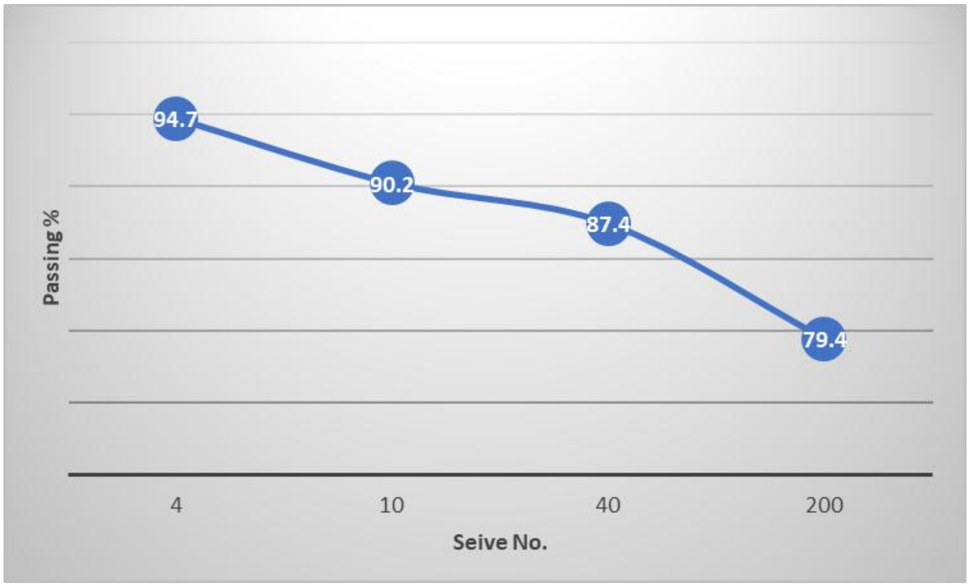


Figure 5.24.BH-1 Sieve Analysis at 100-m depth.

Seive passing percentage for BH-2

Table 5. 25.BH-2 details at 3-m depth.

Borehole	**BH-02**
Depth (ft.)	**3**
Natural Moisture Content	**30.3%**
Sieve Number	**Passing Percentage**
No. 4	**96.4**
No. 10	**94.6**
No. 40	**88.4**
No. 200	**76.2**
Description	**Clay**

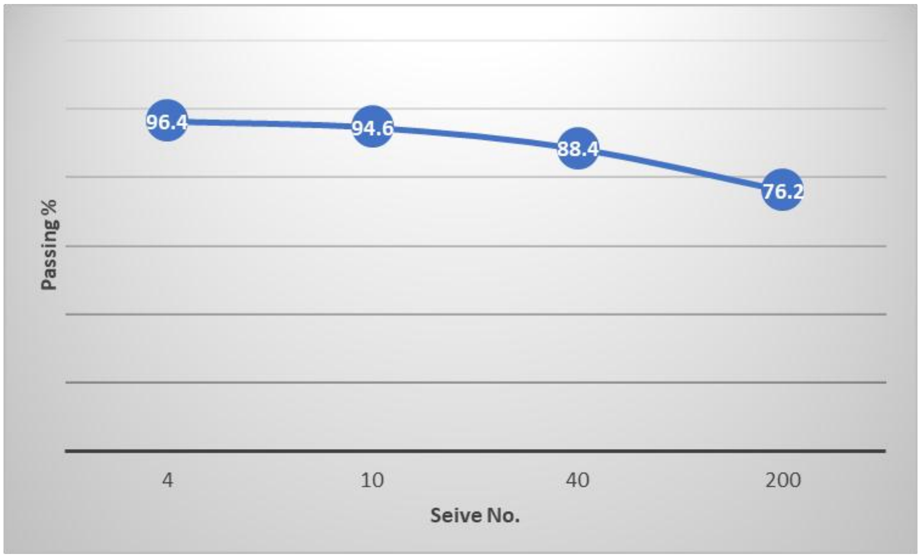


Figure 5.25.BH-2 Sieve Analysis at 3-m depth.

Table 5.26.BH-2 details at 6-m depth.

Borehole	**BH-02**
Depth (ft.)	**6**
Natural Moisture Content	**26.9%**
Sieve Number	**Passing Percentage**
No. 4	**94.3**
No. 10	**90.2**
No. 40	**84.7**
No. 200	**79.4**
Description	**Clay**

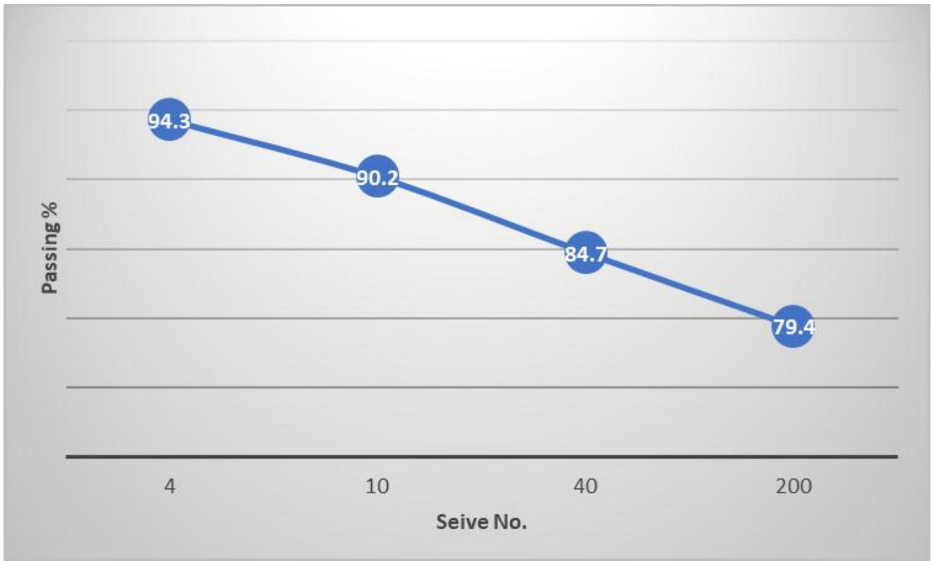


Figure 5.26.BH-2 Sieve Analysis at 6-m depth.

Table 5.27.BH-2 details at 9-m depth.

Borehole	**BH-02**
Depth (ft.)	**9**
Natural Moisture Content	**28.1%**
Sieve Number	**Passing Percentage**
No. 4	**98.1**
No. 10	**96.1**
No. 40	**94.3**
No. 200	**87.4**
Description	**Clay**

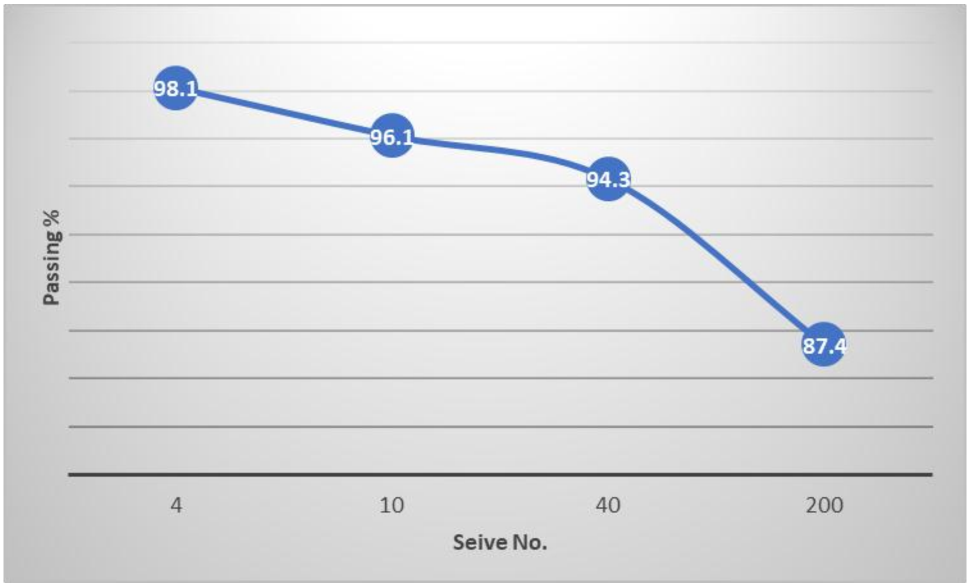


Figure 5.27.BH-2 Sieve Analysis at 9-m depth.

Table 5.28.BH-2 details at 12-m depth.

Borehole	**BH-01**
Depth (ft.)	**12**
Natural Moisture Content	**26.7%**
Sieve Number	**Passing Percentage**
No. 4	**97.1**
No. 10	**95.1**
No. 40	**93.3**
No. 200	**89.7**
Description	**Clay**

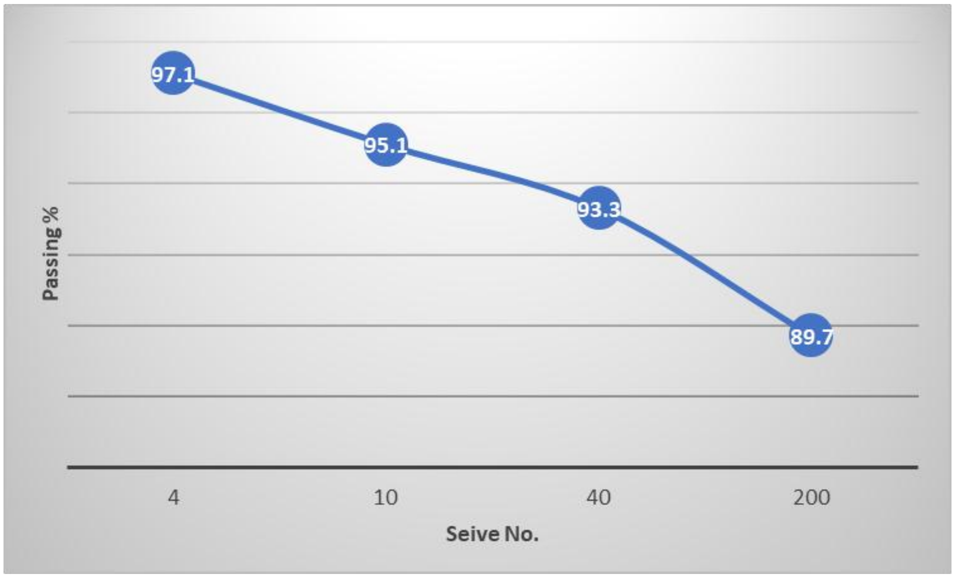


Figure 5.28.BH-2 Sieve Analysis at 12-m depth.

Table 5.29.BH-2 details at 15-m depth.

Borehole	**BH-02**
Depth (ft.)	**15**
Natural Moisture Content	**28.5%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**99.5**
No. 40	**98.5**
No. 200	**96.1**
Description	**Clay**

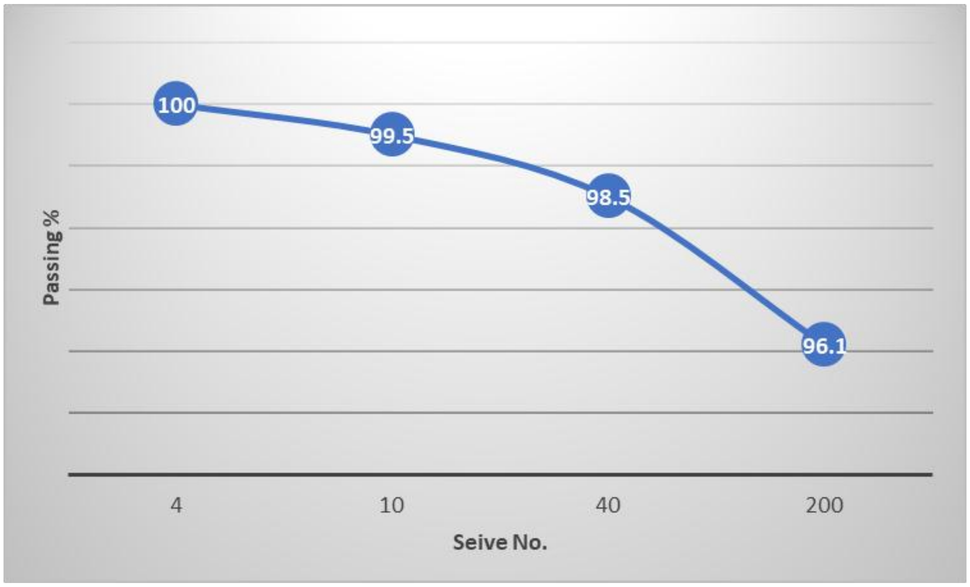


Figure 5.29.BH-2 Sieve Analysis at 15-m depth.

Table 5.30.BH-2 details at 18-m depth.

Borehole	**BH-02**
Depth (ft.)	**18**
Natural Moisture Content	**26.4%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.1**
No. 40	**96.3**
No. 200	**90.2**
Description	**Clay**

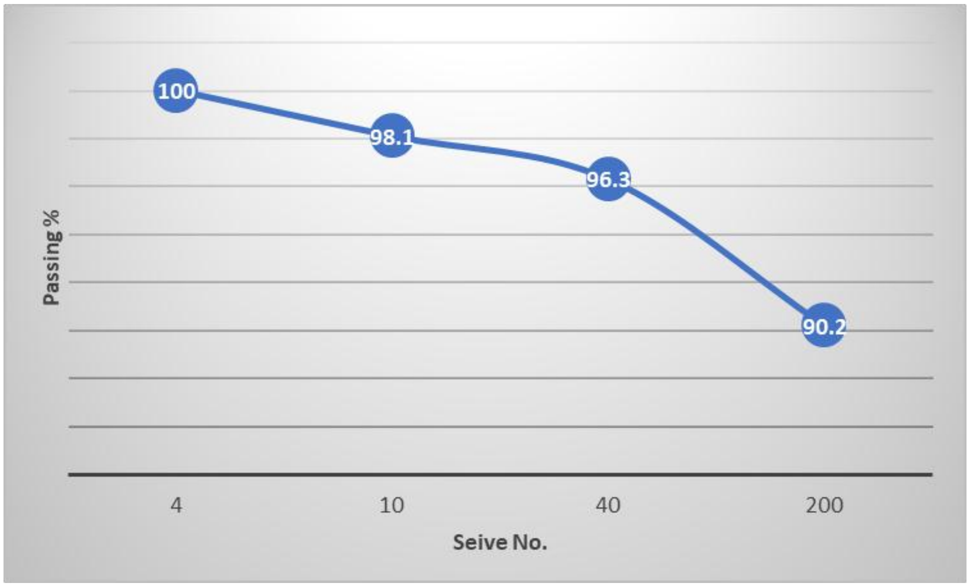


Figure 5.30.BH-2 Sieve Analysis at 18-m depth.

Table 5.31.BH-2 details at 21-m depth.

Borehole	**BH-02**
Depth (ft.)	**21**
Natural Moisture Content	**24.7%**
Sieve Number	**Passing Percentage**
No. 4	**98.1**
No. 10	**96.1**
No. 40	**94.7**
No. 200	**87.4**
Description	**Clay**

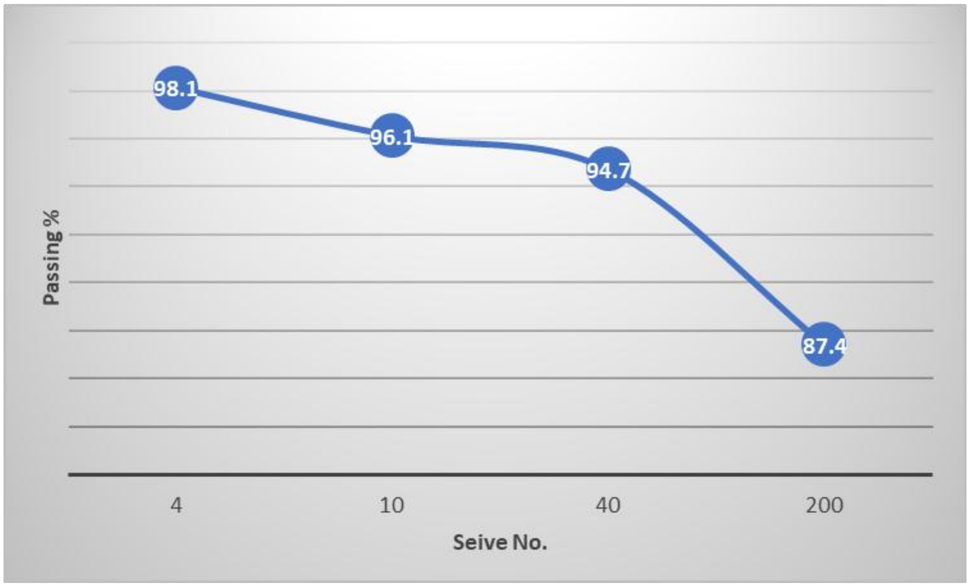


Figure 5.31.BH-2 Sieve Analysis at 21-m depth.

Table 5.32.BH-2 details at 24-m depth.

Borehole	**BH-02**
Depth (ft.)	**24**
Natural Moisture Content	**23.1%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.1**
No. 40	**96.3**
No. 200	**91.7**
Description	**Clay**

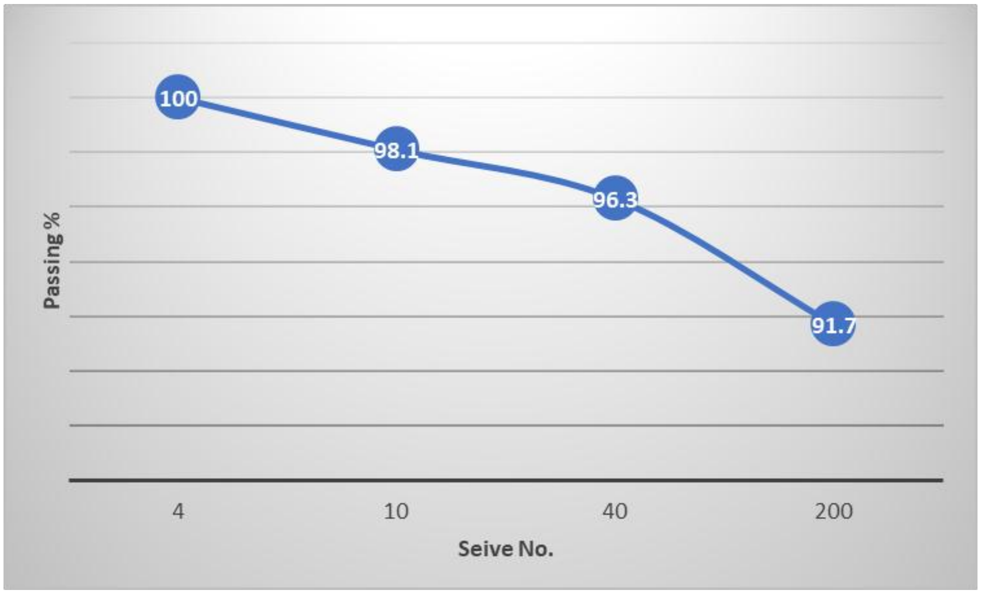


Figure 5.32.BH-2 Sieve Analysis at 24-m depth.

Table 5.33.BH-2 details at 27-m depth.

Borehole	**BH-02**
Depth (ft.)	**27**
Natural Moisture Content	**26.3%**
Sieve Number	**Passing Percentage**
No. 4	**99.1**
No. 10	**97.1**
No. 40	**95.1**
No. 200	**89.2**
Description	**Clay**

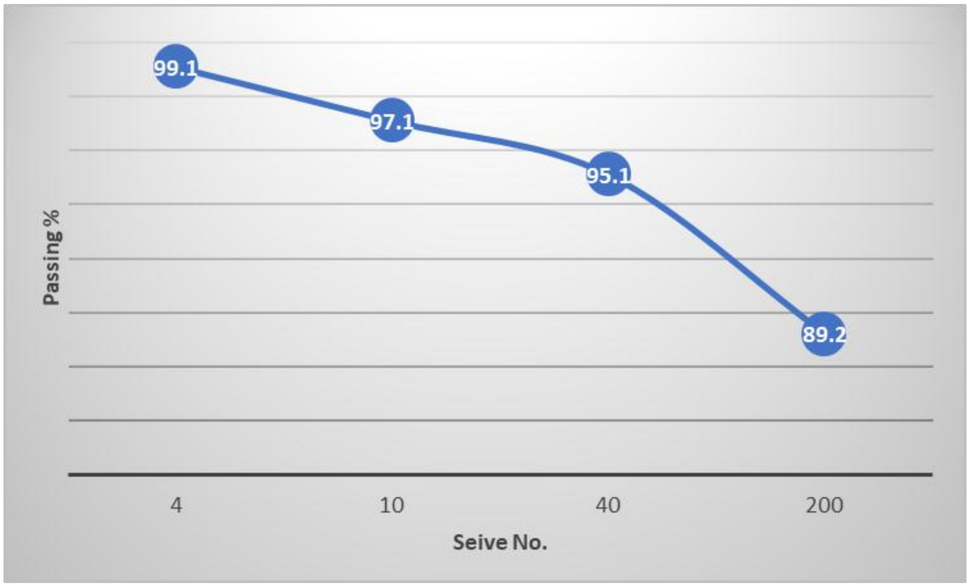


Figure 5.33.BH-2 Sieve Analysis at 27-m depth.

Table 5.34.BH-2 details at 30-m depth.

Borehole	**BH-02**
Depth (ft.)	**30**
Natural Moisture Content	**27.1%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.1**
No. 40	**96.1**
No. 200	**92.3**
Description	**Clay**

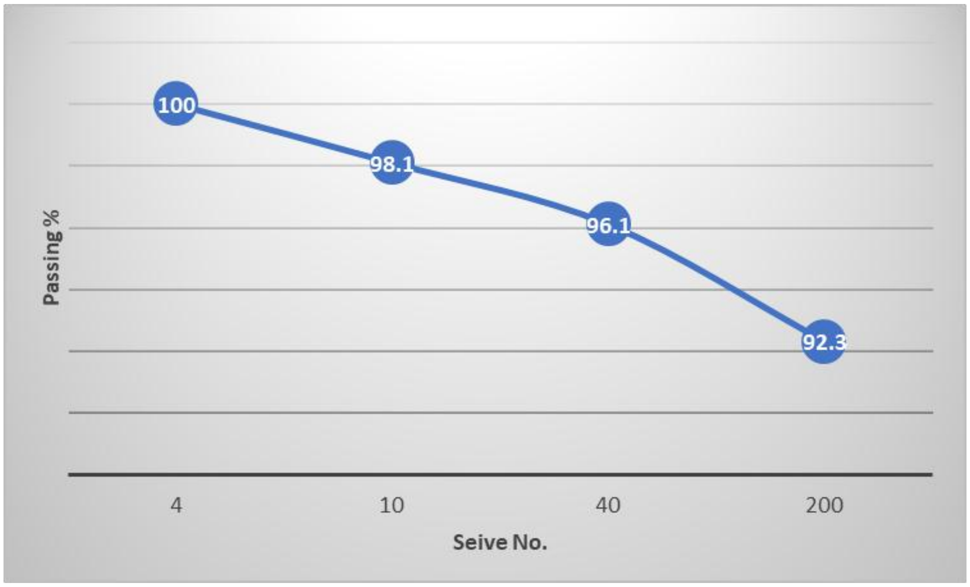


Figure 5.34.BH-2 Sieve Analysis at 30-m depth.

Table 5.35.BH-2 details at 35-m depth.

Borehole	**BH-02**
Depth (ft.)	**35**
Natural Moisture Content	**28.6%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.7**
No. 40	**97.4**
No. 200	**95.1**
Description	**Clay**

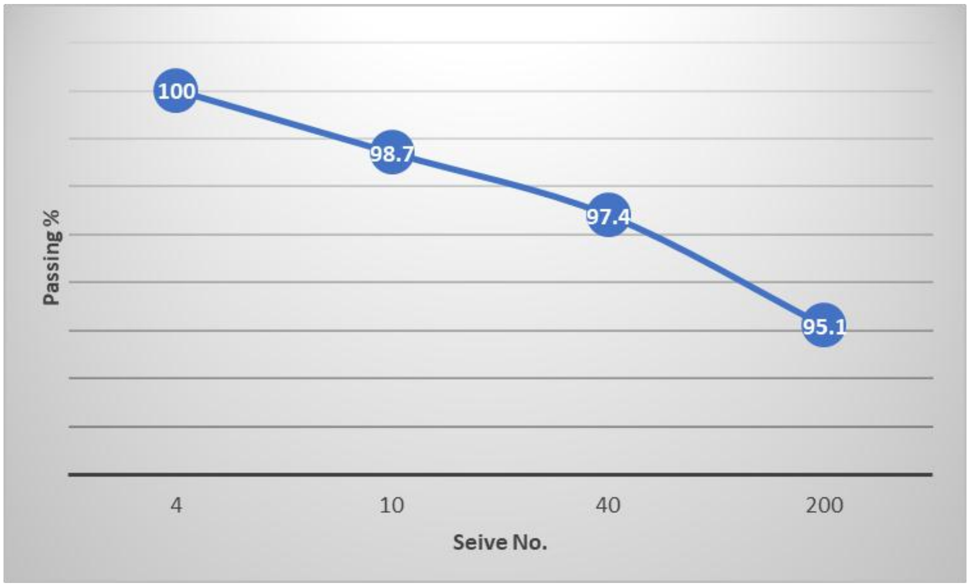


Figure 5.35.BH-2 Sieve Analysis at 35-m depth.

Table 5.36.BH-2 details at 40-m depth.

Borehole	**BH-02**
Depth (ft.)	**40**
Natural Moisture Content	**26.1%**
Sieve Number	**Passing Percentage**
No. 4	**97.1**
No. 10	**94.2**
No. 40	**90.2**
No. 200	**88.4**
Description	**Clay**

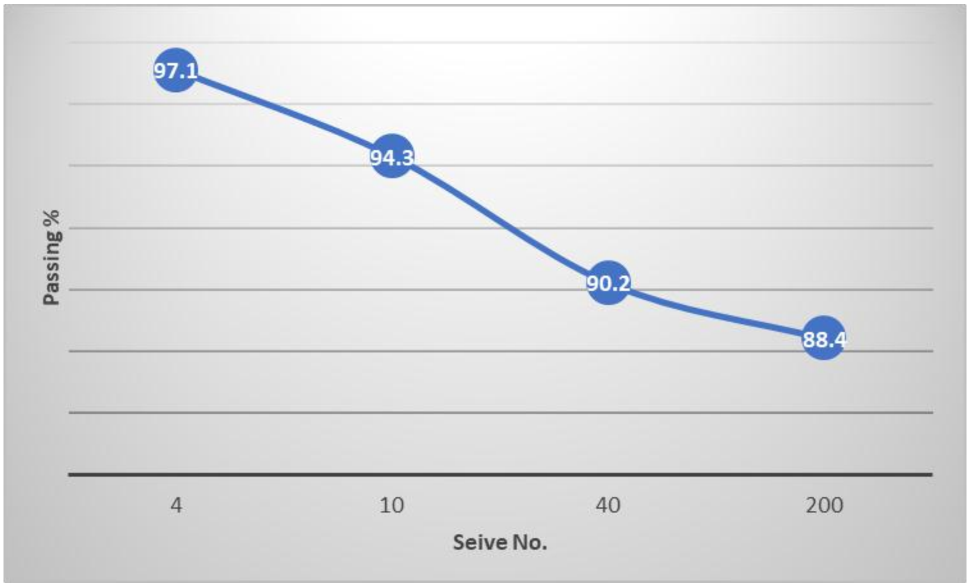


Figure 5.36.BH-2 Sieve Analysis at 40-m depth.

Table 5.37.BH-2 details at 45-m depth.

Borehole	**BH-02**
Depth (ft.)	**45**
Natural Moisture Content	**24.2%**
Sieve Number	**Passing Percentage**
No. 4	**98.7**
No. 10	**97.5**
No. 40	**95.1**
No. 200	**92.2**
Description	**Clay**

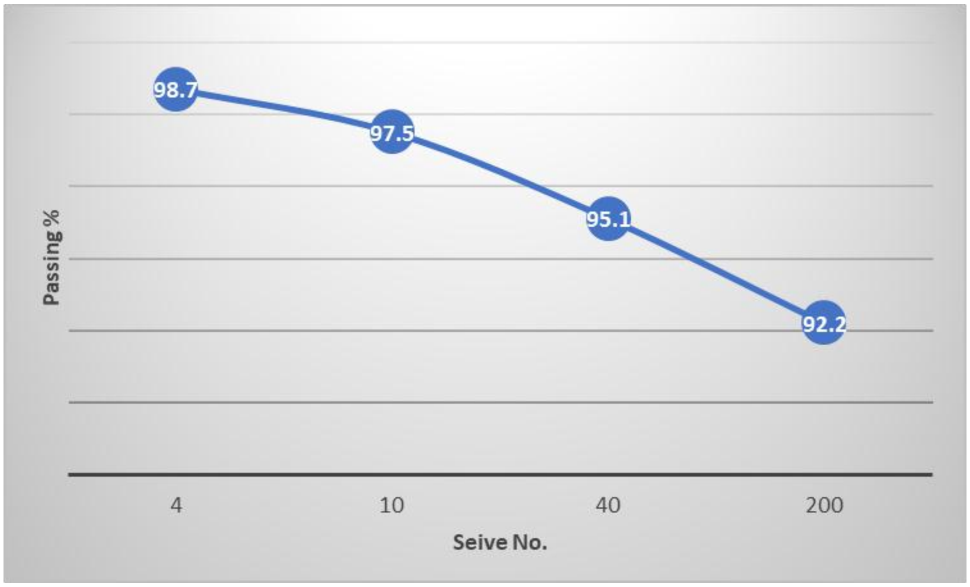


Figure 5.37.BH-2 Sieve Analysis at 45-m depth.

Table 5.38.BH-2 details at 50-m depth.

Borehole	**BH-02**
Depth (ft.)	**50**
Natural Moisture Content	**26.1%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.1**
No. 40	**94.3**
No. 200	**90.4**
Description	**Clay**

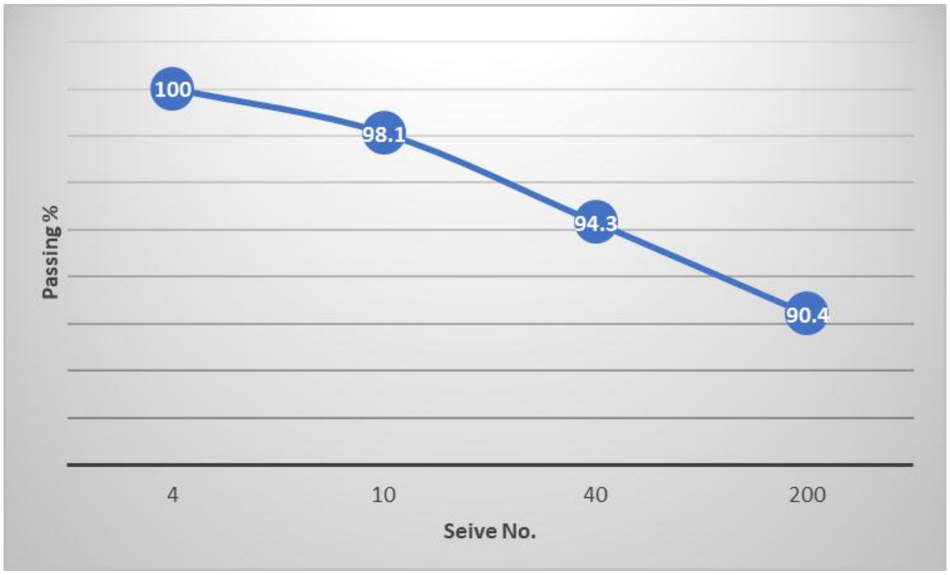


Figure 5.38.BH-2 Sieve Analysis at 50-m depth.

Table 5.39.BH-2 details at 55-m depth.

Borehole	**BH-02**
Depth (ft.)	**55**
Natural Moisture Content	**23.1%**
Sieve Number	**Passing Percentage**
No. 4	**98.1**
No. 10	**96.3**
No. 40	**94.2**
No. 200	**89.4**
Description	**Clay**

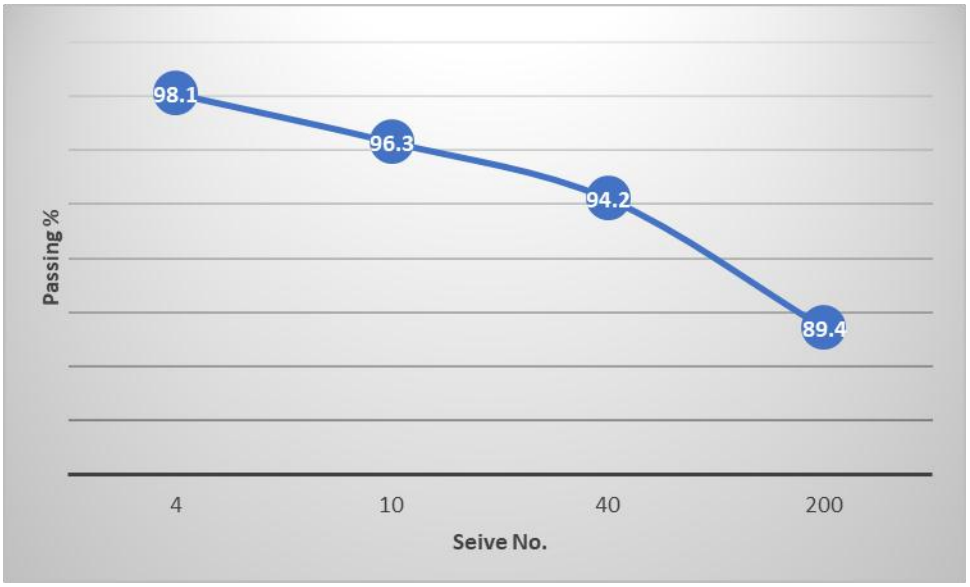


Figure 5.39.BH-2 Sieve Analysis at 55-m depth.

Table 5.40.BH-2 details at 60-m depth.

Borehole	**BH-02**
Depth (ft.)	**60**
Natural Moisture Content	**27.6%**
Sieve Number	**Passing Percentage**
No. 4	**95.6**
No. 10	**92.4**
No. 40	**90.3**
No. 200	**88.1**
Description	**Clay**

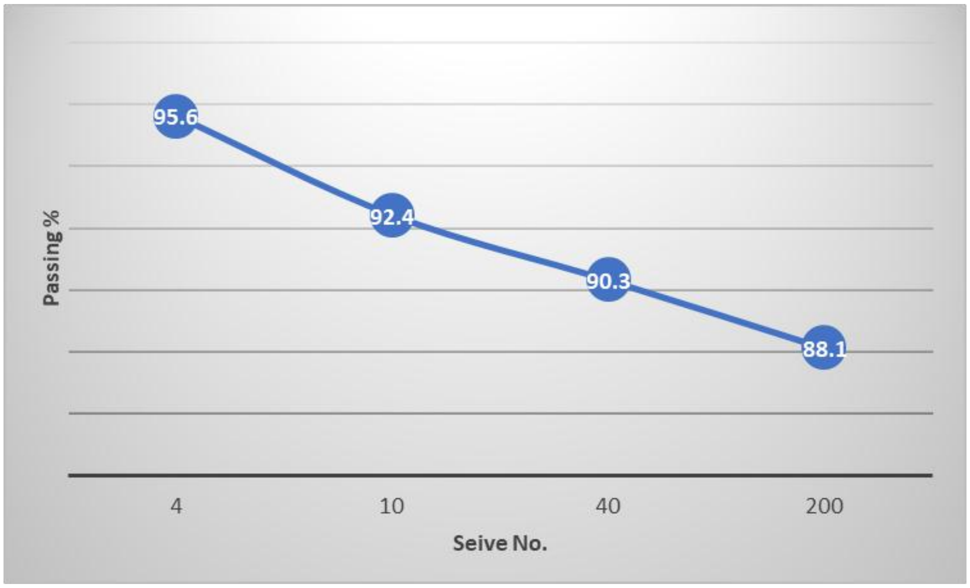


Figure 5.40.BH-2 Sieve Analysis at 60-m depth.

Table 5.41.BH-2 details at 65-m depth.

Borehole	**BH-02**
Depth (ft.)	**65**
Natural Moisture Content	**21.9%**
Sieve Number	**Passing Percentage**
No. 4	**84.1**
No. 10	**79.8**
No. 40	**75.6**
No. 200	**69.5**
Description	**Clay**

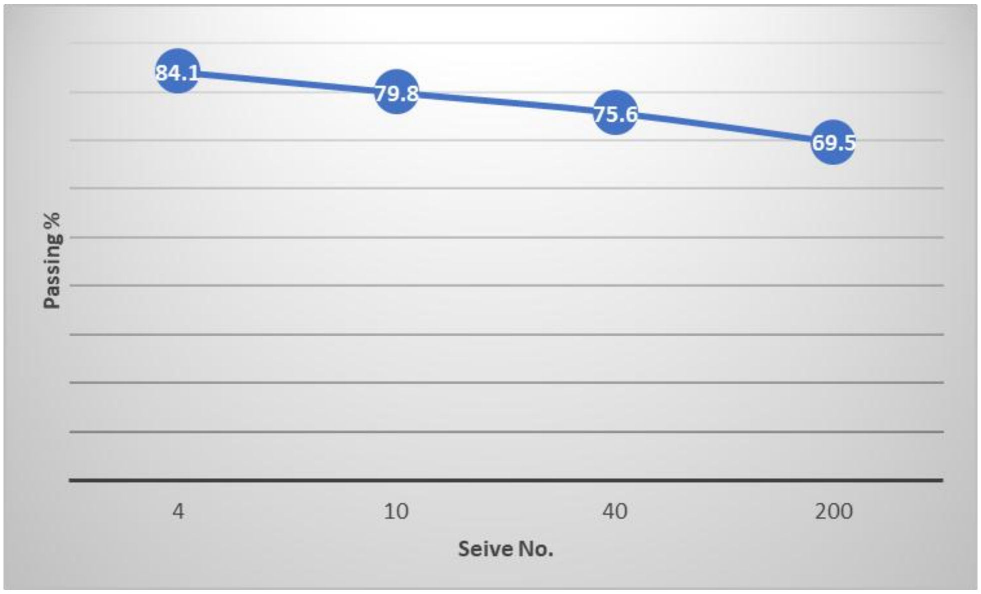


Figure 5.41.BH-2 Sieve Analysis at 65-m depth.

Table 5.42.BH-2 details at 70-m depth.

Borehole	**BH-02**
Depth (ft.)	**70**
Natural Moisture Content	**27.3%**
Sieve Number	**Passing Percentage**
No. 4	**90.1**
No. 10	**81.3**
No. 40	**84.2**
No. 200	**74.3**
Description	**Clay**

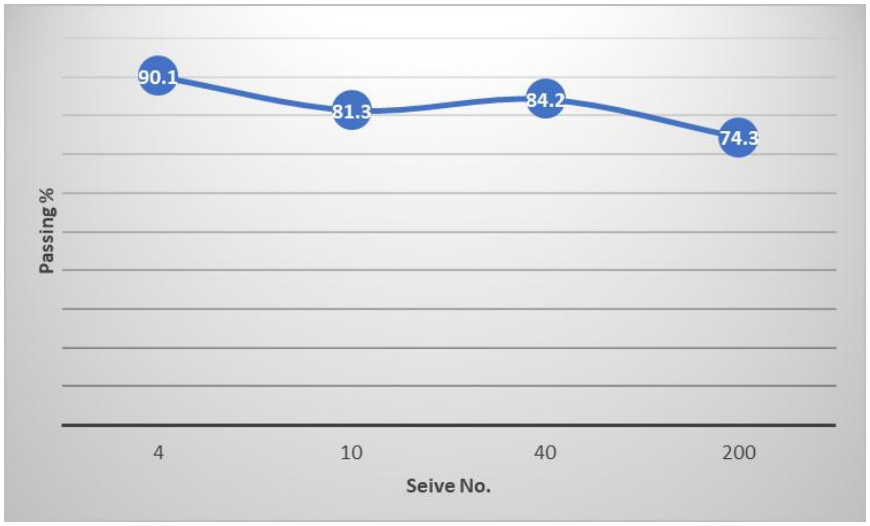


Figure 5.42.BH-2 Sieve Analysis at 70-m depth.

Table 5.43.BH-2 details at 75-m depth.

Borehole	**BH-02**
Depth (ft.)	**75**
Natural Moisture Content	**24.9%**
Sieve Number	**Passing Percentage**
No. 4	**96.1**
No. 10	**90.2**
No. 40	**88.1**
No. 200	**84.3**
Description	**Clay**

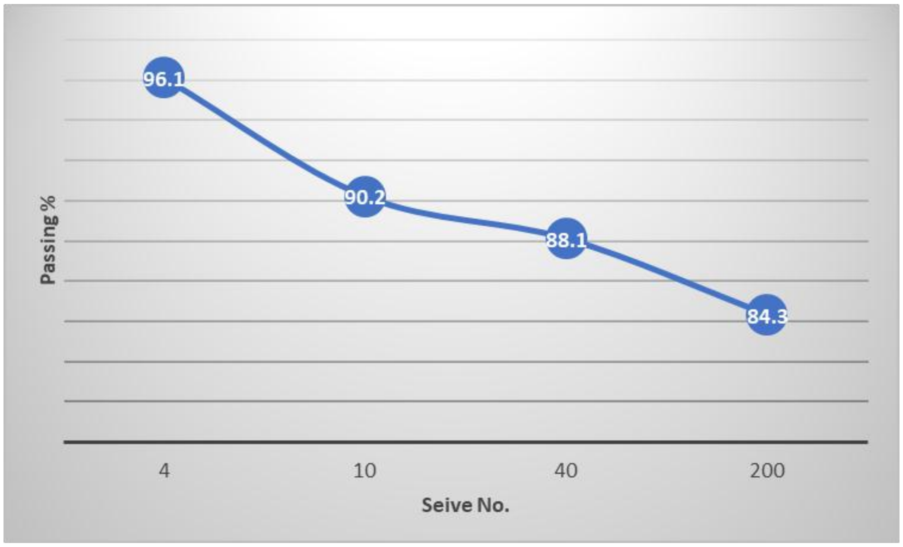


Figure 5.43.BH-2 Sieve Analysis at 75-m depth.

Table 5.44.BH-2 details at 80-m depth.

Borehole	**BH-02**
Depth (ft.)	**80**
Natural Moisture Content	**24.1%**
Sieve Number	**Passing Percentage**
No. 4	**98.1**
No. 10	**95.6**
No. 40	**93.8**
No. 200	**91.9**
Description	**Clay**

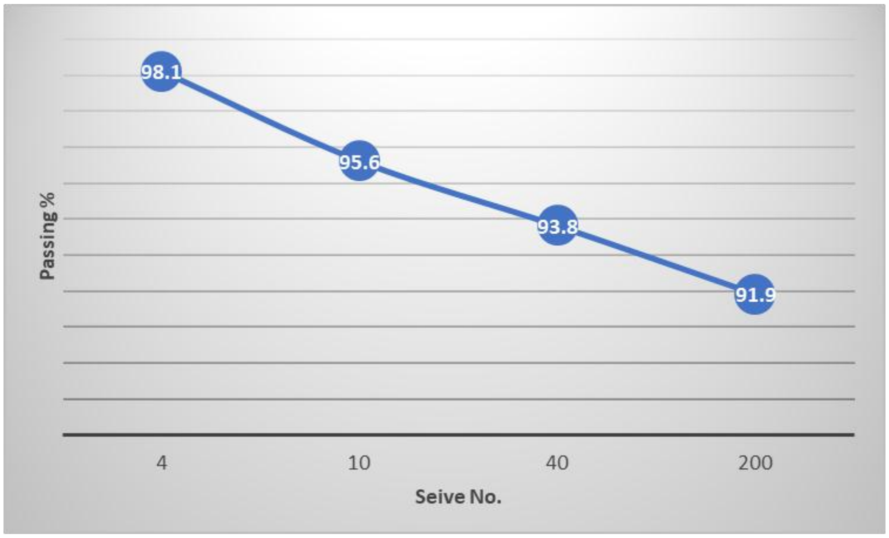


Figure 5.44.BH-2 Sieve Analysis at 80-m depth.

Table 5.45.BH-2 details at 85-m depth.

Borehole	**BH-02**
Depth (ft.)	**85**
Natural Moisture Content	**24.9%**
Sieve Number	**Passing Percentage**
No. 4	**100**
No. 10	**98.1**
No. 40	**96.3**
No. 200	**89.1**
Description	**Clay**

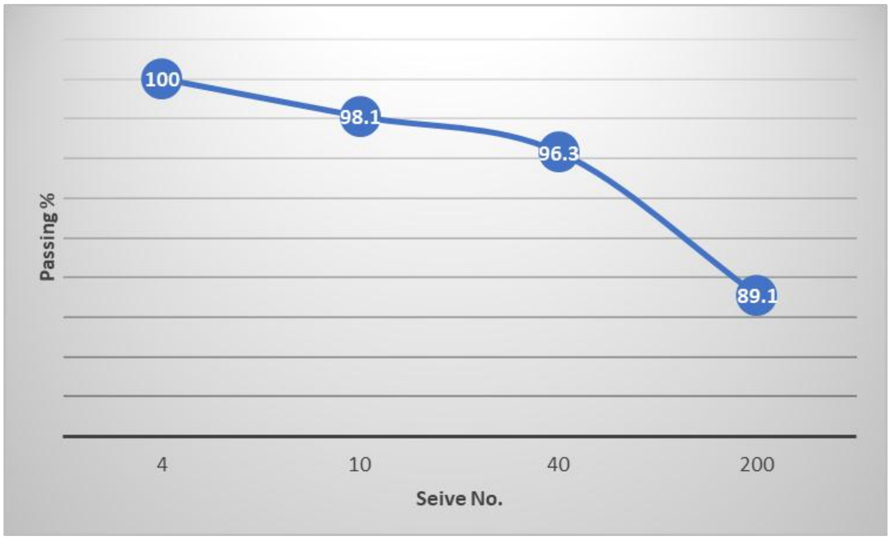


Figure 5.45.BH-2 Sieve Analysis at 85-m depth.

Table 5.46.BH-2 details at 90-m depth.

Borehole	**BH-02**
Depth (ft.)	**90**
Natural Moisture Content	**26.7%**
Sieve Number	**Passing Percentage**
No. 4	**94.1**
No. 10	**90.2**
No. 40	**87.1**
No. 200	**85.1**
Description	**Clay**

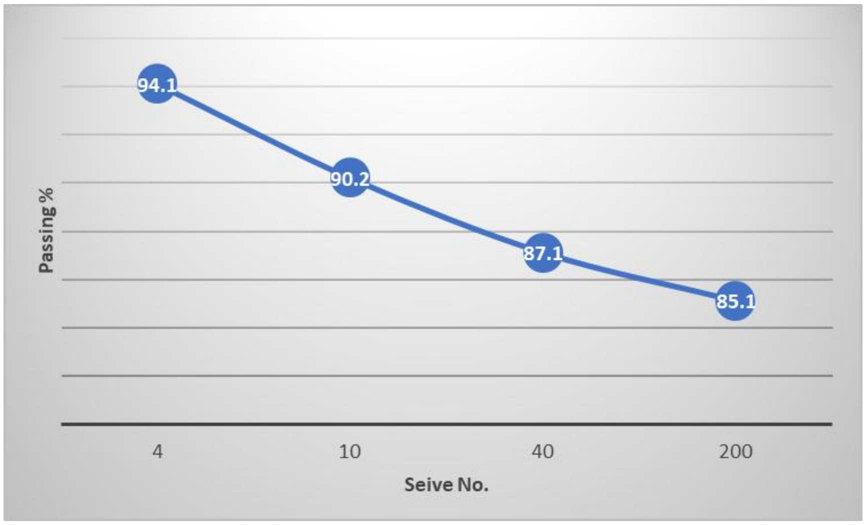


Figure 5.46.BH-2 Sieve Analysis at 90-m depth.

Table 5.47.BH-2 details at 95-m depth.

Borehole	**BH-02**
Depth (ft.)	**95**
Natural Moisture Content	**27.4%**
Sieve Number	**Passing Percentage**
No. 4	**95.8**
No. 10	**94.5**
No. 40	**91.1**
No. 200	**83.9**
Description	**Clay**

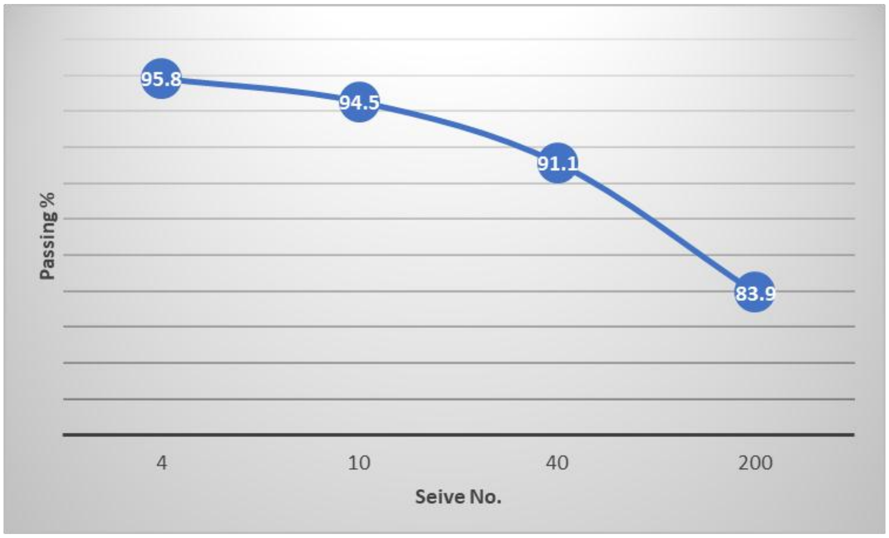


Figure 5.47.BH-2 Sieve Analysis at 95-m depth.

Table 5.48.BH-2 details at 100-m depth.

Borehole	**BH-02**
Depth (ft.)	**100**
Natural Moisture Content	**28.4%**
Sieve Number	**Passing Percentage**
No. 4	**93.1**
No. 10	**90.2**
No. 40	**87.1**
No. 200	**89.4**
Description	**Clay**

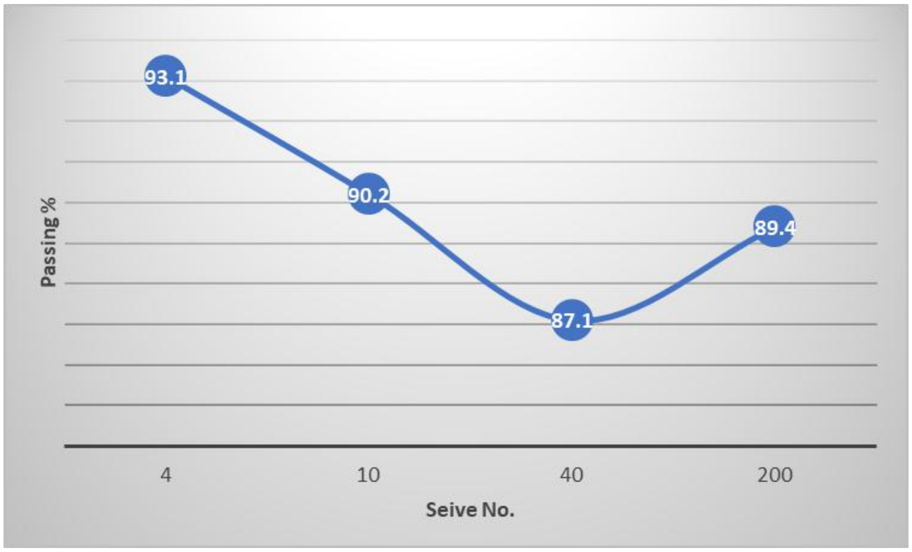


Figure 5.48.BH-2 Sieve Analysis at 100-m depth.

ATTERBURG LIMITS

Table 5.49. BH-1 details

Project	Multistory Building
Location	Islamabad
Borehole	BH-01
Depth (ft)	100
Classification group	Lean Clay

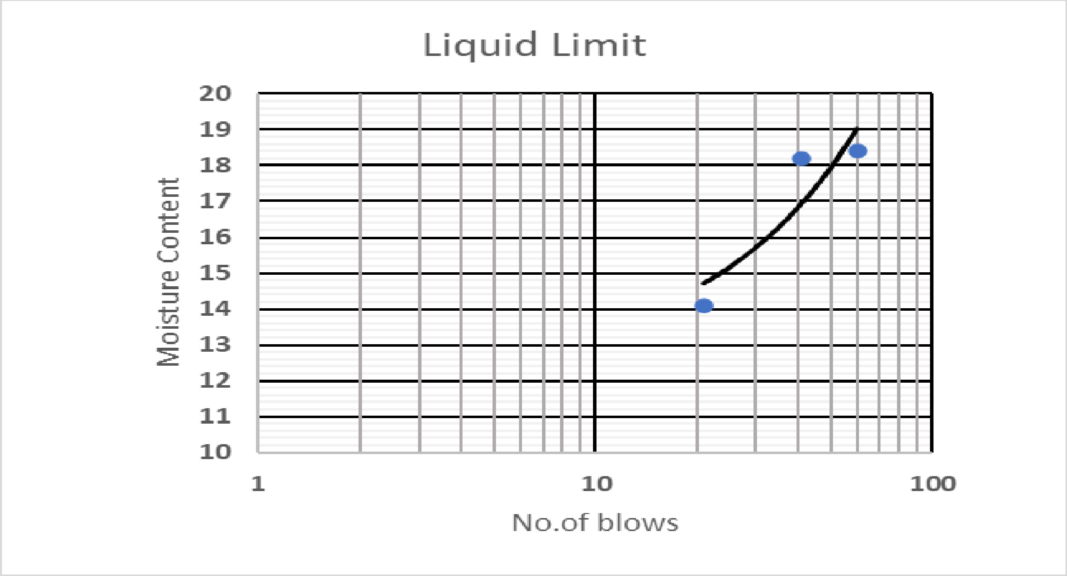


Figure 5.49 Liquid Limit graph for BH-1.

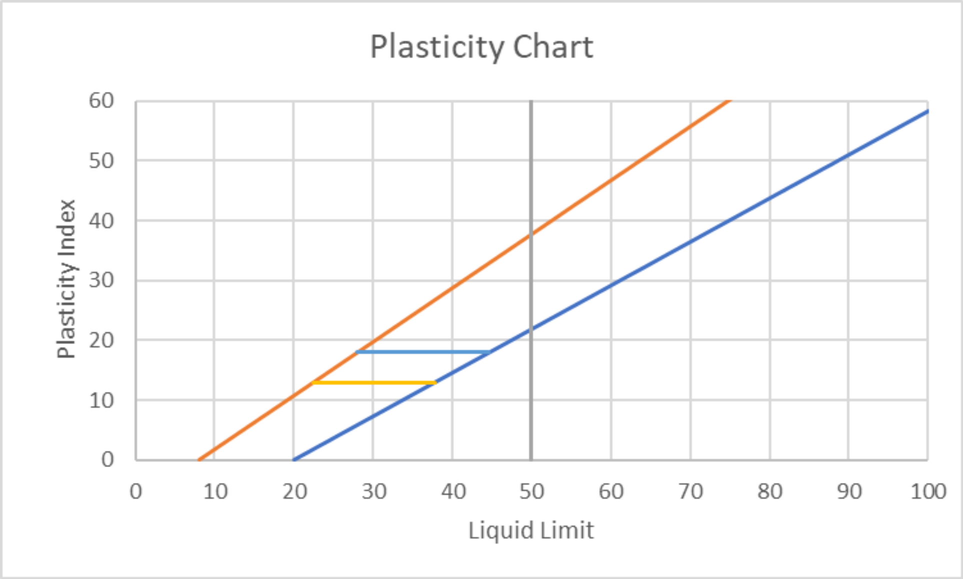


Figure 5.50. Plasticity graph for BH-1.

Table 5.50.BH-2 details.

Project	Multistory Building
Location	Islamabad
Borehole	BH-02
Depth (ft)	100
Classification group	Lean Clay

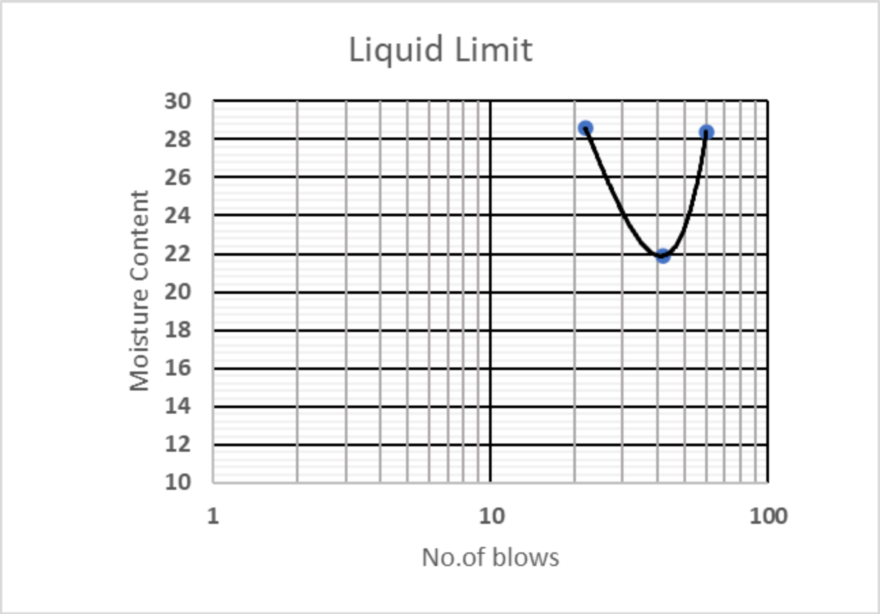


Figure 5.51. Liquid limit graph for BH-2.

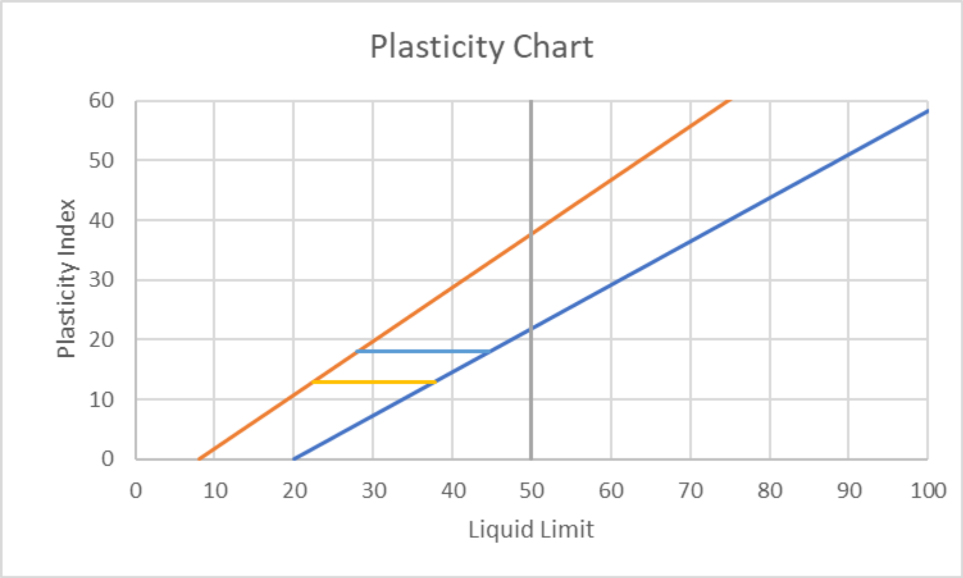


Figure 5.52. Plasticity graph for BH-2.

For an overview, the summary of lab results and the borehole logs are given below:

BH-01

Table 5.51. Summary of Laboratory results.

Depth (ft.)	Atterburg Limits			NMC (%)	Sieve Analysis			
	LL	PL	PI		No.4	No.10	No.40	No.200
3	33	18	15	14.3	94.1	92.1	89.1	86.3
6	32	18	14	3.9	100	98.4	96.1	90.2
9	33	18	15	15.2	100	98.7	97.1	94.2
12	32	18	14	15.2	98.1	96.3	94.1	89.3
15	32	18	14	14.9	100	96.1	94.7	88.1
18	33	18	15	15.2	100	98.1	96.1	92.7
21	32	18	14	16.7	98.1	96.3	92.1	87.4
24	32	18	14	16.1	97.1	95.1	91.3	86.2
27	33	18	15	15.7	100	98.8	97.6	95.2
30	33	18	15	16.1	100	97.1	95.2	92.3
35	33	18	15	14.1	100	98.7	97.5	94.3
40	33	18	15	18.1	98.1	96.3	94.1	90.3
45	33	18	15	19.1	97.1	96.1	94.1	92.3
50	33	18	15	18.4	100	97.1	96.3	94.7
55	33	18	15	18.8	100	98.2	96.1	94.7
60	33	18	15	19.1	94.3	90.2	89.1	81.3
65	33	17	14	18.2	91.7	87.8	83.4	79.1
70	31	18	14	17.4	89.1	84.3	76.1	70.3
75	32	17	14	16.8	79.8	74.4	70.2	68.2
80	31	18	15	18.4	84.1	80.2	77.1	64.2
85	33	17	13	19.2	97.1	94.3	90.2	82.3
90	30	18	14	18.4	100	94.7	89.1	83.7
95	31	17	14	17.2	84.1	80.2	77.4	69.2
100	32	18	14	17.2	94.7	90.2	87.4	79.4

BH-02

Table 5.52. Summary of Laboratory results.

Depth (ft.)	Atterburg Limits			NMC (%)	Sieve Analysis			
	LL	PL	PI		No.4	No.10	No.40	No.200
3	30	17	13	30.3	96.4	94.6	88.4	76.2
6	32	18	14	26.9	94.3	90.2	84.7	79.4
9	31	17	14	28.1	98.1	96.1	94.3	87.4
12	33	18	15	26.7	97.1	95.1	93.3	89.7
15	33	18	15	28.5	100	99.5	98.5	96.1
18	32	18	14	26.4	100	98.1	96.3	90.2
21	32	18	14	24.7	98.1	96.1	94.7	87.4
24	33	18	15	23.1	100	98.1	96.3	88.6
27	33	18	15	26.3	99.1	97.1	95.1	89.7
30	32	18	14	27.1	100	98.1	96.1	89.2
35	33	18	15	28.6	100	98.7	97.4	92.3
40	33	18	15	26.1	97.1	94.3	90.2	95.1
45	33	18	15	24.2	98.7	97.5	95.1	88.4
50	33	18	15	26.1	100	98.1	94.3	92.2
55	33	18	15	23.1	98.1	96.3	94.2	90.4
60	33	18	15	27.6	95.6	92.4	90.3	89.4
65	33	18	15	21.9	84.1	79.8	75.6	88.1
70	31	17	14	27.3	90.1	81.3	84.2	69.5
75	30	17	13	24.9	96.1	90.2	88.1	74.3
80	33	18	15	24.1	98.1	95.6	93.8	84.3
85	33	18	15	24.9	100	98.1	96.3	90.9
90	31	17	14	26.7	94.1	90.2	87.1	89.1
95	33	18	15	27.4	95.8	94.5	91.1	85.1
100	32	17	15	28.4	93.1	90.2	87.1	89.4

Bearing Capacity Analysis:

Meyerhof (1976) also proposed that bearing capacity can be acquired by SPT by using field observations. SPT method is used to calculate the ultimate bearing capacity of piles by the following formula given:

Bearing Capacity formula:

$Q_x = N/F_1 \times kd$ (Less than 4ft)

$Q_{q=} N_{55}/F_2(B+F_3/B)^2$ kd (Greater than 4ft)

$N_{60} = E_m C_B C_S C_R C N/0.60$

$N_{60} = 0.60x1\ x1x\ 0.73\ xN/0.60$

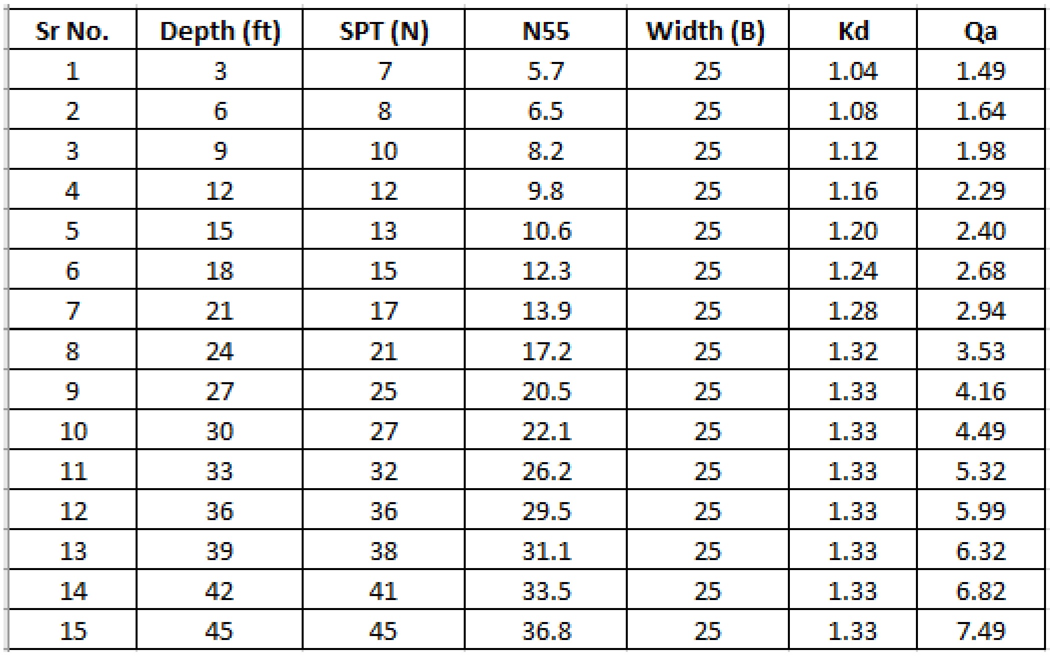
$N_{55} = 60/55x(N_{60})$

Qa = Allowable Bearing Capacity

N60 = Corrected N-value

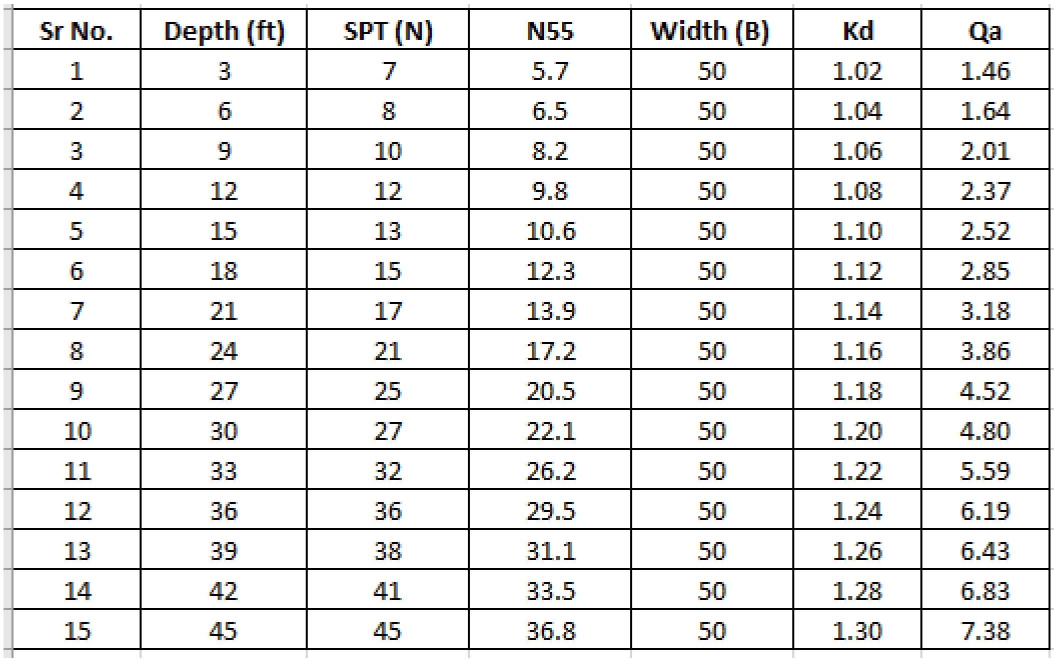
N55 = Corrected N-value

Borehole 1 at width 25:



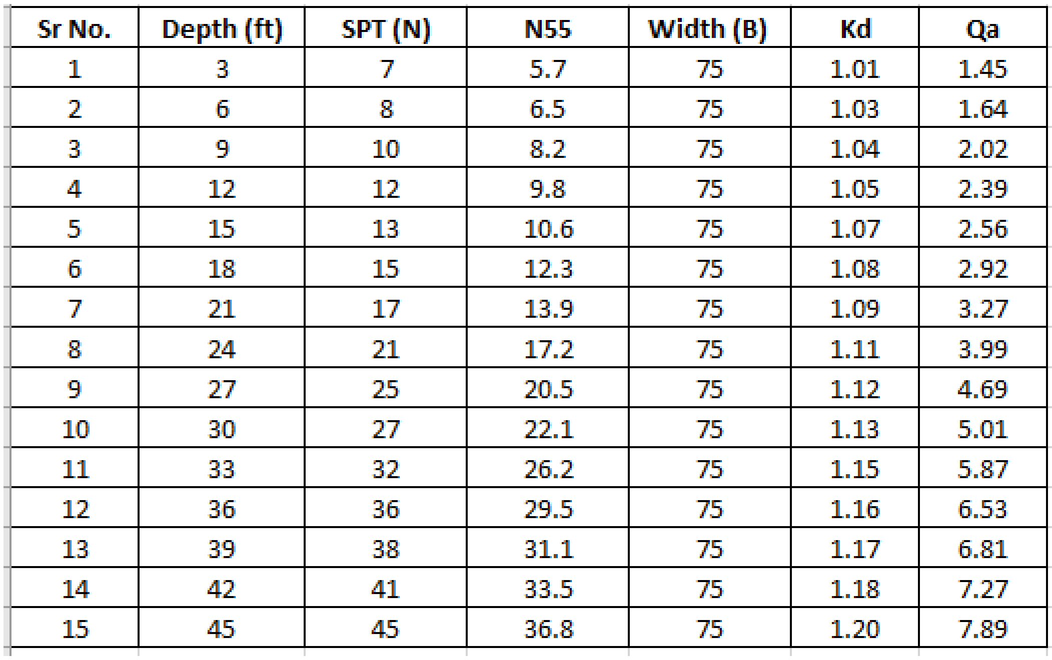
Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	7	5.7	25	1.04	1.49
2	6	8	6.5	25	1.08	1.64
3	9	10	8.2	25	1.12	1.98
4	12	12	9.8	25	1.16	2.29
5	15	13	10.6	25	1.20	2.40
6	18	15	12.3	25	1.24	2.68
7	21	17	13.9	25	1.28	2.94
8	24	21	17.2	25	1.32	3.53
9	27	25	20.5	25	1.33	4.16
10	30	27	22.1	25	1.33	4.49
11	33	32	26.2	25	1.33	5.32
12	36	36	29.5	25	1.33	5.99
13	39	38	31.1	25	1.33	6.32
14	42	41	33.5	25	1.33	6.82
15	45	45	36.8	25	1.33	7.49

Borehole 1 at width 50:



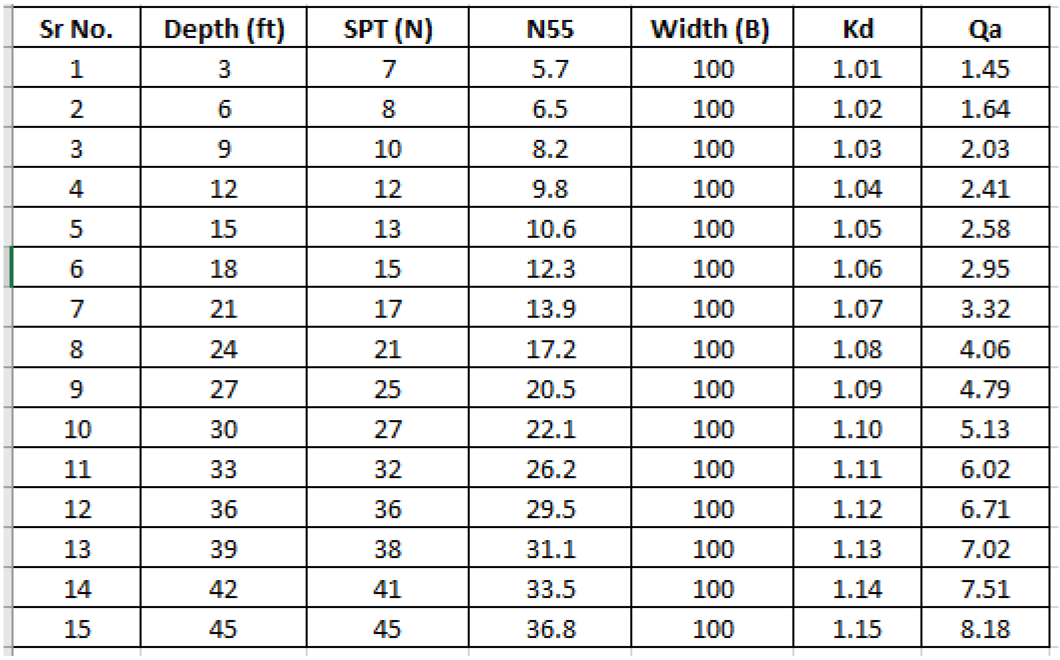
Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	7	5.7	50	1.02	1.46
2	6	8	6.5	50	1.04	1.64
3	9	10	8.2	50	1.06	2.01
4	12	12	9.8	50	1.08	2.37
5	15	13	10.6	50	1.10	2.52
6	18	15	12.3	50	1.12	2.85
7	21	17	13.9	50	1.14	3.18
8	24	21	17.2	50	1.16	3.86
9	27	25	20.5	50	1.18	4.52
10	30	27	22.1	50	1.20	4.80
11	33	32	26.2	50	1.22	5.59
12	36	36	29.5	50	1.24	6.19
13	39	38	31.1	50	1.26	6.43
14	42	41	33.5	50	1.28	6.83
15	45	45	36.8	50	1.30	7.38

Borehole 1 at width 75:



Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	7	5.7	75	1.01	1.45
2	6	8	6.5	75	1.03	1.64
3	9	10	8.2	75	1.04	2.02
4	12	12	9.8	75	1.05	2.39
5	15	13	10.6	75	1.07	2.56
6	18	15	12.3	75	1.08	2.92
7	21	17	13.9	75	1.09	3.27
8	24	21	17.2	75	1.11	3.99
9	27	25	20.5	75	1.12	4.69
10	30	27	22.1	75	1.13	5.01
11	33	32	26.2	75	1.15	5.87
12	36	36	29.5	75	1.16	6.53
13	39	38	31.1	75	1.17	6.81
14	42	41	33.5	75	1.18	7.27
15	45	45	36.8	75	1.20	7.89

Borehole 1 at width 100:



Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	7	5.7	100	1.01	1.45
2	6	8	6.5	100	1.02	1.64
3	9	10	8.2	100	1.03	2.03
4	12	12	9.8	100	1.04	2.41
5	15	13	10.6	100	1.05	2.58
6	18	15	12.3	100	1.06	2.95
7	21	17	13.9	100	1.07	3.32
8	24	21	17.2	100	1.08	4.06
9	27	25	20.5	100	1.09	4.79
10	30	27	22.1	100	1.10	5.13
11	33	32	26.2	100	1.11	6.02
12	36	36	29.5	100	1.12	6.71
13	39	38	31.1	100	1.13	7.02
14	42	41	33.5	100	1.14	7.51
15	45	45	36.8	100	1.15	8.18

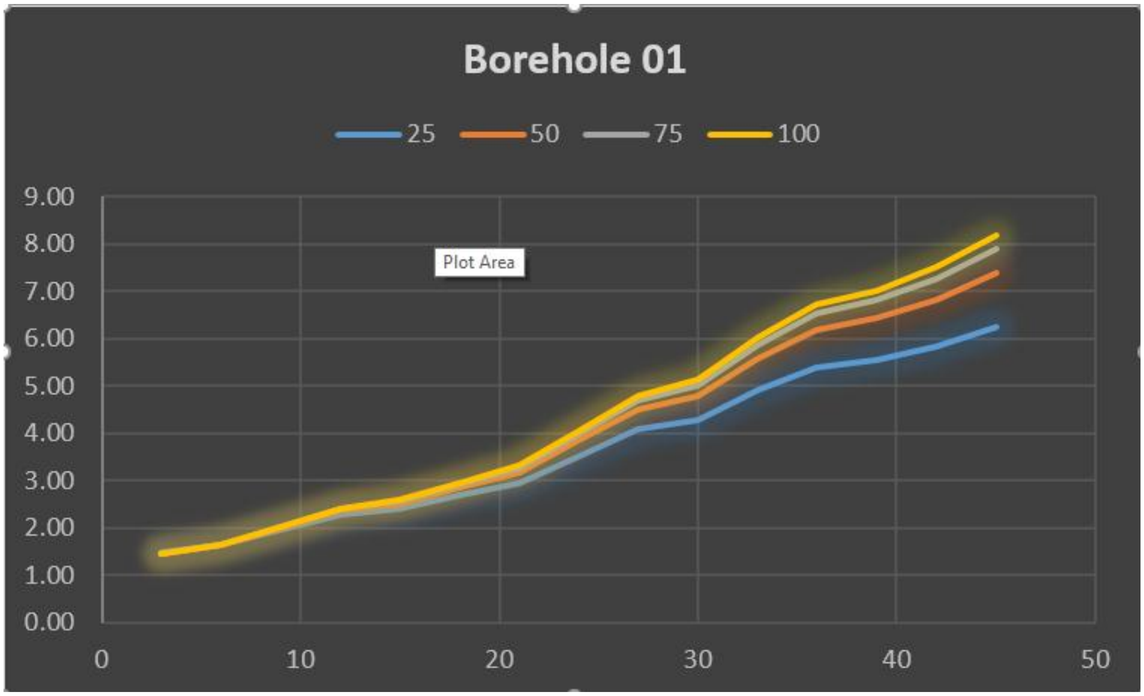
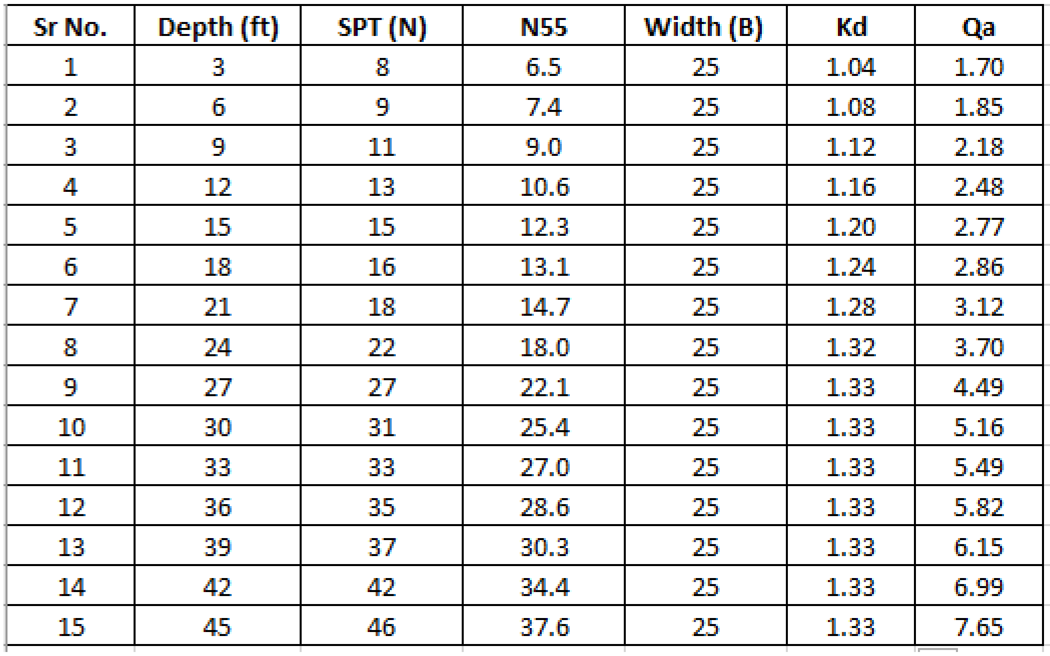


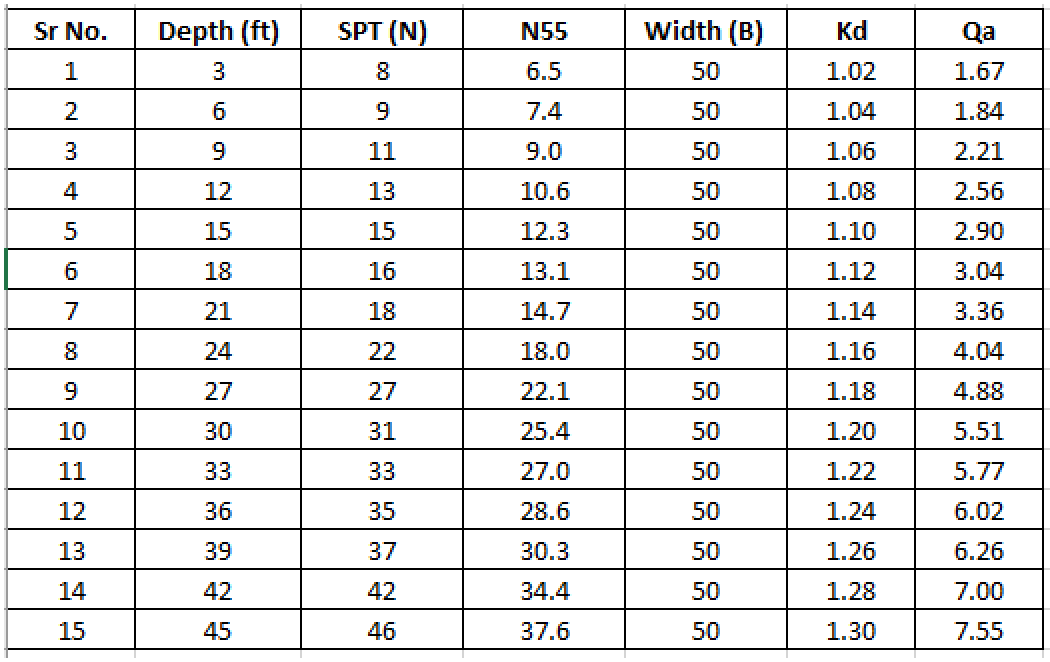
Figure 5.53. Bearing Capacity graph for BH-1.

Borehole 2 at width 25:



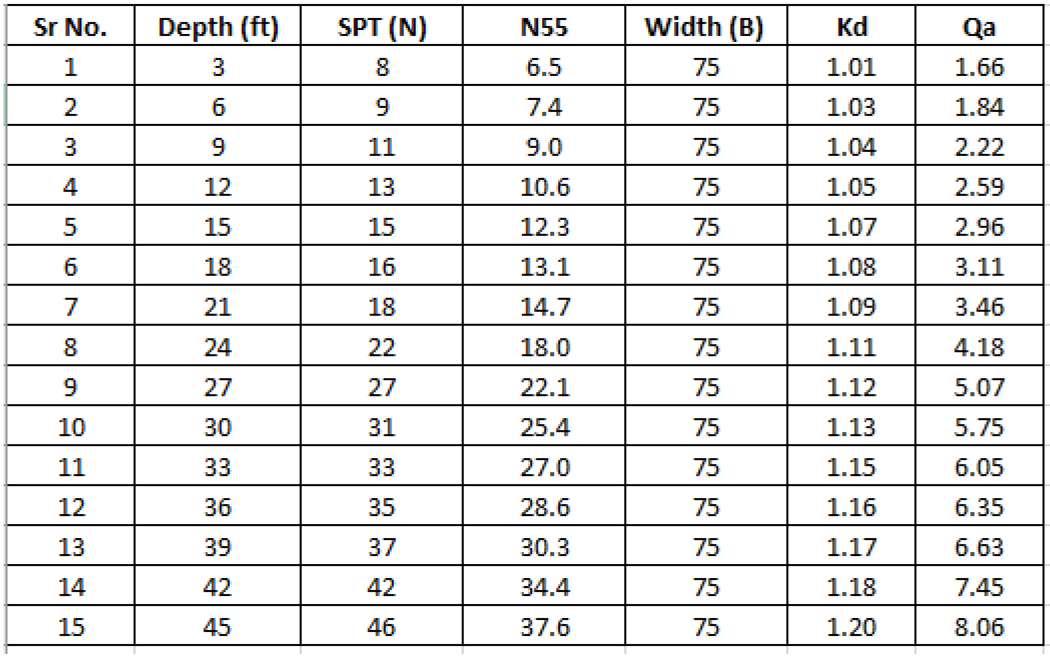
Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	8	6.5	25	1.04	1.70
2	6	9	7.4	25	1.08	1.85
3	9	11	9.0	25	1.12	2.18
4	12	13	10.6	25	1.16	2.48
5	15	15	12.3	25	1.20	2.77
6	18	16	13.1	25	1.24	2.86
7	21	18	14.7	25	1.28	3.12
8	24	22	18.0	25	1.32	3.70
9	27	27	22.1	25	1.33	4.49
10	30	31	25.4	25	1.33	5.16
11	33	33	27.0	25	1.33	5.49
12	36	35	28.6	25	1.33	5.82
13	39	37	30.3	25	1.33	6.15
14	42	42	34.4	25	1.33	6.99
15	45	46	37.6	25	1.33	7.65

Borehole 2 at width 50:



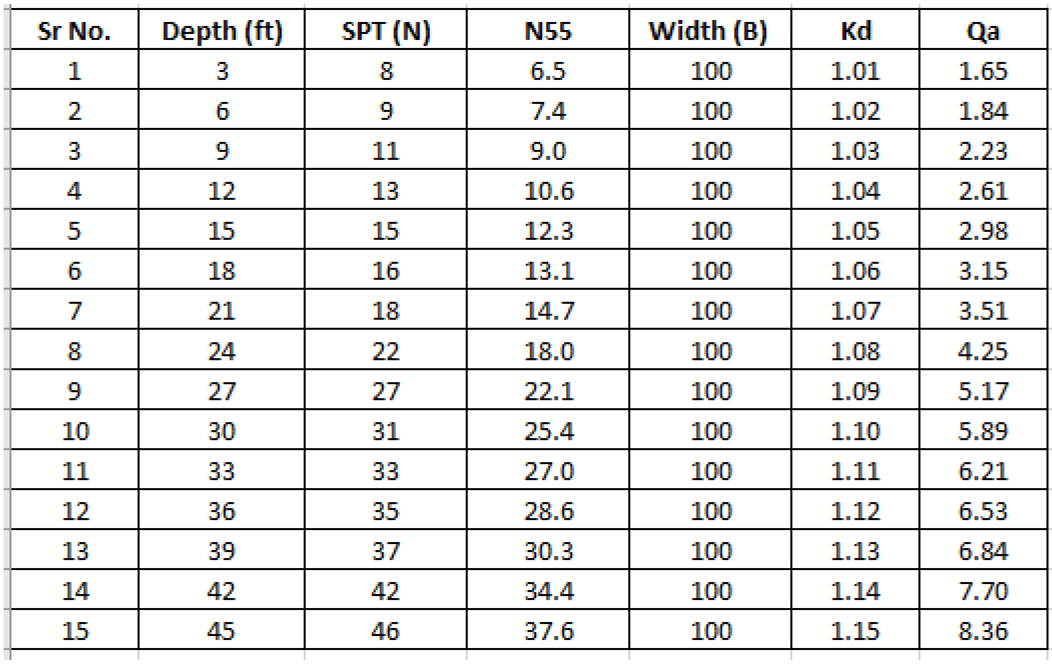
Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	8	6.5	50	1.02	1.67
2	6	9	7.4	50	1.04	1.84
3	9	11	9.0	50	1.06	2.21
4	12	13	10.6	50	1.08	2.56
5	15	15	12.3	50	1.10	2.90
6	18	16	13.1	50	1.12	3.04
7	21	18	14.7	50	1.14	3.36
8	24	22	18.0	50	1.16	4.04
9	27	27	22.1	50	1.18	4.88
10	30	31	25.4	50	1.20	5.51
11	33	33	27.0	50	1.22	5.77
12	36	35	28.6	50	1.24	6.02
13	39	37	30.3	50	1.26	6.26
14	42	42	34.4	50	1.28	7.00
15	45	46	37.6	50	1.30	7.55

Borehole 2 at width 75:



Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	8	6.5	75	1.01	1.66
2	6	9	7.4	75	1.03	1.84
3	9	11	9.0	75	1.04	2.22
4	12	13	10.6	75	1.05	2.59
5	15	15	12.3	75	1.07	2.96
6	18	16	13.1	75	1.08	3.11
7	21	18	14.7	75	1.09	3.46
8	24	22	18.0	75	1.11	4.18
9	27	27	22.1	75	1.12	5.07
10	30	31	25.4	75	1.13	5.75
11	33	33	27.0	75	1.15	6.05
12	36	35	28.6	75	1.16	6.35
13	39	37	30.3	75	1.17	6.63
14	42	42	34.4	75	1.18	7.45
15	45	46	37.6	75	1.20	8.06

Borehole 2 at with 100:



Sr No.	Depth (ft)	SPT (N)	N55	Width (B)	Kd	Qa
1	3	8	6.5	100	1.01	1.65
2	6	9	7.4	100	1.02	1.84
3	9	11	9.0	100	1.03	2.23
4	12	13	10.6	100	1.04	2.61
5	15	15	12.3	100	1.05	2.98
6	18	16	13.1	100	1.06	3.15
7	21	18	14.7	100	1.07	3.51
8	24	22	18.0	100	1.08	4.25
9	27	27	22.1	100	1.09	5.17
10	30	31	25.4	100	1.10	5.89
11	33	33	27.0	100	1.11	6.21
12	36	35	28.6	100	1.12	6.53
13	39	37	30.3	100	1.13	6.84
14	42	42	34.4	100	1.14	7.70
15	45	46	37.6	100	1.15	8.36

Figure 5.54. Bearing Capacity graph for BH-2

CONCLUSIONS

1. Water table was not countered in any borehole. Minimum and Maximum SPT values for BH-1 are 7 and 45 respectively while, 8 and 46 respectively for BH-2. Moisture content ranges from 13.9% to 30.3%. The liquid and Plastic limit ranges from 30% to 33% and 17% to 18% respectively for BH-1 while, liquid limit and plastic limit for BH-2 ranges from 31% to 33% and 17% to 18% respectively. %.
2. Allowable bearing capacity is measured against raft footing at various depths and footing width i.e. 25ft, 50ft, 75ft and 100ft. The depth of footing may be taken at or below 6ft from the road level.

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