BEARING CAPACITY FOR FOUNDATION OF FAQIR API FLYOVER, MANDI MOR, ISLAMABAD, PAKISTAN

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ABSTRACT

The aim of this study is to conduct a bearing capacity analysis and to provide the most suitable foundation design for the construction of Faqir Api Flyover, Mandi Mor, Islamabad, Pakistan using ASTM standards. The subsoil investigation program was carried out by drilling two (2) boreholes at the site's location. The method of percussion drilling was adopted and boreholes drilled up to a maximum depth of 36m.The subsoil mainly encountered was silty clay (CL-ML) and clay (CL). Disturbed samples were obtained, in situ tests were carried out and soil strength assessment analysis was performed. The Atterberg Limits (ASTM D-4318) and grain size analysis (ASTM C-136) were performed to interpret the sub soil geotechnical behavior. Average Minimum and Maximum SPT values encountered in both bores were 13 and 50 respectively Moisture content ranges from 7.8 to 9.7 %. Allowable Bearing Capacity for Pile is measured from 0 to 30 meter against 5 different diameters i.e. 0.75m, 0.90m, 1.0m, 1.2m and 1.5m. The diameter and length of the pile found to be 1.2m and 25m to achieve the necessary allowable load. Using AASHTO LRFD equation for deep foundation, bearing capacity was calculated. On the basis of field and lab test results bearing capacity was calculated and pile foundation is recommended for the site.

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In completion of our thesis, we wish to express our gratitude to the individuals who have provided their heart whelming support. First of all, for supervising us in this study, Mr. Saqib Mehmood (Senior Assistant Professor, E&ES Department) for supporting us, we really appreciate his efforts, care and conditional support and guidance.

This research work is dedicated to our parents, who have always loved and respected us. We thank them for providing us with motivation and guidance. We are grateful to all of our teachers, too, who have taught us and developed our skills. In every area of life, our teachers' advice and recommendations will always help us.

ABBREVIATIONS

ASTM	The American Society of Testing Materials
BH	Bore Hole Logs
CL	Lean Clay
DS	Disturbed Soil Sample
GC	Clayey Gravels
GW	Under Ground Water
PMD	Pakistan Meteorological Department
R	Refusal
UDS	Undisturbed Soil Sample
USGS	United States of Geological Survey
SPT	Standard Penetration Test
Qp	Point Bearing Capacity
Qu	Ultimate Bearing Capacity
Mya	Million years

CONTENT

CHAPTER 3

TECTONIC SETTINGS

CHAPTER 4

METHODOLOGY

CHAPTER 5

RESULTS AND DISCUSSION

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Introduction

Engineering geology is a field where evaluations can be made based on the performance of soil mechanics, moreover it includes properties that impact the project and subsurface conditions and evaluation of the issues related to technical conditions are coped up. Site, construction, and foundation supervision is executed as per proper earthquake design.

The foundation is primarily the bottommost part of a superstructure. The mass of the structure is absorbed, and stress is moved to the soil or plane underneath. Thus, this engineered part is referred as super structure. Overall, foundation is the utmost main feature of an engineering system.

Good quality foundations have capabilities of equally distributing all load across ground while restricting stress over the soil. It is important as too much stress on the soil causes depression or subsidence in an area causing damage to the engineering structures over time. To avoid such consequences, different surveys have to be conducted by companies. Bearing capacity of the area also must be calculated.

1.2 Types of Foundation

The foundation type that is to be applied is largely dependent on the structure and consequently the soil found there. The basic forms are mainly categorized into two classes: shallow and deep foundations. Such definitions are used to describe the depth of soil with which to lay the base. The shallow foundations are laid at depths of about 9 meters while the deep foundations are laid at approximately 20-60 meters deep. As the name suggests, shallow foundations deal with light and small structures, and deep foundations deal with high weight and large structures.

1.2.1 Shallow Foundation

A shallow foundation is a sort of construction base that transports construction loads very close to the surface rather than to a subsurface layer or a variety of depths, like a deep foundation.

The foundation depth must meet the safety requirements of the breakdown, whereby after the load application, the complete structure settlement will be within acceptable limits (Fig. 1.1).

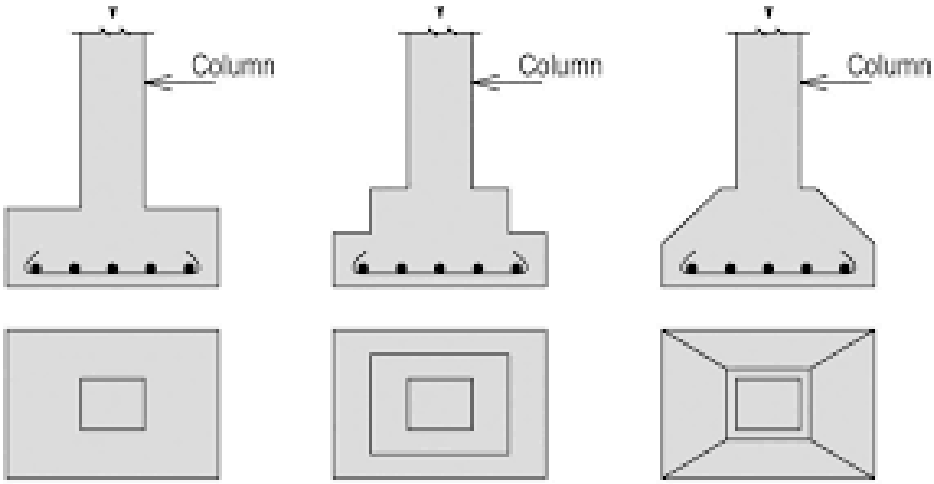

Figure 1.1. Shallow Foundation (Bowels, 1996).

The types of shallow foundation are:

1. Raft Foundation
2. Isolated foundation
3. Strip Foundation

1.2.1.1 Raft Foundation

It consists of cemented thick slice of block strengthened by steel on a large area of soil supporting the columns/walls and moves the load throughout the soil. It is also known as Mat Foundation shown in (Fig. 1.2). It is applicable where:

1. Bearing capacity of soil is low.
2. When the load of the structure is to be divided equally on a large area.
3. Basement needs to be built.

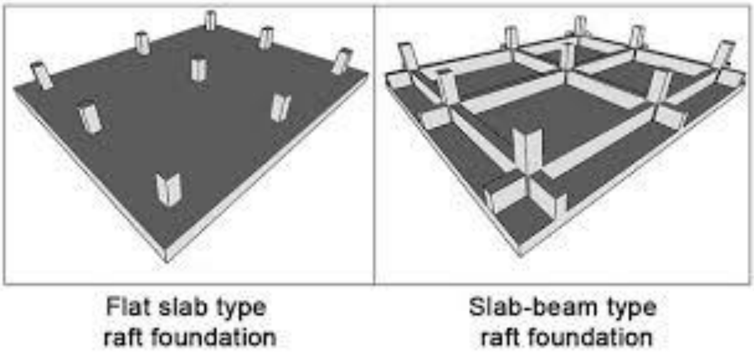

Figure 1.2. Raft Foundation (Bowels, 1996).

1.2.1.2 Isolated Footing

They are called as pad or spread footing foundation. They are also called as Pillars as they divide and carry the load of the structure and are utilized in shallow foundations. This type of footing may be enforcement or non-enforcement. The non-strengthened footing should be large or heighted to give the required load division (Fig. 1.3).

It is applicable where:

1. A single column needs to be supported.
2. When columns are arranged at relatively longer distance.

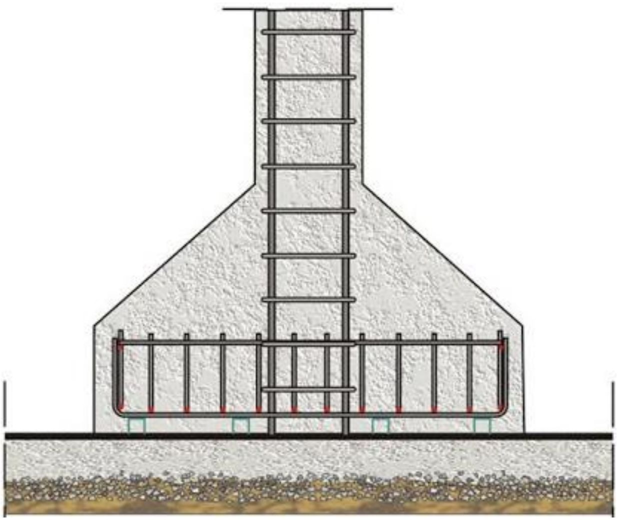

Figure 1.3 Isolated Footing (Bowels, 1996).

1.2.1.3 Strip Foundation

The foundation of a wall or a strip is a continuous concrete strip, which helps to distribute the weight of a bearing wall over a surface of the ground. It is a shallow base component. They are largely used for load carrying walls foundation. Its width is generally twice to that of the wall, or it can be more to that as well. The width and the material used for strengthening varies with bearing capacity of soil under the influence of foundation.

The soil is of reasonable bearing ability and the strip foundations are used. The main strip base sizes are identical for the construction of a concrete cavity wall and a wood frame wall cavity. The band size and location are directly correlated with the wall width (Fig. 1.4).

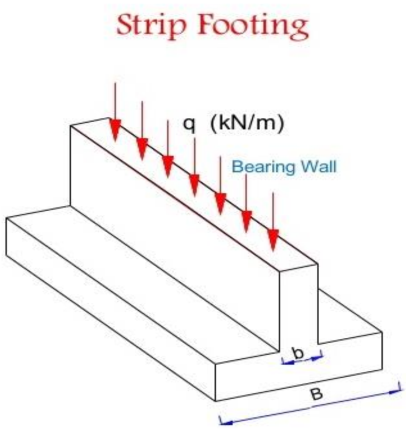

Figure 1.4. Strip Footing (Bowels, 1996).

When the surface soil is solid and firm to support the structure, a shallow base form is applied. Nevertheless, they are fragile and can be compressed by the building when there is poorly compacted land or when there are alluvial deposits.

1.2.2 Deep Foundation

Deep foundation is laid deep into the ground. This foundation is more prone to earthquakes and natural instabilities hence more stable overall. The depths vary up to 60 meters deep. Deep foundation furthermore types are as under:

1. Pile Foundation
2. Drilled shafts

1.2.2.1 Pile Foundation

Pile foundations are relatively lengthy and lean components built by driving preformed units to the required founding point, or by driving or drilling in tubes to the needed depth – tubes that are filled with concrete before or during withdrawal, or by drilling unlined or partially lined boreholes that are later fille d with cement. It consists of a strong cylindrical material made of concrete or timber. They can be used to lay down deep foundation which costs more than the shallow foundation.

They are used in the scenarios like:

1. It will support the systems in the same way if they are below the water table to prevent forces from moving upwards.
2. In case of horizontal forces acting in that area, same can be done to prevent bending and support the structure's load at the same time.

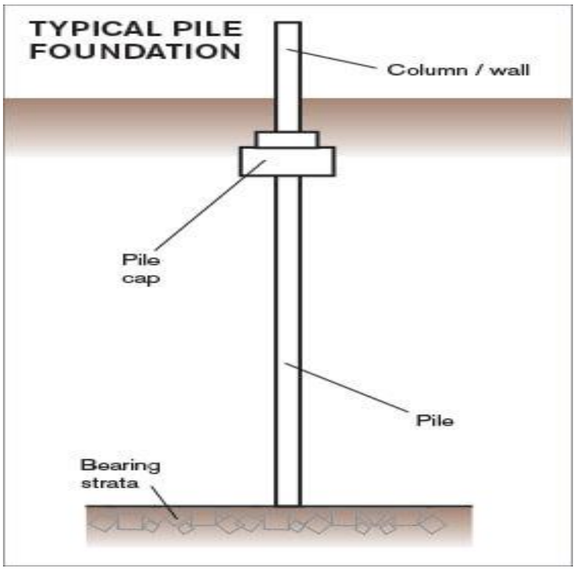

Figure 1.5. Typical Pile Foundation (Bowels, 1996).

1.2.2.2 Drilled Shaft Foundation

It is also called as a caisson, drilled shaft, Cast-in-drilled-hole piles (CIDH piles) or Cast-in-Situ piles. Shaft foundations are built inside deep burial sites supported by liner built on site and subsequently filled with concrete or other load bearing units. They are piles that are mounted deep into the field and have a diameter of around 0.6 meters.

This has many benefits, some of which include:

1. Only one drilled shaft can be adequate rather than a collection of piles.
2. No noise pollution caused by hammering as opposed to pile pushing.
3. They can withstand strong forces coming from lateral loads.

As other devices drilled shafts have their own drawbacks, such as delaying the process due to bad weather, and they also need continuous monitoring.

1.3 Location of Study Area

Geographically, the proposed project site is located in the western Limb of Hazara Kashmir syntaxis which situates in the central periphery of Lesser Himalayas of Pakistan. For geographic location of the Proposed Project Site and a detailed view of the Project location on Satellite Imagery along with the Project Area Boundary (PAB).

The proposed site is located along IJP road near Carriage Factory, at Location Coordinates of 316937.384 E, 3723117.721 N in the Federal Area Pakistan. Proposed site is approachable via metaled road from Srinagar Highway (Fig. 1.6).

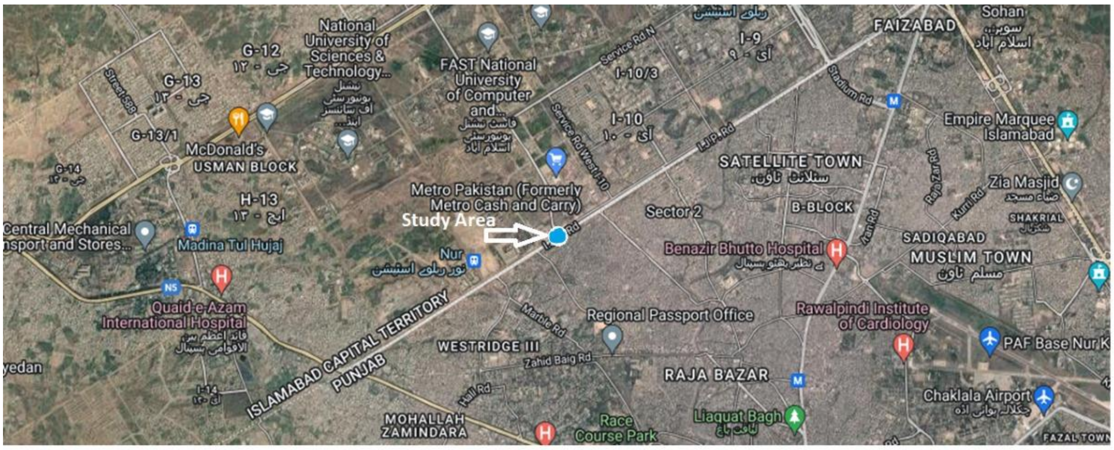

Figure 1.6. Project site (Google earth).

1.4 Objectives of Investigation

1. Bearing capacity assessment of the foundation.
2. Selection of correct foundation design.
3. To evaluate geotechnical parameters on the basis of type of soils

1.5 Methodology

The methodology of research work involves field activities including borehole drilling and excavation of representative samples. Field testing was done by using Standard Penetration Test (SPT) for soil and Rock Quality Designation (RQD) was performed for rock strata. Tests that were performed in the laboratory include Atterberg limit and Sieve analysis. Eventually, test results were interpreted, and the bearing capacity of the foundation is calculated by using Meyerhof equation. The flow chart of the methodology is shown in (Fig. 1.7).

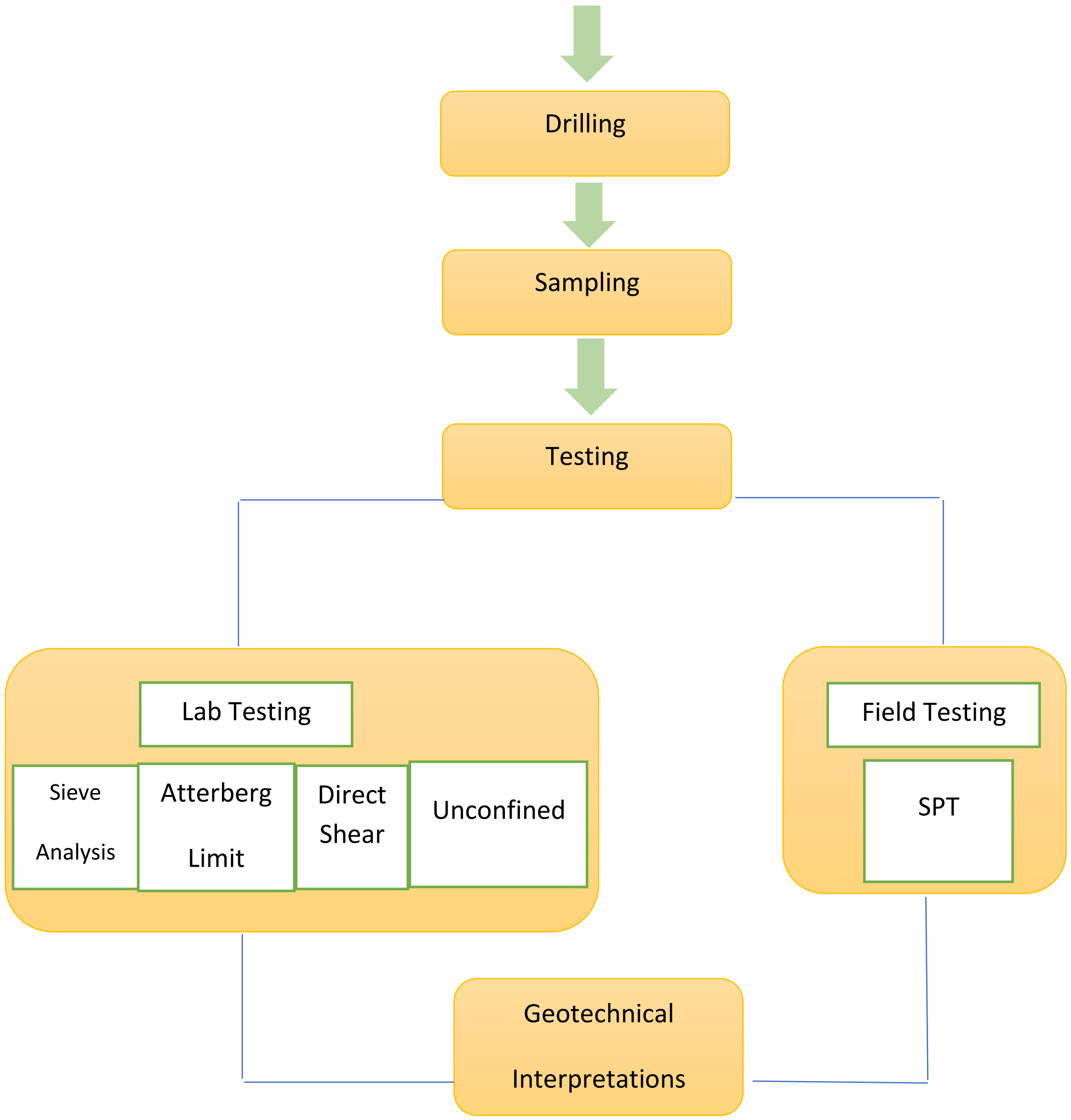

Figure 1.7. Methodology flow-chart

CHAPTER 2

LITERATURE REVIEW

2.1 Background

Human beings have been utilizing soil for development functions since early ancient times and the archeological facts proves that like the Indus civilization in Pakistan that was the most prosperous civilization of Asia. It proves that soil development and geological sector initiated in early ancient times and 19^{th} century was the time where engineering geology field progress rapidly.

The early development and utilization of geological engineering is considered to be originated from Egypt, China, India and all the developing world followed it, the dam like structure built on Indus basin in about 2000 B.C is an example that was used as a water source by the Mohenjo Doro civilization. We are still not sure that how the foundation of the structure was balanced in that time for such a long period of time.

Leonardo da Vinci contributed a lot to field of architecture and engineering geology in late 15^{th} century. He developed calculating of bearing capacity to calculation of angle of response of sand based on his observations of the behavior of soil. After this he also contributed in ground water studies and the principles of hydrology and written all his lifetime work in a book that tactlessly wasn't applied practically.

Pisa tower of Italy is a case where the tower tilted as the soil and strata underneath was not stable and there was definite lack of soil investigation. Later we have come to know that there was loose and compressible soil underneath the tower that led to its tilting and because of this, proper investigation of site area for big structures initiated, to avoid such mishaps in future.

In 1857, Rankine presented the force types acting on the soil as well as along the plain fracture. He proposed his theory (known as maximum stress theory) for soil failure, according to him, soil failure is expected to occur when maximum principle stress gets to the equivalent value as the tensile stress. This theory can only be applied on the fragile materials and can't be applied on ductile material, it does not explain how the failure effect is created by remaining two forces.

Later, effects of soil expansion were explained by Osborne Reynold in 1887, at the same time, other scientists like John Clibborn and Stuart Beresford demonstrated use of sandbags and increasing the water's flow pressure.

Field of engineering geology greatly excelled in the 19^{th} century; this developmental period is also referred as the golden time of this field where famous scientist William Penning also wrote book in this field. In 1911, by establishing the idea of homogenous cohesive soil consistency, Atterberg explained the concepts of shaking limit, liquid limit and plastic limit.

Stress and consolidation theory were explained by book 'Mechanics of earth construction based on soil physics' by Terzaghi in 1925. This book also created a spark of interest and importance of different observations needed in this field.

Failures like San Francis Dam, California ending up with about 426 deaths caused attention of world to focus on this field. Soil investigation was made compulsory to avoid such disasters in the future.

Soil compaction, seepages and soft clay were examined by plasticity chart developed by Casagrande in 1932.

Meyerhof in 1951 redefined Terzaghi's work and presented the equation of shallow and deep foundations adding depth term Nq (supercharge) and shape factor term (s-q), factors of depth and inclination were also introduced.

For geotechnical purposes, devices like hydraulic piezometer, SGI inclinometer and various settlement measuring gadgets were developed by Kallstenius. He also practiced interpretations with different is meters and penetrometers and his contributions were appreciated by all over the world.

Advancements in soil investigations and hazards evaluations have led to better and stable structures evidencing the importance of this field in construction projects.

CHAPTER 3
TECTONIC SETTINGS

3.1 Geological Record

From the middle Jurassic age to the Quaternary age, sedimentary strata found in this area reveals 150 Mya. Slow sediment accumulation and slight tectonic activity (primarily marine) is observed in the interval from about 150 to 24 Mya. Time as of around 24 to 1.9 Mya is described with means of big voluminous mainland rapid deposition and gradual subsiding. Since then, the rapid tectonism, massive erosion and subordinate regional deposition with thick clastic crustal sediments overshadowed. Prior to merging with the Eurasian tectonic plate, along northern margin of India-Pakistan plate Jurassic marine limestone and dolomite were accumulated thus they are the oldest rocks exposed in this area. The varying quantities of energy in the various carbonate depositional settings are revealed with different limestones forms of the biomicritic, oolitic and intrasparitic through the Samana Suk Formation.

The unconformity presents between Chichali and Samana Suk represents a brief pause in deposition through the (late) Jurassic phase. Through (late) Jurassic to (early) Cretaceous Chichali sandstone and glauconitic shale were accumulated in reducing settings and bottom anaerobic environment. From the start of Cretaceous, conditions swung to a fairly salty, shallow-water, atmospheres turned to reducing when the glauconitic sandstone of Lumshiwal was formed, and calcareous facies from Lumshiwal are near-shore shallow-water deposits. The unconformity presents between the Kawagarh and Lumshiwal implies that the section above sea level raised during the (middle) Cretaceous. Then sea transgressed again during the beginning of Late Cretaceous causing the erosion of the marl and the limestone from Kawagarh leading to their deposition from shallow to deeper aquatic conditions.

Coastland arose above sea level again throughout the Late Cretaceous to Paleocene periods. Initially, exposed surface was eroded then covered behind extremely worn Hangu Formation continental deposits. The Kawagarh was eliminated from the map area, and the Hangu now sit as unconformable on the Lumshiwal Formation. The Hangu Formation's strong weathering reveals the equatorial position of the Indian tectonic plate during Paleocene. Marine conditions restored once the Hangu was deposited and weathered, and they lasted until the early Eocene. During this period calcareous and argillaceous deposits were produced by the Patala formation, Lockhart Limestone, Margalla Limestone and Chorgali and the Kuldana formation was deposited in alternate marine and continental conditions after this marine depositional period.

The initial collusion Indian plate with Asia during the (middle) Eocene raised the land higher than the sea level, resultant as unconformity under the Murree formation mainland. The sea had entirely withdrawn south of the map area and large thick mainland deposits of the Siwalik Groups and Rawalpindi accumulated in the orogeny throughout the period of Miocene and Pliocene. These deposits are made up of sediments eroded from the northern highlands and elevated and distorted by tectonic processes in the convergence zone. The deformed zone's south boundary extended southward into the Islamabad region, causing coarser sedimentation at first but subsequently deforming and uplifting the area to the point where deposition was considerably reduced and eroded.

The Eocene period’s tectonic movement that began continues to this day. During the Pliocene, the typical rate of southerly movement was 3 centimeters for every 1,000 years, whereas the accretion of sand, gravel and mud in the sinking foredeep region was about 28 cm for every 1,000 years (Raynolds, 1980, p. 191). An eastward-flowing river system regulated sedimentation during the Pliocene (Raynolds, 1980). The Soan’s conglomerate formed through (late) Pliocene. Clasts seen in present Indus River gravels are mostly metamorphic clasts and quartzite that eroded from the Himalayas. From 3 to 1 Mya, regional accumulation came to a halt and Hazara fault geographical zone formed, the Margalla Hills limestone got thrust up alongside the northern boundary of the region, the sandstone and mudstone from Siwalik were faulted and folded through the region.

The southwards flowing Soan river (much smaller) disrupted and replaced the eastwards flowing river system, limestone gravels became the major component of Lei Conglomerates. Climatic abrupt changes during the Quaternary, sideways tectonic uplift caused repeated drainage and incision of the southern Margalla Inclines and substitute recurring accumulations of alluvial gravel and silt from the Margalla Rises, which laterally scatter the valley. The Eolian silt loess that was deposited from Indus water glaciers formed the thick deposits on the countryside and contribute to the interment of already existing lowland.

As big glaciers were present in Indus River basin, loess deposition continued leading to enormous quantities of fine-grained sediment contribution, causing Indus to make a braided channel under the highland. Through the time from 170 to 20 ka rates calculated of loess accumulation varies from 6 to 27 cm/1,000 years. Due to lack of stable surfaces, discontinued dry climate and due to loess accumulation and constant erosion well-established soil profiles are meager in this region. A few paleosols however maybe preserved within the loess. Along the fore of highland, fan-terrace deposits and Pleistocene stream are also present in disturbed pattern.

Equilibrium of degradation and aggradation patterns may also have been disturbed by far off tectonic events. The tectonic uplifting and tilting through the route of the river Indus close to the Kalabagh gorge has caused large shifts in the Indus pattern and also affected soan river base. In this area thrust faulting, folding and seismicity indicate active tectonics. The Taxila Buddhist village near Islamabad was demolished by a huge enough earthquake in 25 A.D. The recent earthquake that caused significant damage near Rawalpindi was in 1977 having magnitude of 5.8, and 7.6 magnitude earthquake in 2005 causing damaging in Islamabad as well as wide northern area of Pakistan

3.2 Tectonic settings

Pakistan a very important country in the context of Geology and Tectonics of Subcontinents. In context of stratigraphy and mineral deposits Pakistan has a clear edge in subcontinent and Asia. If we talk about the tectonics settings of Pakistan, then there are many important features which are present in Pakistan like Himalayas and its sub-components. Himalayas are formed due to the Northward movement of Indian Plate. The sub-components of Himalayas are:

1. Higher or Upper Himalayas
2. Lesser Himalayas
3. Sub Himalayas

The Indo-Pak subcontinent, which was earlier a member of Gondwanaland, split from the motherland around 130 Mya ago and drifted northward. Intra-oceanic subduction produced a sequence of volcanic arcs such as the Chagai Arc, Kohistan-Ladakh, Nuristan, and Kandahar Arcs when the Indian Plate was drifting north around 55 million years ago in the Eocene. The Kohistan-Ladakh arc collided with Eurasia from 10-85 Mya ago when the back arc basin closed. The Kohistan arc developed an Andean Type passive margin after accretion to Eurasia. Around 65 to 60 million years ago, the northward advancing Indian plate collided with the Eurasian Kohistan-Ladakh margin. These activities were responsible for the formation of the Karakorum and Himalayan Ranges, as well as the sedimentation and evolution of sedimentary basins. They formed significant tectonic features, created magmatic sequences, and, most importantly, linked the associated mineral deposits.

3.3 Main Karakoram Thrust (MKT)

In Pakistan's northwestern province, it is a prominent feature that in the past where Karakoram Block from north collided with Kohistan Island Arc in south. Kohistan Island Arc collided with Himalayas is believed to have happened between 50 and 55 million years ago. The northern boundary of the Eurasian Plate and the southern boundary of the Indian Plate are marked by MKT (400 km). MKT distinguishes KIA's Cretaceous Tertiary rocks from Karakoram Block's Late Paleozoic metasediments.

MKT is also called the northern suture zone. In the Ladakh region, it is also called the Shayok suture zone. Some mineral deposits occurring along the MKT zone are Iron ore, graphite, topaz, tourmaline, quartz crystals, ruby, epidote and tungsten.

3.3 Mian Mantel Thrust (MMT)

MMT represents the southern boundary of KIA and Northern margin of Indian Plate (Kazmi and Jan, 1997). It was formed by subduction and subsequent collision of Indian Plate and KIA in Eocene time. This fault zone is having mantle related ultra-mafic, metavolcanic, meta-gabbros and phyllite. This zone is comprised of complex sequence of melanges which are composed of tectonic blocks of ophiolites, blue schist, greenschist and metasediments in matrix of sheared metasediments. Deposits of asbestos, chromite, peridot, emerald, magnesite, talc, soapstone, and minerals associated with gold are found in MMT fault zone (Kazmi and Jan, 1997).

3.4 Main Boundary Thrust

MBT is a fault mechanism that wraps around the HKS (Hazara Kashmir Syntaxis) in a hairpin formation (Kazmi and Jan, 1997). In the west, north, and east, MTB has taken Mesozoic rocks into faulted contact with Murree Fm. The MBT fault zone is made up of a series of thrust faults that separate the deformed and metamorphosed northern zone or hinterland from the deformed sedimentary southern zone or foreland in the NW Himalayan chain. Tight folding of MBT's hanging wall resulted in increased shortening. This may be the primary cause of the Margalla Hills' folds. The Jurassic Samanasuk Formation unconformably overlies Paleocene Hangu, Lockhart, and Patala Formations, which are underlying Eocene Margalla Hill Limestone, Chorgali, and Kuldana Formations in MBT's hanging wall stratigraphy.

3.5 Geology of Study Area

The geological for safe and modern construction of international standards, the past and present geologic factors should be considered for this purpose a brief geologic description is given below.

Conditions of the Rawalpindi/Islamabad region are characterized by the clash of the plates, which began about 55 million years ago. Many Pakistani and international geologists have examined the complex structures and strata that resulted from these events in the Rawalpindi/Islamabad region. In just about roughly 675 m of predominantly marine sedimentary rocks reflect the 150 Mya period deposition sequence from Samana Suk (Jurassic age) to the beginning of Murree Formation (lower Miocene age). More than 7,572 meter of mainland sedimentary strata reflects the preceding 20 million years. Erosion has dominated deposition throughout the previous 1.5 million years of uplift and structural deformation, leaving mainly thin, discontinuous masses of alluvium and eolian silt. Islamabad is located on the Hazara fault zone's southern perimeter and leading edge.

Except the southern faults of Rawalpindi, all the faults in map area constitute a fault zone. This fault zone is arched towards south and stretches toward south-west wards (away from the Himalayan region). North of Islamabad, it comprises of folded arc of uplifted rocks of around 150 km length and 25 km width where more than 20 distinct thrust sheets have been recognized however just 5 primary thrusts lie within the map region (as shown under in fig 2.1). In this region, several thrust faults are a little oblique to the fore of Margalla Hills; thus, projecting south-west ward underneath the area of piedmont fold belt (Fig. 1.6).

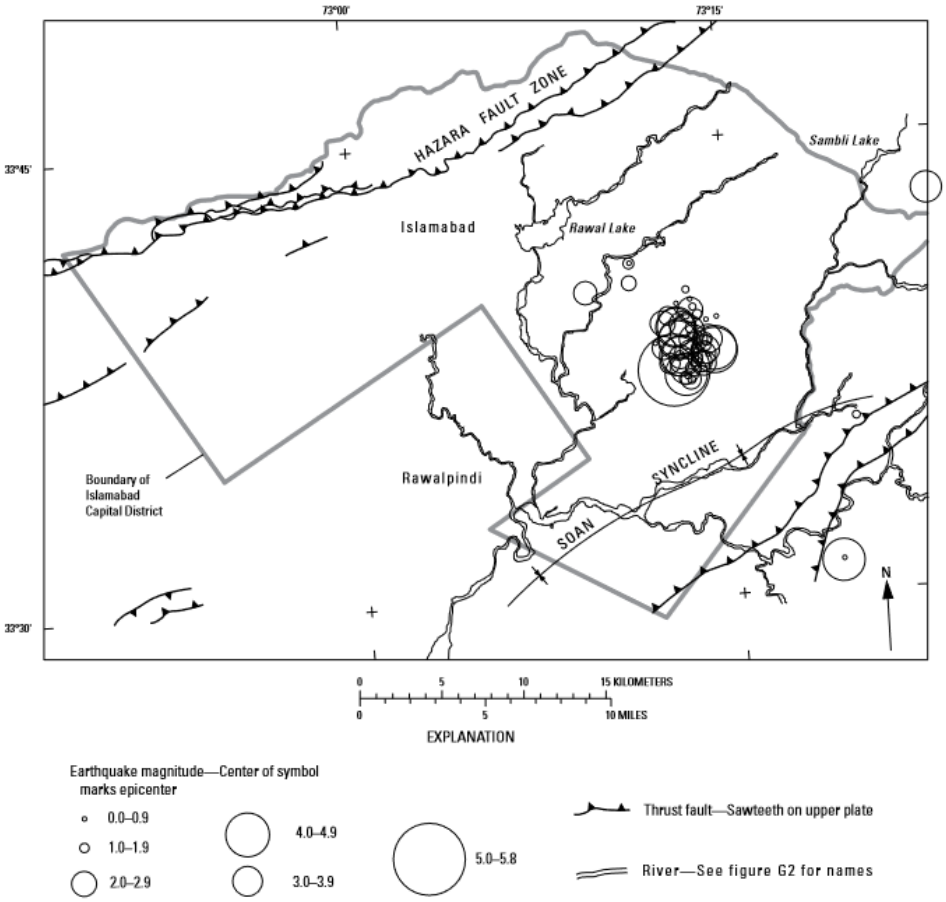

Figure 3.1. Map of the Northern Potwar Area

3.6 Geologic Stratification

As explained earlier, the geological history reveals that the Galiat hills were formed during the mountain building process (Orogeny) of Himalayas. Himalayan orogeny is the result of the subduction of the Indian Plate beneath the Eurasian Plate at a constant rate of 4cm/year. The result is the formation of several thrust faults among which Main Boundary Thrust (MBT) have traces near the study area. MBT is resulting in the thrusting of Murree Formation with the Paleozoic rocks (Gohar, 1987). MBT can be traced at Darya Gali, located about 8km from Murree on Murree-Nathiagali Road.

The geologic map of the study area is showing Fig. 2.4 (GSP, Sheet No.1, 2000). The rock units on the southern side of MBT consist of Miocene age Murree Formation, while rocks units exposed on the northern side of MBT (Darya Gali to Abbottabad) consist of several Formations ranging from pre-Cambrian to Eocene age. The project area entirely consists of Murree Formation (Tmm) and top layer consist of quaternary deposits having over burden soil firm to Hard Silty clay/Clayey silt, low to medium plasticity, low to medium dry strength (GSP, 2000) (Fig. 3.2).

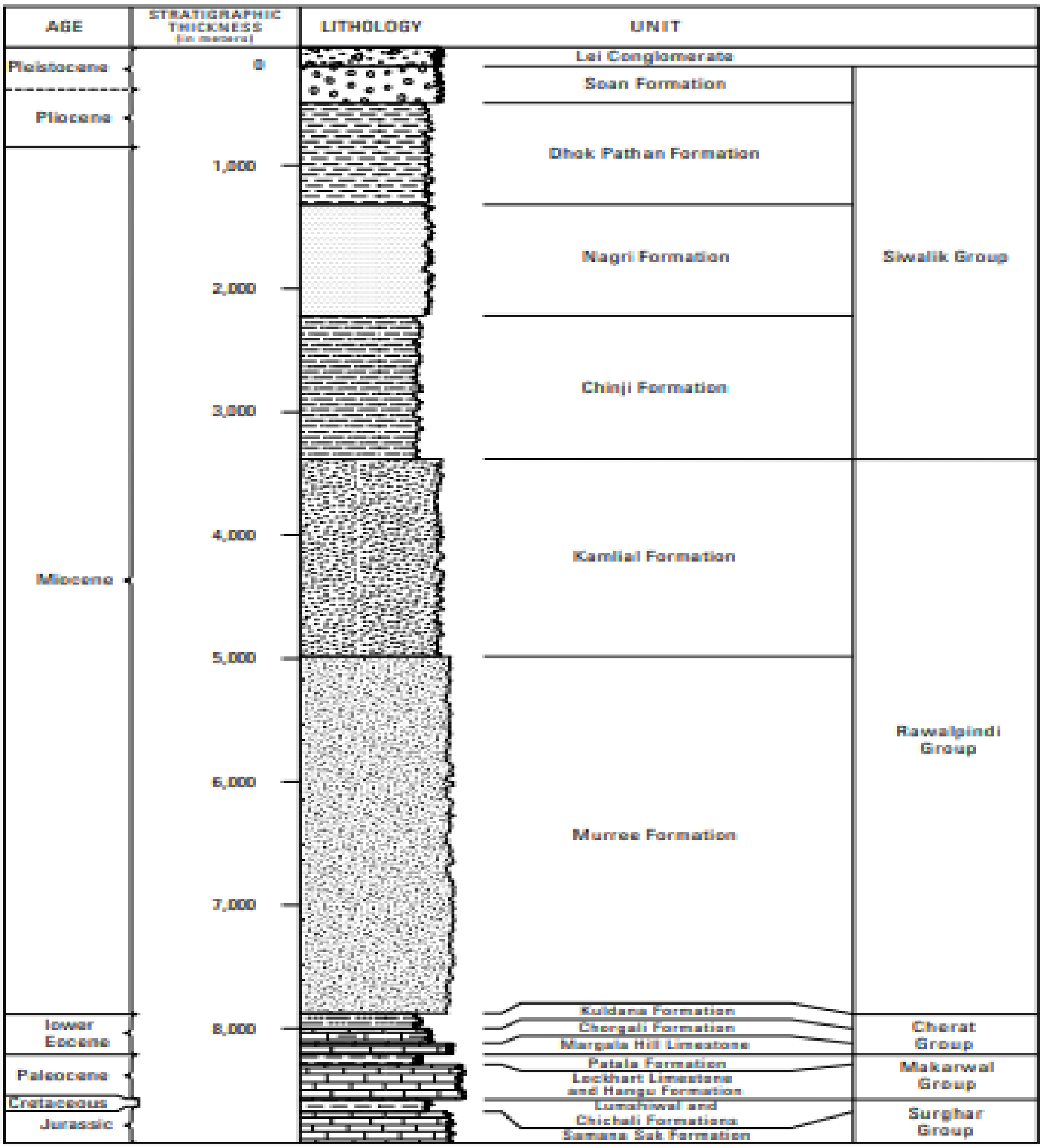

Figure 3.2 Generalized stratigraphic section of consolidated rocks in the Islamabad/Rawalpindi area.

3.7 Seismicity of the Region

As per seismic provisions (2007) of building code of Pakistan, the project location is in Zone 2B. Moderate level of harm in duration of the seismic loading is observed in this zone. Considering that the structure is supposed to be constructed to survive 0.16 - 0.24 g values of maximum horizontal peak ground acceleration. For this PGA, in 50 years it has 10% possibility of exceedance (Fig. 3.3).

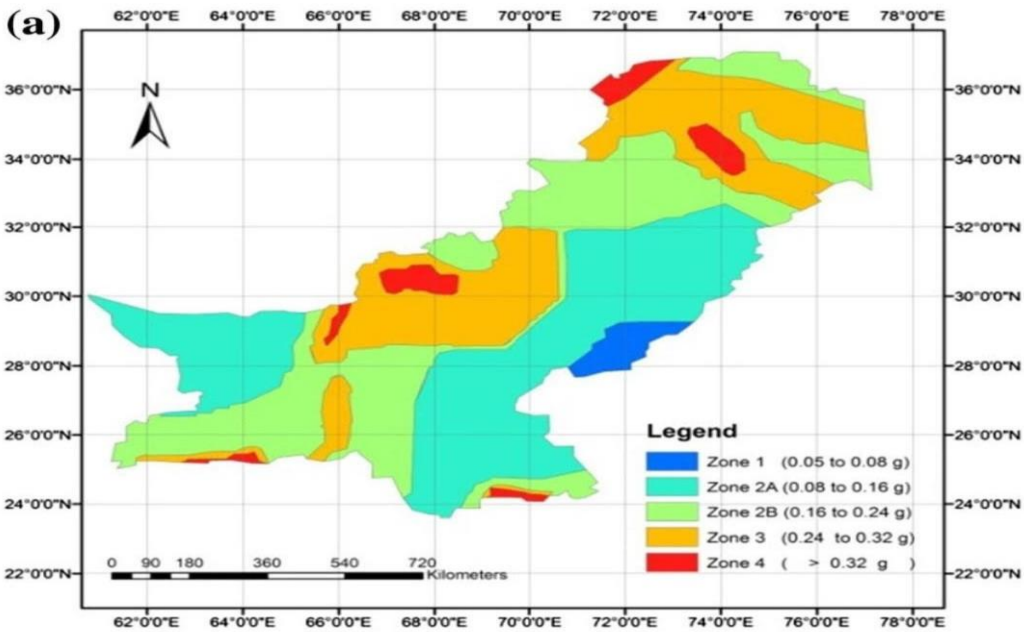

Figure 3.3. Seismic zones of Pakistan

CHAPTER 4

METHODOLOGY

4.1 Field Activities and Testing

On the site, initially boreholes were to be dugged at two different points on the working area. Depending upon the nature of structure and sub-soil condition we could have increased the number of boreholes.

4.2 Planning

Foundation conditions evaluations at the Project sites require boreholes of variable depths at major locations in the reference to project requirements. Maximum coverage of area is acquired by these locations. The location of all the boreholes is marked on the Geotechnical Investigation Plan.

4.3 Standard Penetration Test (ASTM D1586)

SPT is assessment test of soil resistance to penetration in underneath associated strata in drilled holes. This test is utilized to estimate relative density as well as the angle of resistance for less soil cohesion. Moreover this test can be used to get unconfined compressive strength of cohesive soil.

4.3.1 Equipment

1. 63.5kg Hammer
2. Guiding rod
3. Drilling rig
4. Split Spoon Sampler
5. Anvil (driving head)

Figure 4.1 Split Spoon Sampler.

Figure 4.2 Sample (Split Spoon Sampler).

Figure 4.3. Drop hammer of 63.5kg used for SPT.

4.3.2 Safety Measures

1. Split-Sampler should be in good/working condition.
2. Cutting shoe should not be broken.
3. Drop height of the hammer should be 30 inches for the accuracy of N values.
4. Drill rods should not be allowed to bend for the accurate results.
5. Bottom potion of the borehole must be clean before performing the test.

4.4 Disturbed Sampling

The SPT samples obtained from overburden soils from boreholes were properly labeled and preserved as disturbed samples. All the disturbed samples were transported to an approved geotechnical testing laboratory as per ASTM D 4220.

4.5 Undisturbed sampling

Two (02) relatively undisturbed soil sample was recovered from borehole using Shell by tube/samplers as per ASTM D 1587. After determining the in-situ density, the soil sample was properly waxed, labeled, preserved, and transported to an approved geotechnical testing laboratory.

4.6 Rock Quality Designation (RQD)

Rock-quality designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm or more. High-quality rock has an RQD of more than 75%, low quality of less than 50%. The RQD denotes the percentage of intact rock retrieved from a borehole. All pieces of intact rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run.

4.7 Laboratory Testing

After the examination of samples on the field, laboratory testing is performed to find the typical engineering characteristics of sub-strata.

1. The Sieve Analysis Test (ASTM D–422)
2. Atterberg Limit Test (ASTM D-4318)
3. Dry Density and NMC %
4. In-Situ moisture content (ASTM D-4944)

4.7.1 Sieve Analysis Test (ASTM D 422)

4.7.1.1 General Procedure

1. Soil sample is made oven dried initially that should weight just about 300g.
2. Pestle and mortar are utilized in case where soil is lumped or conglomerated.
3. Ascertain mass of sample precisely Wt (g).
4. Stack of sieves is set up in a way that larger sieves (containing larger opening sizes with lower numbers) are to be positioned above the smaller sieves (containing smaller opening sizes with higher numbers) to 200th position at last.
5. Then soil passing through 200th sieve is collected by a pan.

6.Cleaned sieves are used. soil trapped in the openings is taken out by using brush.

7.Poured from top, soil is fixed from clamps by using shaker for around 10-15 min.

8.Sieve retained soil mass is then measured by stopping the sieve shaker (Fig. 4.4).

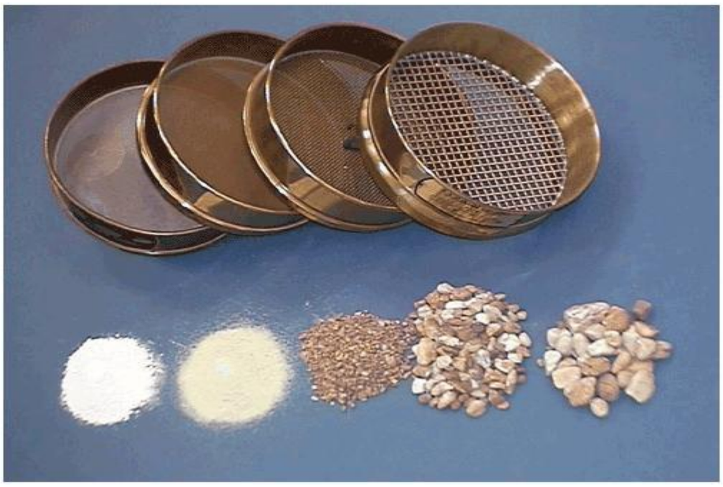

Figure 4.4. Sieve analysis.

4.7.1.2 Equipment

1. The stacked sieves (with a pan and cover)
2. Highly precise electronic weighting machines (0.01 gram precision)
3. The ceramic mortar and pestle (for crushing lumped soil)
4. Binder
5. The sieve shaker

4.7.1.3 Calculations

Outcomes are embodied graphically (percent passing vs sieve size) where sieve size scale is logarithmic. Evaluating retained percent for each sieve is important to finding aggregate percentage passing across every sieve. For this purpose, equation used is:

%Retained= $\frac{w.sieve}{w.total} \times 100$

Here, w.sieve is equal to aggregate weight in sieve, w.total is equal to total aggregate weight.

After this we add total retained aggregate and amount in previous sieves to evaluate cumulative percent of retained aggregate in each sieve, then cumulative percent passing of that aggregate can be obtained by subtracting that retained percent from 100 percent.

Percentage Cumulative Passing = 100% - Percentage Cumulative Retained.

Then the values can be plotted graphically as log sieve size along x axis whereas cumulative percent passing along y axis.

4.7.1.4 Precautions

1. For accurate sieving thorough care of sieves is essential.
2. Keep a check on broken wires and solder breaks and discard such sieves on daily basis.
3. Hot samples are recommended not to be sieved, as it distorts the finer mesh, especially in 10 and 20 numbered sieves.
4. In the duration of rinsing, material loss is avoided by proper technique of transferring of the sample from washing pot to the sieves.
5. Sieves are never to be overloaded.
6. Special care is taken to avoid material loss due to pressure or volume of water while rinsing samples all through the 200 sieve.

4.7.1.5 Limitations

Dry sieving is less exact for finer materials (less than 100 mesh) as decreased grain size there is increased surface attraction effect as required energy making particles pass across an opening increase. When there is matter that is investigated not to be influenced by liquid but to disperse it, then wet sieving is utilized. Finer material transfer more effectively if suspended in appropriate liquid while shaking dry material.

Size of the square opening is assumed to be larger than nearly rounded/spherical particles in account of sieve analysis. For the flat and elongated particles, sieve analysis can’t be yielded reliable. An elongated particle while end-on may pass on through screen however may not pass if presented side-on.

4.7.2 Atterberg Limit Test

Fine grained soil has specific water content measures as shrinkage limits, plastic limits, and liquid limits, these are called Atterberg limits. When a dry clayey soil is treated with increased water amount, distinct changes in behavior and consistency can be observed. Water amount influences soil appearance in four states that includes semi-solid, solid, liquid and plastic. Every state has different properties; therefore, boundaries are defined for changing soil behavior, these boundary limits were founded by a Swedish agriculturist, Albert Atterberg. Later Arthur Casagrande (1958 enhanced it and it started being utilized in soil evaluation for superstructures to be constructed upon. A lot of soils retain water along with volume increase when they are wet and the expansion in volume is dependent on soil ability of taking water in and the physical build-up (atom type present). Silty as well as clayey soils are sensitive to water content and different limits can be evaluated by these tests (Fig. 4.5).

Figure 4.5 Atterberg Limit Test appratus.

1. Liquid Limit.
2. Plastic Limit.

1.7.2.1 Liquid limit

1. The liquid limit (LL) roughly 3/4 of soil is carried and positioned in porcelain dish. Soil is assumed to be air dried and pulverized at this stage, then it's blended with a little distilled water to make a paste, then covered by cellophane to avoid moisture escape.
2. Four empty moisture cans along with their lids are then weighted as well as recorded at data sheet.
3. Height of drop of the cup is examined to correct the apparatus for liquid limit to height rise of 10 mm. Soil properties are established on block at end of grooving tool

on lab assessment of Professor Krishna Reddy, UIC 64 this 10 mm height is referred as gage. Cup should be practiced to approx. 2 times drop per second.

4. When the cup rests on base, formerly mixed soil portion is placed in cup. Then the soil is compressed to eradicate air compartments leading to soil pat forming approx. horizontal surface.
5. For the cutting of a neat straight groove cautiously down to center of cup grooving tool is applied. As groove is being prepared, tool perpendicularity is to be maintained with the cup's surface. It is vital to inhibit Soil sliding is inhibited relatively to cup surface very carefully.
6. Cleaning of soil below cup base area in apparatus and at bottom of the cup is done. After that the drops are counted till two shares of soil pat get in contact to base of the groove along 13mm distance by turning apparatus crank on about two drops per sec. (can be seen in photo D). N (number of drops) if less than 50, then record it on the data sheet otherwise (if greater) then go to step eight directly.
7. Edge to edge sample is taken from soil pat with the help of spatula, then its engineering properties are recorded centered on testing in laboratory by Professor Reddy, UIC mass 65, then placed in oven almost 16 hours and soil is putted in cup placed in porcelain dish. Then tools are again dried and cleaned.
8. Soil samples are remixed in porcelain dish, then added with small amount of distilled water leading to decreased water drop numbers obligatory to close groove.
9. Repetition of steps 6, 7 and 8 is done to continuously less the numbers of drops to close the groove until it comes down to around 15-25 drops. Using the same balance for all weighing and same first laboratory method find out the water content from every trial

4.7.2.1.1 Safety measures

1. The whole equipment should be washed after every examination.
2. Blow counting must only be recorded before the closing of grooves.

4.7.2.2 Plastic Limit

1.Void moisture cans are then weighted along their lids and recorded on statistic sheet.

2.Residual 1/4 of sample is treated with water till its rollable not to be stuck to hands.

3.Then soil is shaped in ellipsoidal mass. Then rolling to glass plate with palm for about 90 stokes per minute and adequate pressure the mass is rolled into a thread of uniform engineering characteristics as per testing in laboratory by Professor Reddy, UIC diameter 66. Then immediately the thread is deformed so that its diameter reaches about 3.2 mm.

4. Then thread is broken into various pieces only to reform it back to ellipsoidal mass by rolling again. This process is repeated till there is cramblage under required pressure of thread so that it can’t be rolled to diameter of 3.2 mm. (can be seen in photo H).
5. Then the crumbled thread portions are collected together and placed in moisture can and cover it up. If soil is less than six grams, then soil from the next trail is added to can (can be seen in step 6). Then instantly it is weighted, mass is recorded, and positioned in oven removing its lid, leaving it for around 16 hours.
6. Repetition of step three, four, and five is performed at least twice. Using the same balance for all weighing and same first laboratory method find out the water content from every trial.

4.7.2.2.1 Instruments

1. Dish for mixing
2. Spatula
3. Glass plate
4. Sieve plate
5. Sieve no.40 with pan

4.7.2.2.2 Precautions

1. Tools are ought to be kept cleaned as soon as every test is performed.
2. The blow quantity must significantly close groove.
3. And blow quantity should be 10-40.

4.7.2.2.3 Limitations

With the Atterberg limits one main limitation is that it will give no indication of residual bonds or particle fabric in particles which may be formed in raw soil however annihilated while preparing sample for these limits’ values.

4.7.2.3 Impact on strength

This test informs us about the soil type i.e., silt or clay and from this test we can derive load bearing capacity and the shear strength of the area.

4.7.2.4 Plasticity Index

Plasticity Index is actually the range of water at which the soil behaves plastically. The plasticity Index is determined as under:

$$PI = LL - PL$$

Where;

PI is the Plasticity index,

LL is the Liquid Limit,

And PL is the Plastic Limit.

4.7.3 In-Situ moisture content (ASTM D-4944)

In-situ moisture content test is a quick, portable and well-established method for evaluation of (fine aggregate) moisture content. The procedure implies the reaction between water and calcium carbide, creating a gas that is directly proportional to the amount of water in the sample as seen (percentage) from pressure gauge.

4.7.4 Natural Moisture Content (ASTM D 2216)

The natural moisture content is the ratio of the weight of water to the weight of the solids in a given mass of soil. This ratio is usually expressed as percentage. In almost all soil tests natural moisture content of the soil is to be determined.

The knowledge of the natural moisture content is essential in all studies of soil mechanics. To sight a few, natural moisture content is used in determining the bearing capacity and settlement. The natural moisture content will give an idea of the state of soil in the field.

4.7.5 Bulk and Dry Density (ASTM D 7263)

This method is utilized to initially find the volume of soil by inserting soil in a steel ring. Given by the formula as under:

Volume = πr^2h

Then dry weight is determined of sample by oven-drying to 105 ° C for 24 hours, bulk density is calculated using equation as under:

ρ = $\frac{w}{v}$

Where;

ρ is the bulk density (g/cm^3),

w is the dry soil weight (g),

v is the soil volume (cm^3).

4.7.6 Direct Shear Test

Direct shear test is used to immediately predict numerous possible difficulties in required engineering design (such as slab bridges, retaining walls, pipes, sheet piling, including angle value of internal friction as well as soil cohesion). Cohesion-free soil values are obtained by lab procedures.

4.7.6.1 Apparatus

Apparatus equipment is shown is the following figure

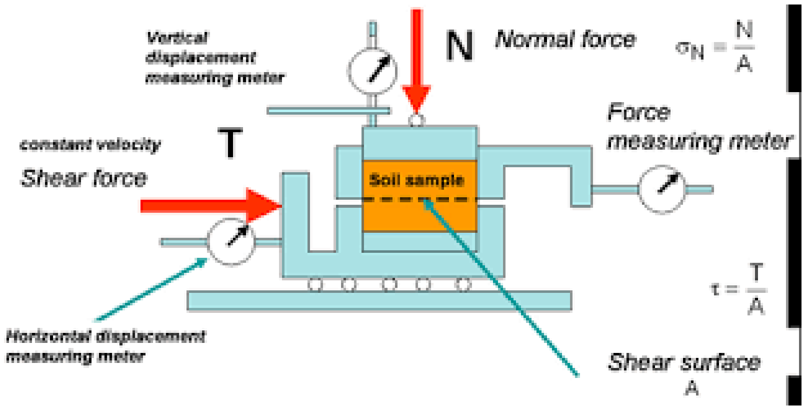

Figure 4.6 Direct Shear Test appratus.

4.7.6.2 General Procedure

1. The interior dimension of soil container is examined individually.
2. Put all the parts of soil container back all together.
3. Container is then weighted, and volume is calculated.
4. Approximately 10 mm layers of soil in smooth out layer are placed if desired dense sample tamp the soil.
5. Soil container is weighted and subtracted from weight of soil, then density of soil is calculated.
6. Soil plane surface is then formed.
7. Place block on top of the soil and position upper grating on stone.
8. Soil sample thickness is then measured.
9. Desired normal load is then applied.
10. Shear pin is then withdrawn.
11. Dial gauge measuring change of volume is then attached to apparatus.
12. Initial calibration value and dial gauge value is then noted.
13. It is made sure that there is no contact in 2 parts except soil/sand prior to starting the test.
14. Motor is started and value of shear force is noted. Volume change values are taken until the failure.
15. 5 kg normal stress 0.5 kg/cm^2 is added up to continue testing until the failure.
16. Every value is noted precisely, dial gauges are set to zero prior to initiating the test.

4.7.6.3 Limitations

The practical is not performed on the undisturbed soil which not according to the code.

4.7.7 Bearing Capacity of Piles

As the subsoil along the alignment of the section, in general, consists of cohesive soil layer consisting of soft to firm silty clay within top 1~8 m followed by very loose to loose silty sand up to 10 m and the load from the bridge structures will be very high, therefore, the shallow foundation may not be feasible to be provided for supporting such heavy structures, hence deep foundation in the form of pile group to support piers will be a feasible solution. Therefore, the following section gives the details of deep foundation including determination of axial pile capacity, group efficiency, horizontal and vertical modulus of subgrade reaction etc.

4.7.8 Axial Capacity of Bored Cast-In-Situ Piles

Depending on <<AASHTO LRFD Bridge Design Specifications>> (2012) 10.7.3.8.6, where a static analysis prediction method is used to determine pile installation criteria, i.e., for bearing resistance, the nominal pile resistance shall be factored at the strength limit state using the resistance factors. Associated with the method used to compute the nominal bearing resistance of the pile. The factored nominal bearing resistance of piles, RR, may be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s$$

In which

$R_p = q_p A_p$

$R_s = q_s A_s$

Where:

R_p = nominal shaft tip resistance (kips)

R_s = nominal shaft side resistance (kips)

φ_{qp} = resistance factor for tip resistance

φ_{qs} = resistance factor for shaft side resistance

q_p = unit tip resistance (ksf)

q_s = unit side resistance (ksf))

A_p = area of shaft tip (ft^2)

A_s = area of shaft side surface (ft^2)

4.7.8.1 Side Resistance (q_S)

The nominal unit side resistance, qs, in ksf, for shafts in cohesive soil loaded under undrained loading conditions by the α-Method shall be taken as:

$q_s = \alpha S_u$

in which:

$\alpha = 0.55$, for $\frac{S_u}{P_a} \le 1.5$, $\alpha = 0.55 - 0.1(S_u / P_a - 1.5)$, for $1.5 \le S_u / P_a \le 2.5$

where:

S_u = undrained shear strength (ksf)

α = adhesion factor (dim)

P_a = atmospheric pressure (=2.12 ksf)

The nominal axial resistance of drilled shafts in cohesionless soils by the β-method shall be taken as:

$q_s = \beta\sigma'_v \le 4.0 \; for \; 0.25 \le \beta \le 1.2$

in which, for sandy soils:

• for $N_{60} \ge 15$:

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

• for $N_{60} < 15$:

$\beta = N_{60}(1.5 - 0.135\sqrt{z})/15$

4.7.8.2 Tip Resistance (q_p)

For axially loaded shafts in cohesive soil, the nominal unit tip resistance, q_p, by the total stress method as provided in O'Neill and Reese (1999) shall be taken as:

$q_p = N_c S_u \le 80.0$

in which:

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

Where:

D=diameter of drilled shaft (ft)

Z= penetration of shaft (ft)

S_u= undrained shear strength (ksf)

The nominal tip resistance, qp, in ksf, for drilled shafts in cohesionless soils by the O'Neill and Reese (1999) method shall be taken as:

for N60≤50, q_p=1.2N_{60}

where:

N_{60} = average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

Cohesion less soils with SPT-N60 blow counts greater than 50 shall be treated as intermediate geomaterial (IGM) and the tip resistance, in ksf, taken as:

$$q_p = 0.59\left[N60(\frac{p_a}{\sigma'_v})\right]^{0.8} \sigma'_v$$

where:

p_a = atmospheric pressure (=2.12ksf)

σ'_v = vertical effective stress at the tip elevation of the shaft (ksf).

Using the procedure above, the pile capacity values for 760mm, 900 mm, 1000 mm, 1200 mm and 1500mm diameter piles have been calculated. The following table and graph present the allowable pile capacities separately for all boring locations with length of pile for different diameters, whereas the pile capacity calculation sheets are appended with the report.

Chapter 5

RESULTS AND DISCUSSION

The analysis of study area was carried out by the methods as under:

1. 2 boreholes up to 36 meter depth were drilled using percussion drilling machine with 100 mm diameter.
2. In-situ testing was conducted.
3. Disturbed samples were collected.
4. Laboratory Analysis.
5. Geotechnical investigation report interpretation.

Subsurface strata was studied by obtained samples on field and by borehole logs. Concluded summary table, borehole logs and moisture content findings are provided (in annexure).

Water table was not found in our drilled boreholes. 14 was the minimum SPT value encountered. The natural moisture content ranged from 7.8 - 9.7 %. The dry density ranged from 1.889 - 2.010 gm/cc. The gravel ranged from 0.0-30.0% according to grain size analysis. The sand ranged from 8.6% - 12.0% whereas the silty clay ranged from 20.0% - 95.0%. The liquid limit ranged from 25% - 30%. The plastic limit ranged from 19% - 23%. The values for the plasticity index varied from 5 % - 10 %.

5.1 Encountered Strata

Two boreholes was drilled at purposed site. The site is mainly consisted of overburden soil Brown to dark, firm to staff silty clay/ clayey silt, low to medium plasticity, low to medium dry strength followed with a massive alternate bed of gravels intermixed with silty clay. Detail of strata encountered in each borehole is given in borehole logs listed below in annexure, it can be seen.

5.2 Sieve analysis

Sieve analysis was performed on Six (06) pulverized onsite overburden soil samples as per ASTM D 422.

These test results indicated that the onsite overburden soils generally comprise of Silty Clay (CL-ML) and (GW) groups as per Unified Soil Classification System (USCS). However, the overburden soil belongs to A-7-6 and A-5 groups as per

AASHTO soil classification. The percentage of fines (passing sieve no. 200) varies from 70.5 to 91.4 %.

5.2.1 Bore Hole-1

Table 5.1. Sieve analysis BH-1 from 5m depth.

Project:	Faqir Api Flyover							
BH No:	1							
Depth ft:	5m							
Symb	**Bore No.**	**Sample**	**Depth (m)**	**Group Name**	**Gravel %**	**Sand %**	**Fines -200 %**	**Group Symbol**
·	1	DS	5	Silty Clay	0	8.2	91.8	CL-ML

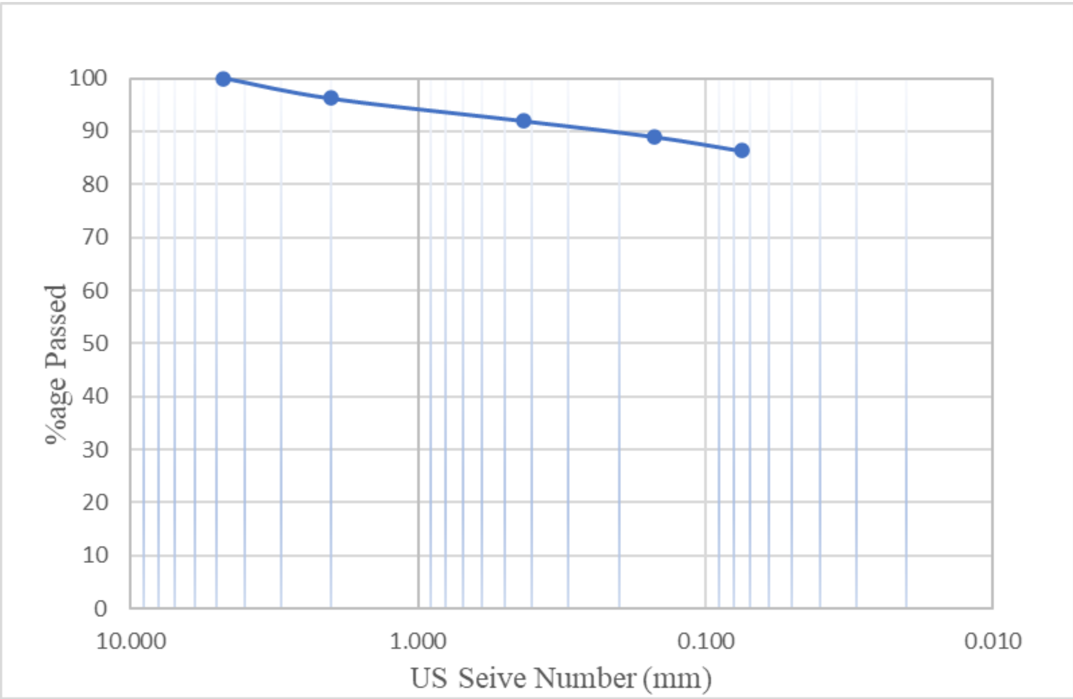

Figure 5.1 Sieve analysis BH-1, Depth 5m

5.2.2 Borehole-1

Table 5.2. Sieve analysis BH-1 from 35m depth.

Project:	Faqir Api Flyover							
BH No:	1							
Depth ft:	35m							
Symb	**Bore No.**	**Sample**	**Depth (m)**	**Group Name**	**Gravel**	**Sand**	**Fines**	**Group**
					%	**%**	**-200%**	**Symbol**
•	1	DS	35	Silty Clay	0	8.8	91.2	CL-ML

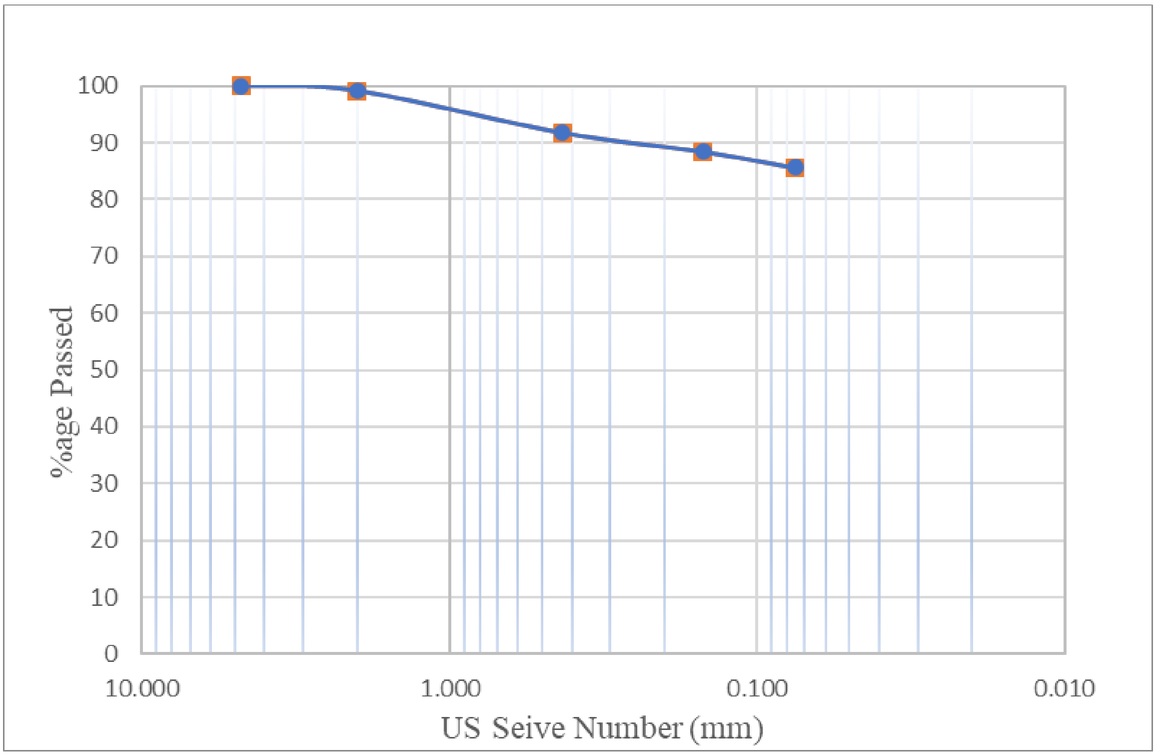

Figure 5.2 Sieve analysis BH-1, Depth 35m

5.2.3 Bore Hole-2

Table 5.3. Sieve analysis BH-2 from 15m depth

Project:	Faqir Api Flyover							
BH No:	2							
Depth ft:	15m							
Symb	**Bore No.**	**Sample**	**Depth (ft)**	**Group Name**	**Gravel**	**Sand**	**Fines**	**Group**
					%	**%**	**-200 %**	**Symbol**
·	1	DS	15	SILTY CLAY	2	7.9	92.1	CL-ML

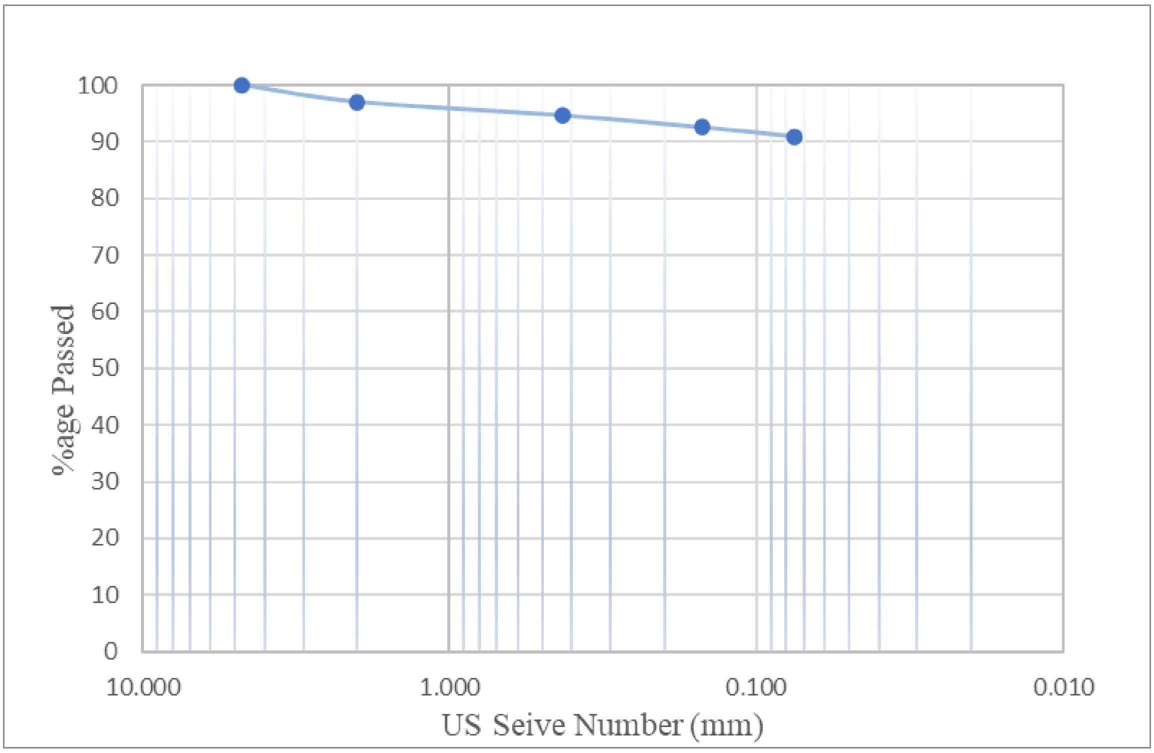

Figure 5.3 Sieve analysis BH-2, Depth 15m.

5.2.4 Bore Hole-2

Table 5.4. Sieve analysis BH-2 from 36m depth.

Project:	Faqir Api Flyover							
BH No:	2							
Depth ft:	36m							
Symb	**Bore No.**	**Sample**	**Depth (ft)**	**Group Name**	**Gravel**	**Sand**	**Fines**	**Group**
					%	**%**	**-200 %**	**Symbol**
·	1	DS	36	SILTY CLAY	2	8.4	91.6	CL-ML

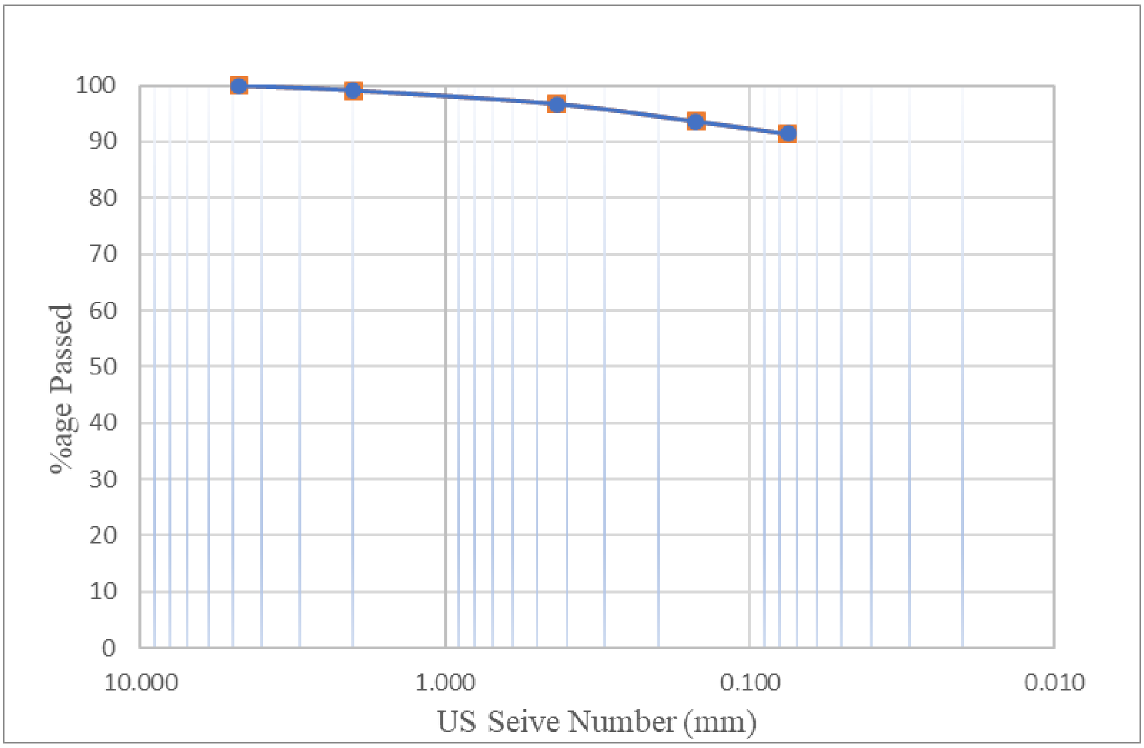

Figure 5.4 Sieve analysis BH-2, Depth 36m.

Atterberg Limit Test

Atterberg limit test was performed on Six (06) onsite overburden soil samples as per ASTM D-4318 to check the index properties of soil. Liquid limit range 25 % - 30 % is indicating overburden soil samples along with the plasticity index 5 % - 10 %.

5.3.1 Bore Hole-1

Table 5.5. Liquid limit test of BH-1.

Liquid Limit Test (AASHTO T89,93)			
Bore Hole -1	**Depth 25m**		
CAN NO	1	2	3
WEIGHT OF WET SOIL+CAN (GMS)	25.5	27	25
WEIGHT OF DRY SOIL+CAN (GMS)	21.9	22	20
WEIGHT OF WATER (GMS)	3.6	5	5
WEIGHT OF CAN (GMS)	7.4	8.2	7.6
WEIGHT OF DRY SOIL (GMS)	14.5	13.8	12.4
% MOISTURE CONTENT	27	30	35
NO OF Blows	38.0	26	18

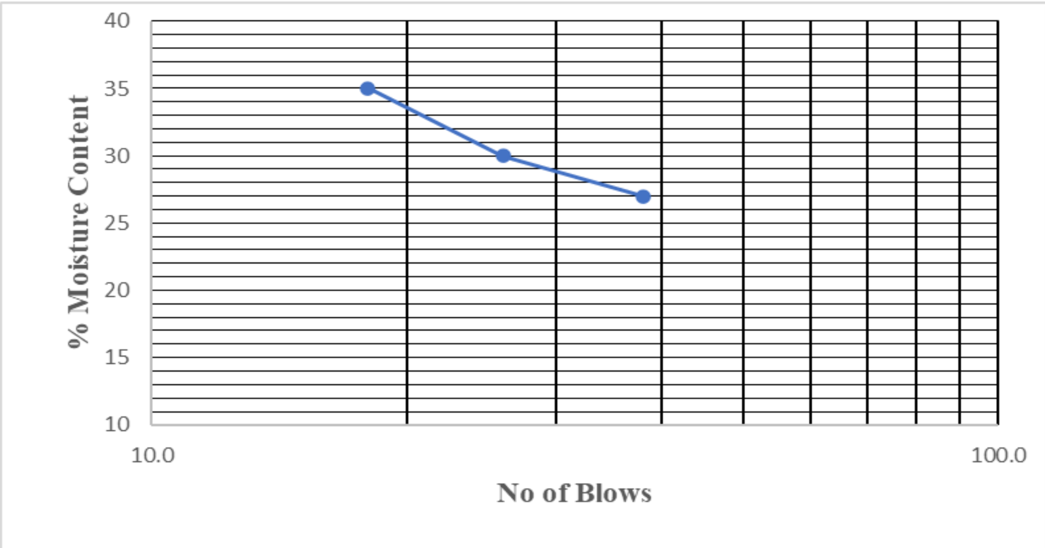

Figure 5.5 Atterberg limit BH-1, Depth 25m.

Figure 5.6. Plastic limit test.

PLASTIC LIMIT (AASHTO T90-92)			
Bore hole -1	**Depth 25m**		
CAN NO	**4**	**5**	**AV.**
WEIGHT OF WET SOIL + CAN (GMS)	14	15.1	
WEIGHT OF DRY SOIL + CAN (GMS)	13.3	14.5	
WEIGHT OF WATER (GMS)	0.7	0.6	
WEIGHT OF CAN (GMS)	9.3	8.7	
WEIGHT OF DRY SOIL (GMS)	4	5.8	
% MOISTURE CONTENT	17.5	10.3	13.9
LL= 27 **PL**=20 **PI=** 7.2			

5.3.2 Borehole-2

Table 5.7. Liquid limit test of BH-2.

Liquid Limit Test (AASHTO T89,93)			
Bore Hole-2	**Depth 35m**		
CAN NO	1	2	3
WEIGHT OF WET SOIL+CAN (GMS)	23.5	24.9	23.5
WEIGHT OF DRY SOIL+CAN (GMS)	20.4	21.2	19.4
WEIGHT OF WATER (GMS)	3.1	3.7	4.1
WEIGHT OF CAN (GMS)	7.4	8.2	7.6
WEIGHT OF DRY SOIL (GMS)	13.0	13	11.8
% MOISTURE CONTENT	23.8	28.5	34.7
NO OF Blows	33.0	24	19

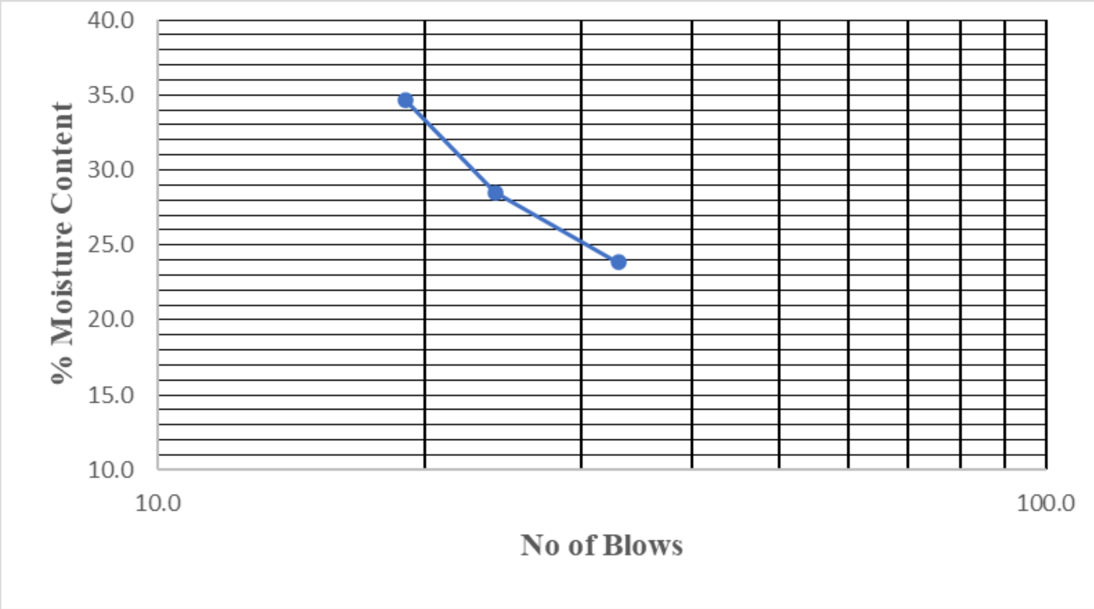

Figure 5.6 Atterberg limit BH-2, Depth 35m.

Plastic Limit Test

Table 5.8. Liquid limit test BH-2.

PLASTIC LIMIT (AASHTO T90-92)			
BH-2	**Depth 35m**		
CAN NO	**4**	**5**	**AV.**
WEIGHT OF WET SOIL + CAN (GMS)	15.2	16.7	
WEIGHT OF DRY SOIL + CAN (GMS)	14.3	15.3	
WEIGHT OF WATER (GMS)	0.9	1.4	
WEIGHT OF CAN (GMS)	9.7	8.3	
WEIGHT OF DRY SOIL (GMS)	4.6	7	
% MOISTURE CONTENT	19.56522	20.0	19.8
LL= 29 **PL= 20** **PI**= 9			

5.4 Unconfined Compressional Test

Compressive unconfined strength examination was performed on five (5) relatively undisturbed cohesive soil samples extracted from boreholes. Test results indicated that the unconfined compressive strength of soil samples varies from 1.659 to 4.407 TSF.

Table 5.9. Unconfined compression test.

Strain Dial Reading	Dial Reading	Load.P (lbs)	Vertical Dial difference Δ L (inch)	Unit Strain ε	1-ε	Corrected Area (ft 2)	Unit stress (lb/sq.ft)	Stress (tons/ft^2)	Unit Stress (lb/sq.ft)
0	0	0	0	0	1	0	0	0	0
20	0.8	3	0.020	0.007	0.993	0.0124	512.3	0.229	256.15
40	1.6	6	0.040	0.013	0.987	0.0125	987.5	0.441	493.75
60	2.7	10	0.060	0.02	0.980	0.0126	1646.5	0.735	823.1
80	3.5	13	0.080	0.027	0.973	0.0126	2114.7	0.944	1057.35
100	4.5	17	0.100	0.033	0.967	0.0127	2678.9	1.196	1339.45
120	6.4	24	0.120	0.040	0.96	0.0128	3788.1	1.691	1894.05
140	7.9	30	0.140	0.047	0.953	0.0129	4661.9	2.081	2330.95
160	9.6	37	0.160	0.053	0.947	0.0130	5641.9	2.519	2820.95
180	11	42	0.180	0.06	0.94	0.0131	6412.2	2.863	3206.1
200	12.5	48	0.200	0.067	0.933	0.0132	7226.9	3.226	3613.45
220	14.7	56	0.220	0.073	0.927	0.0133	8425.8	3.762	4212.9
240	13.8	52	0.240	0.08	0.92	0.0134	7846.9	3.503	3923.45

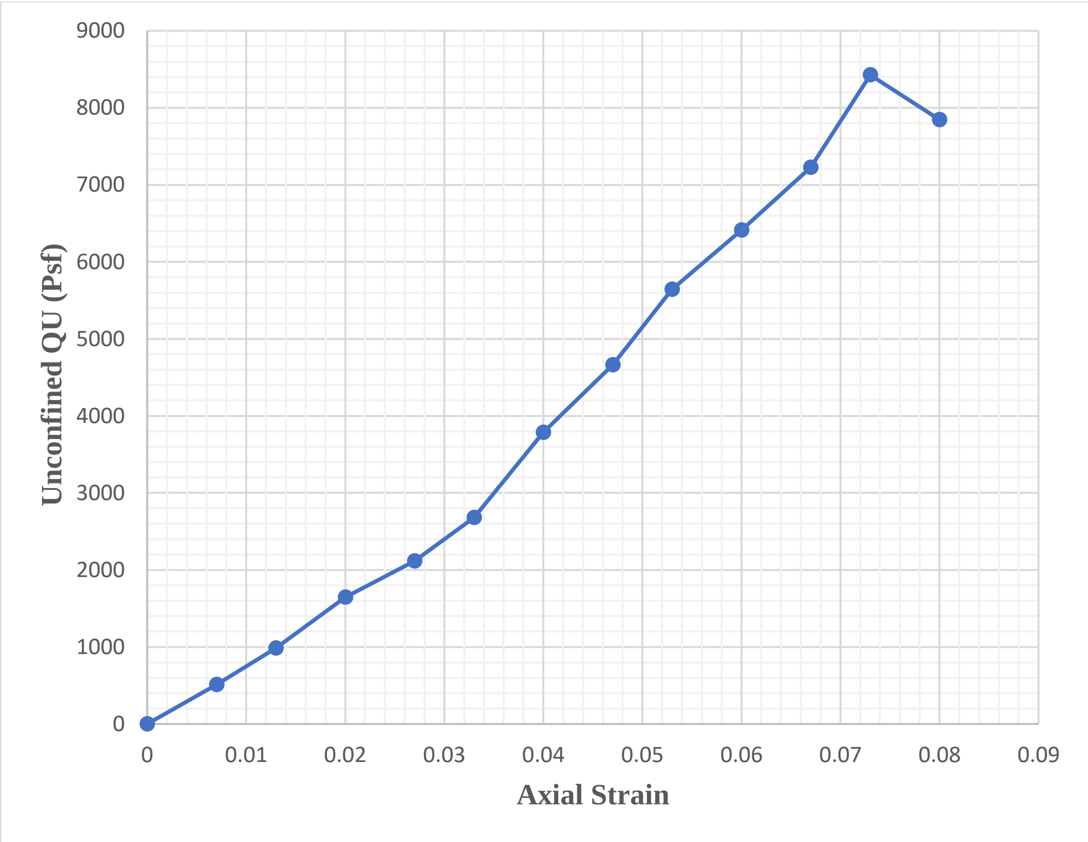

Figure 5.7 Unconfined compression.

Unconfined comp Strength Qu 8425.8psf 3.762tsf

Undrained Shear Strength Su 4212.90psf 1.881tsf

5.5 Direct Shear Test

Table 5.10. Direct shear test.

Vertical Stress (lbs)	Weight of hanger (lbs)	Net Applied loads (lbs)	Ring Dial reading (Div)	Normal stress (psf)	Shear stress (psf)
132	8	8	14	196	877
132	8	16	22	392	1325
132	8	32	31	784	1885

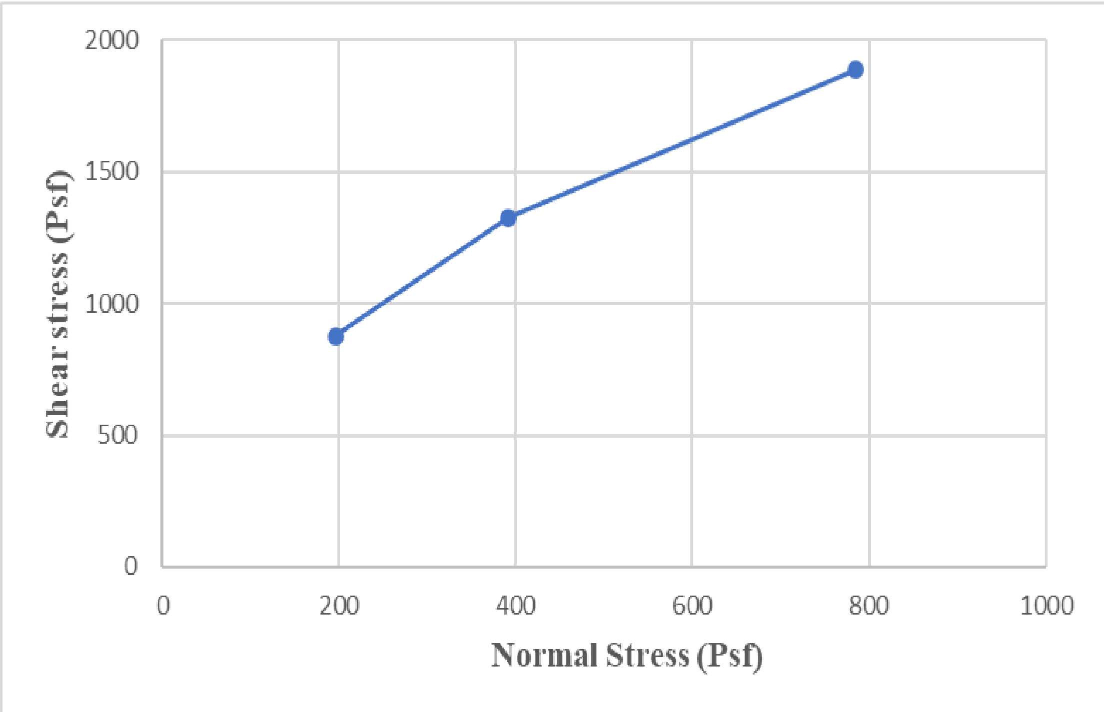

Figure 5.8 Direct shear test

Angle of internal friction ϕ = 38°

Undrained Cohession (psf) = 626

5.6 Bearing Capacity of Piles

Depending on <<AASHTO LRFD Bridge Design Specifications>> (2012), where a static analysis prediction method is used to determine pile installation criteria, i.e., for bearing resistance, the nominal pile resistance shall be factored at the strength limit state using the resistance factors associated with the method used to compute the nominal bearing resistance of the pile.

Table 5.11. Bearing capacity of piles.

Depth	Pile Dia = 0.76m	Pile Dia = 0.90m	Pile Dia = 1.0m	Pile Dia = 1.2m	Pile Dia = 1.5m
1.0	0	0	0	0	0
2.0	11	15	17	23	33
3.0	14	18	22	30	41
4.0	19	25	29	39	57
5.0	25	32	38	50	71
6.0	34	43	50	66	95
7.0	39	49	57	75	105
8.0	44	56	64	83	116
9.0	50	62	71	92	126
10.0	55	68	78	100	136
11.0	60	74	85	108	147
12.0	65	81	92	117	157
13.0	71	87	99	125	168
14.0	76	93	106	134	178
15.0	81	100	113	142	189
16.0	100	124	142	180	245
17.0	109	134	154	194	262
18.0	118	145	165	208	280
19.0	127	155	177	222	297
20.0	136	166	188	236	315
21.0	144	176	200	250	332
22.0	153	187	212	264	350
23.0	162	197	223	278	367
24.0	171	208	235	292	384
25.0	180	218	247	306	402
26.0	189	229	258	320	419
27.0	198	239	270	334	437
28.0	206	250	282	348	454
29.0	215	260	293	362	472
30.0	224	271	305	376	489
31.0	233	281	317	390	507
32.0	242	292	328	404	524

Depth	Pile Dia = 0.76m	Pile Dia = 0.90m	Pile Dia = 1.0m	Pile Dia = 1.2m	Pile Dia = 1.5m
33.0	251	302	340	418	542
34.0	260	313	352	432	559
35.0	268	323	363	446	577
36.0	277	334	375	460	594

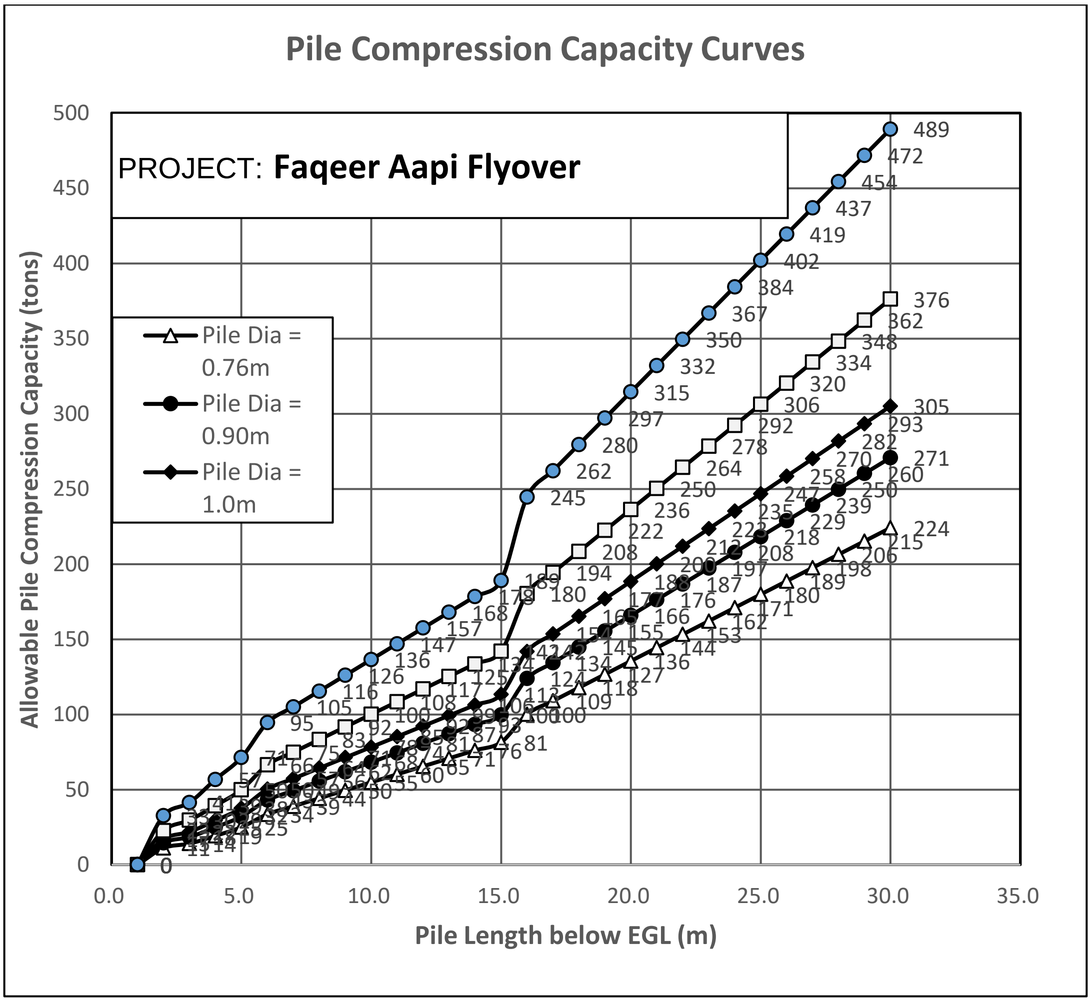

Figure 5.9 AASHTO LRFD Bridge Design Specifications (2012),

CONCLUSIONS

1. In both boreholes that were drilled, the water table was not found. Average Minimum and Maximum SPT values encountered in both bores were 13 and 50 respectively. Moisture content ranges from 7.8 to 9.7 %. In the both bore holes the major lithology encountered was silty clay (CL-ML) and clay (CL) was also found in some localities. Allowable Bearing Capacity for Pile is measured from 0 to 30 meter against 5 different diameters i.e. 0.75m, 0.90m, 1.0m, 1.2m and 1.5m. The diameter and length of the pile found to be 1.2m and 25m to achieve the necessary allowable load i.e.306tons.
2. Pile foundation is the most suitable foundation for the site location, to compensate for the dead load of the heavy structure.

RECOMMENDATIONS

1. Before constructing working piles, pile load testing must be done.
2. The safety of workers and adjacent structures during all construction activities should be assured by adequate measures.

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ANNEXURES

Bore Hole log -1

SR No	Depth (Meter)	SPT/CPT	Classification	Material Description	Graphic Log	Density	SPT / N Value
1	1.0	SPT	CL-ML	Silty clay with brownish colored		-	32
2	2.0	SPT	CL	Silty clay with brownish colored		-	29
3	3.0	SPT	CL	Clay with gravel light brown in colored		-	40
4	4.0	SPT	CL-ML	Clay with gravel light brown in colored		-	15
5	5.0	SPT	CL-ML	Reddish brownish silty clay		-	13
6	6.0	SPT	CL-ML	Silty clay with reddish coloured		-	20
7	7.0	SPT	CL-ML	Silty clay with reddish coloured		-	35
8	8.0	SPT	CL-ML	Reddish brownish silty clay		-	45
9	9.0	SPT	CL-ML	Reddish brownish silty clay		-	42
10	10.0	SPT	CL-ML	Reddish brownish silty clay		1.888	15
11	11.0	SPT	CL-ML	Reddish brownish silty clay		-	33
12	12.0	SPT	CL-ML	Silty clay with reddish brownish in coloured		-	28
13	13.0	SPT	CL-ML	silty clay brownish in coloured		-	32
14	14.0	SPT	CL-ML	silty clay brownish in coloured		-	25
15	15.0	SPT	CL-ML	silty clay brownish in coloured		-	15
16	16.0	SPT	CL-ML	Silty clay brown in colour		-	33
17	17.0	SPT	CL-ML	Silty clay brown in colour		2.005	49
18	18.0	SPT	CL-ML	Slity clay reddish to brownish in coloured		-	35
19	19.0	SPT	CL-ML	Silty clay light brown in coloured		1.957	26
20	20.0	SPT	CL-ML	Silty clay light brown in coloured		-	21
21	21.0	CPT	GL/GW	Clay with concretion & gravels reddish in coloured		-	R
22	22.0	CPT	CL/GW	Silty sand fine in nature clay & gravel to brownish in coloured		-	R
23	23.0	CPT	SM/GW	Silty sand fine in nature clay & gravel to brownish in coloured		-	R
24	24.0	SPT	CL-ML	Silty caly with concretion to reddish coloured		-	30
25	25.0	SPT	CS	Silty caly with concretion to reddish coloured		1.961	30
26	26.0	SPT	CS	Sandy clay with concretions reddish in colour		-	40
27	27.0	CPT	CS	Sandy clay reddish in coloured		-	R
28	28.0	CPT	CL/ML	Silty clay with concretion & gravels		1.986	33
29	29.0	CPT	CL	Clay with boulder coarse sand reddish in colour		-	R
30	30.0	SPT	CL-ML	silty clay reddish in colour		-	R
31	31.0	SPT	CL	silty clay reddish in colour		-	R
32	32.0	SPT	CL	Clay reddish to brownish in colour		-	40
33	33.0	SPT	CL	Clay light brownish in colour		2.09	46
34	34.0	SPT	CL	Clay brownish in colour		-	50
35	35.0	SPT	CL-ML	silty clay reddish in colour		-	38

Bore Hole log -2

SR No	Depth (Meter)	SPT/CPT	Classification	Material Description	Graphic Log	Density	SPT / N Value
1	1.0	SPT	CL-ML	Silty clay with brownish colored		-	30
2	2.0	SPT	CL	Clay with concentration in light grey colored		-	28
3	3.0	SPT	CL	Clay with gravel light brown in colored		-	39
4	4.0	SPT	CL-ML	Silty clay with concentration greyish in colored		-	18
5	5.0	SPT	CL-ML	Reddish brownish silty clay		-	17
6	6.0	SPT	CL-ML	Silty clay with reddish coloured		-	34
7	7.0	SPT	CL-ML	Silty clay with reddish coloured		-	R
8	8.0	SPT	CL-ML	Reddish brownish silty clay		-	R
9	9.0	SPT	CL-ML	Reddish brownish silty clay		-	R
10	10.0	SPT	CL-ML	Reddish brownish silty clay		1.889	14
11	11.0	SPT	CL-ML	Reddish brownish silty clay		-	26
12	12.0	SPT	CL-ML	Silty clay with reddish brownish in coloured		-	28
13	13.0	SPT	CL-ML	silty clay brownish in coloured		-	38
14	14.0	SPT	CL-ML	silty clay brownish in coloured		-	16
15	15.0	SPT	CL-ML	silty clay brownish in coloured		-	15
16	16.0	SPT	CL-ML	Silty clay brown in colour		-	R
17	17.0	SPT	CL-ML	Silty clay brown in colour		2.005	R
18	18.0	SPT	CL-ML	Slity clay reddish to brownish in coloured		-	37
19	19.0	SPT	CL-ML	Silty clay light brown in coloured		1.957	25
20	20.0	SPT	CL-ML	Silty clay light brown in coloured		-	21
21	21.0	CPT	GL/GW	Clay with concretion & gravels reddish in coloured		-	R
22	22.0	CPT	CL/GW	Clay with boulder & concretions		-	R
23	23.0	CPT	SM/GW	Silty sand fine in nature clay & gravel to brownish in coloured		-	R
24	24.0	SPT	CL-ML	Silty caly with concretion to reddish coloured		-	35
25	25.0	SPT	CS	Sandy clay reddish in coloured		1.962	34
26	26.0	SPT	CS	Sandy clay with concretions reddish in colour		-	39
27	27.0	CPT	CS	Sandy clay reddish in coloured		-	R
28	28.0	CPT	CL/ML	Silty clay with concretion & gravels		1.986	36
29	29.0	CPT	CL	Clay with boulder coarse sand reddish in colour		-	R
30	30.0	SPT	CL-ML	silty clay reddish in colour		-	R
31	31.0	SPT	CL	Clay with sand reddish to brownish in colour		-	R
32	32.0	SPT	CL	Clay reddish to brownish in colour		-	44
33	33.0	SPT	CL	Clay light brownish in colour		2.01	46
34	34.0	SPT	CL	Clay brownish in colour		-	R
35	35.0	SPT	CL-ML	silty clay reddish in colour		-	37
36	36.0	SPT	CL-ML	Silty clay reddish to brownish in colour		-	R

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