ALLOWABLE BEARING PRESSURE ANALYSIS FOR SHALLOW FOUNDATION OF KHYBER INSTITUTE OF NEURO SCIENCES & CLINICAL RESEARCH, MARDAN, KPK, PAKISTAN

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ABSTRACT

This study is carried out to perform the bearing capacity analysis for Construction of Khyber Institute of Neuro Sciences & Clinical Research at Mardan using ASTM standards. The subsoil investigation program was conducted by drilling seven (07) boreholes at different depth and location of the site. Light percussion drilling method was adopted and boreholes were drilled up to maximum depth 100.0 feet. Samples were collected, in-situ testing and analysis were done for assessment of soil strength. Grain size analysis (ASTM C-136), Atterberg Limits (ASTM D-4318), Unconfined Compression Test (ASTM D-2166), Direct Shear Test (ASTM D-3080) were conducted to interpret the geotechnical behavior of the sub-soil. Bearing capacity was calculated using Meyerhof's equation (1976) for shallow foundation. On the basis of field and lab test results bearing capacity was calculated and recommendations are made.

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ABBREVIATIONS

ASTM	The American Society of Testing Materials
BH	Bore Hole Logs
CL	Lean Clay
DS	Direct shear
DS	Disturbed Soil Sample
EGL	Existing Ground Level
GC	Clayey Gravels
GW	Under Ground Water
Bmi	In-situ Moisture Content
PMD	Pakistan Meteorological Department
UCS	Unconfined Compressive Strength
UDS	Undisturbed Soil Sample
USGS	United States of Geological Survey
USCS	Unified Soil Classification System
SPT	Standard Penetration Test
Qp	Point Bearing Capacity
Qu	Ultimate Bearing Capacity

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CHAPTER 1
INTRODUCTION

1.1 Foundations

All engineering constructions resting on the earth must be carried by some kind of interfacing element called a foundation. The foundation is part of an engineering system that transmits to and into the underlying soil or rock, the loads supported by the foundation and its self weight. The resulting soil stress except at the ground surface are in addition to those presently existing in the earth mass from its self weight and geological history.

The term superstructure is commonly used to describe the engineered part of the system bringing load to the foundation, or substructure. The term superstructure has particular significance for buildings and bridges a foundation may only carry machinery and support industrial equipment such as pipes, towers, tanks. For these reasons it is better to describe a foundation as that part of the engineered system that interfaces the load-carrying components to the ground. It is evident on the basis of this definition that a foundation is the most important part of the engineering system.

1.2 Foundation types

The foundation of a structure is in direct contact with the ground and transmits the loads of structure to the ground. Foundations may be characterized as Shallow (Pad, Strip or Raft) and Deep (Pile, Piers or caissons). Design of foundation is mainly dependent on two principal criteria and they are Bearing Capacity and Settlement.

Bearing capacity is the adequate factor of safety against collapse. While settlement is the working load must not cause damage nor adversely affect the serviceability of structure (Das, 2007)

1.2.1 Shallow foundation

Shallow foundations are those that transmit the structural loads to the near surface soil or rock. They are founded near to the finish ground surface generally where the founding depth is less than the width of footing and less than 3 meter. Shallow foundation includes

(1) Strip footing

(2) Spread or Isolated footing

(3) Mat or raft foundation

They are used when surface soils are sufficiently strong and stiff to support the superstructure they are generally unsuitable in weak and highly compressible soils such as poorly compacted fill, peat, recent lake and alluvial deposits.

1.2.1.1 Strip footing

Strip foundations are used where soil is of good bearing capacity. The size and position of strip is directly related to overall width of wall. The load is transmitted at 45˚ from base of the wall to the soil. The depth of strip foundation must be equal or greater then overall width of the wall and the width of foundation must be three times width of supported wall. Strip foundation is strengthened by inclusion of steel reinforcement.

1.2.1.2 Spread or isolated footing

A spread footing foundation is an enlargement at the bottom of the column or a bearing wall that spreads the structural load over a certain area of soil. They are mainly made up of reinforced concrete. The required footing size depends on the magnitude of load, engineering properties of underlying soil and other factors.

1.2.1.3 Mat or raft footing

This is essentially one large spread footing that encompasses the entire structure they spread the weight of the structure across a larger area, thus reducing the induced stresses in the underlying soil. Mat foundation also has advantage of structural continuity and thus reduces the potential for differential settlements. This foundation is used for structures usually too heavy for spread footings but not heavy enough for deep foundation system.

1.3 Foundation engineering

Due to heterogeneous nature of soil and rock mass two foundations even on adjacent construction sites will seldom be the same except by coincidence. The amalgamation of experience study of what others have done in somewhat similar situations, and the site specific geotechnical information to produce an economical, practical and safe substructure design is called foundation engineering (Bowles, 1988). Following steps are minimum required for designing a foundation:

(1) Locate site and position of load. A rough estimate is provided by client. Depending upon site and position of load and complexity a literature survey is carried out to check how others have solved similar problems.

(2) Site visit for geological and other potential evidences that may indicate potential design problems that will be taken in account before design recommendation. This inspection can be supplemented by any previous soil data of location.

(3) Establish field exploration program and on the basis of discovery set up necessary field testing and laboratory survey.

(4) Determine necessary soil design parameters based on test data, scientific principles and engineering judgment. Simple or complex computer analysis may be involved.

(5) Design the foundation using parameters form step 4. Foundation should be economical and constructible in given resources and manpower. Take into account all the local construction practices and practical construction tolerance. Interact loosely with all the concerned so that substructure is not overdesigned and risk is kept within acceptable levels.

1.4 Seismicity of area

Seismic zoning of Pakistan have been developed by Pakistan Meteorological Department in 1998 by earthquake data collected from United States of Geological Survey (USGS) and International Seismological Center (ISC). For this purpose the magnitude range for seismicity map was taken as of 4.5 on Richter scale.

The region of Pakistan is divided into four major seismic zones i.e. 1,2,3,4 in term of major, moderate, minor and negligible. Zone 2 is further divided into 2 parts i.e. 2A and 2B. Project area lies in zone 2B. According to the PDE catalogue this zone seems to be very active, especially parts of KPK. One of the important aspects of this zone is that it includes provincial capital Peshawar, the country capital Islamabad and one densely populated city of India, Amritsar. The other important cities in this zone are Malakand and Mardan. According to this historical PMD database, there is a belt of uniform seismic activity which exists in this zone. This belt starts near Rawalpindi and ends near Peshawar. In addition to the geological condition of the area, it is thought that one of the reasons for many earthquake reports from this area is that it has remained densely populated since ancient times.

1.5 Location of study area

The project area is located at Mardan. The site is for the construction of Khyber Institute of Neurosciences and Clinical Research. Seven (07) bore holes of different depth were planned at different locations of the site for soil investigation. Light percussion drilling method was adopted and drilled the holes up to maximum depth of 100 feet. There is no hard and fast rule for borehole spacing. Spacing can be increased or decreased, depending upon the sub-soil condition and nature of structure. If various soil strata are more or less uniform and unpredictable, fewer boreholes are needed to explore sub-soil strata visualizing expected soil behavior with moisture under building load. Structure engineer's recommendation for deciding no. of boreholes and their location should be weighted for valuable sub-soil information. As a rule of thumb, the test bore hole should be placed at distance of 50-100 feet. Disturbed and undisturbed soil samples from the boreholes were collected and tested in the laboratory for the geotechnical design assessment. The location of study area in Mardan district is shown in figure 1.1.

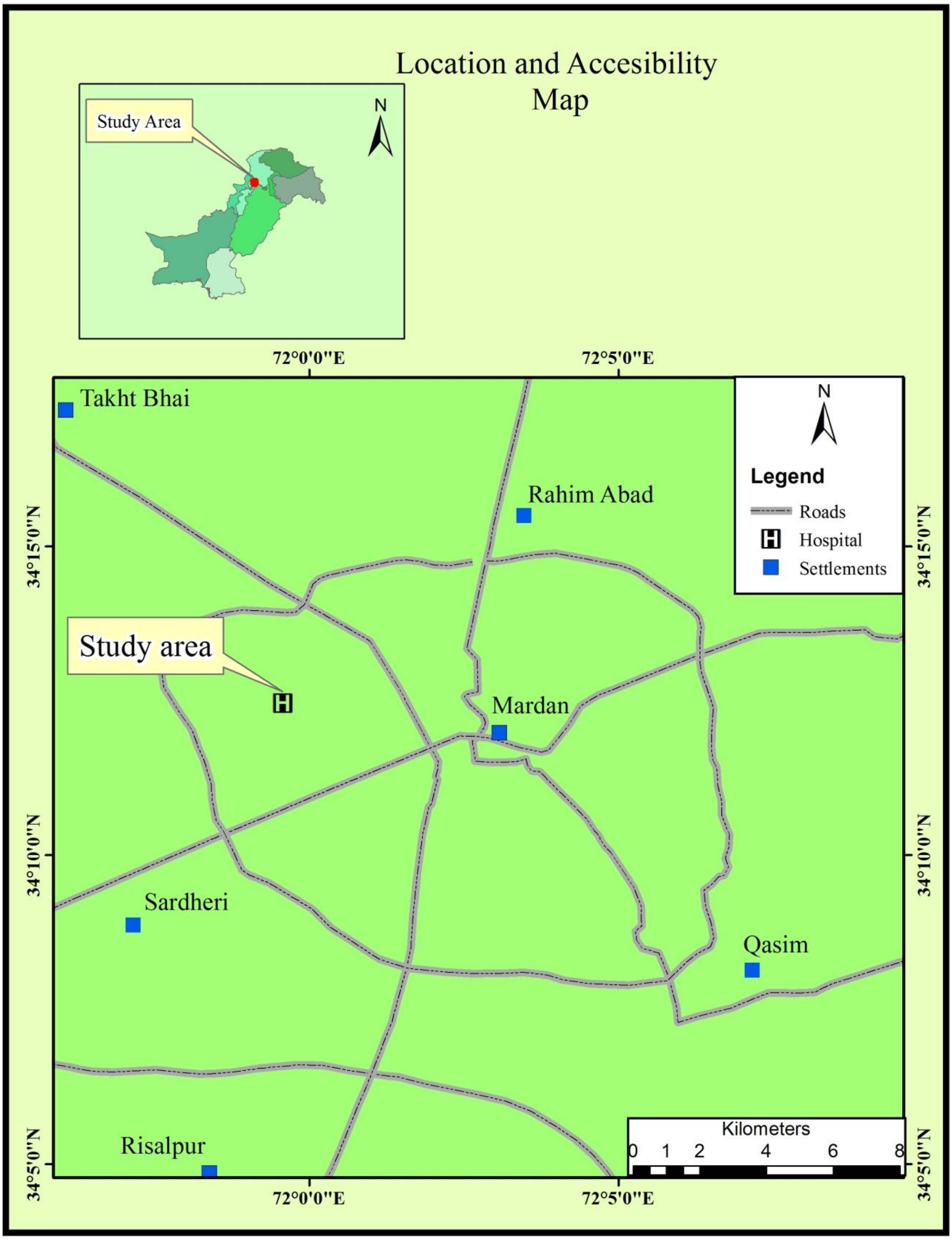

Figure 1.1. Location of study area (Generated using Arc GIS, Google Earth).

1.6 Objectives

The objectives of the research are:

(1) To discuss and evaluate engineering parameters based on field and laboratory testing of foundation soil.

(2) To select most appropriate economical foundation and its proposed depth on sub-soil strata and building requirements.

(3) To deduce different geotechnical solutions by keeping in view the behavior and characteristics of foundation soil.

(4) To determine dewatering technique in case of water table encountered in footing excavation zone.

(5) To calculate bearing capacity of shallow foundation.

(6) Recommendation about net allowable bearing pressure for the proposed foundation level.

1.7 Methodology

The work flow of our research activity is shown in figure 1.2 as below.

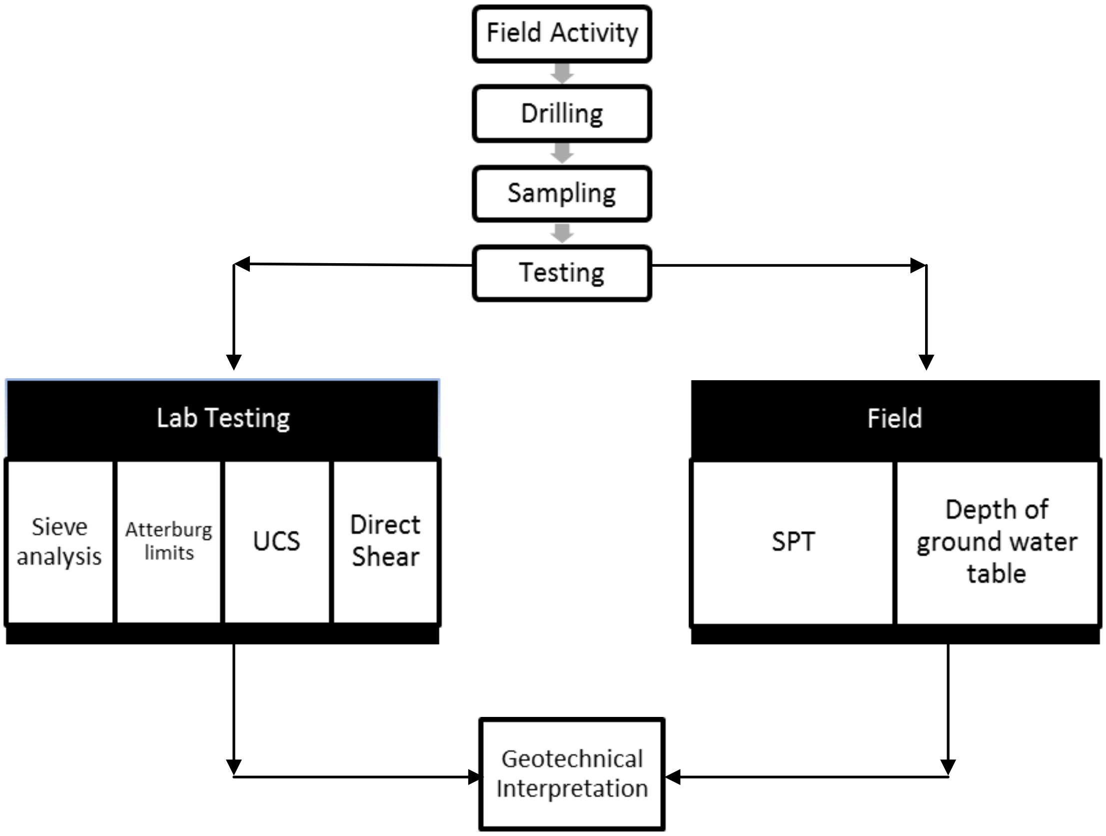

Figure 1.2. Flow chart of methodology.

Field was conducted for reconnaissance survey of the site which was followed by drilling of 7 boreholes. Samples were collected as per requirement for both in-situ and laboratory testing to analysis sub soil strata. In-situ testing was done by SPT which gave both the soil cohesiveness and ground water table depth.

Similarly, Sieve analysis, Atterburg limits, UCS and Direct shear was done for gradation, liquid and plastic limits and UCS and Direct Shear was done to know the compressive strength of soil.

CHAPTER 2
LITERATURE REVIEW

2.1 Historical overview

Geology is science of rocks, mineral, soil and subsurface water, including the study of their formation, structure and behavior. Engineering geology is the branch that deals with application of geological principal to engineering works. Engineering geologist have background of Geology their work includes mapping, describing and characterizing rock at a construction site, accessing stability issues such as landslides, local seismicity and earthquake potential.

It is the application of the geological sciences to engineering study for the purpose of assuring that the geological factors regarding the location, design, construction, operation and maintenance of engineering works are recognized and accounted for. Engineering provide geological and geotechnical recommendations, analysis, and design associated with human development and various types of structures. The realm of the engineering geologist is essentially in the area of earth-structure interactions, or investigation of how the earth or earth processes impact human made structures and human activities (Charlie et al, 1984).

Engineering geology studies may be performed during the planning, environmental impact analysis, civil or structural engineering design, value engineering and construction phases of public and private works projects, and during post-construction and forensic phases of projects. Engineering geology studies are performed by a geologist or engineering geologist that is educated, trained and has obtained experience related to the recognition and interpretation of natural processes, the understanding of how these processes impact human made structures (and vice versa), and knowledge of methods by which to mitigate against hazards resulting from adverse natural or human made conditions. The principal objective of the engineering geologist is the protection of life and property against damage caused by various geological conditions. The practice of engineering geology is also very closely related to the practice of geological engineering and geotechnical engineering. If there is a difference in the content of the disciplines, it mainly lies in the training or experience of the practitioner. Modern form of the science and practice of engineering geology only commenced as a recognized discipline until the late 19th and early 20th centuries (De Mello, 1977).

1928, with the failure of the St. Francis Dam in California and the death of 426 people. Moreover few other engineering failures which occurred the following years also prompted the requirement for engineering geologists to work on large engineering projects during same year need for geologist on engineering works gained worldwide attention.

Civilizations flourished along the river banks such as Nile (Egypt), Tigris and Euphrates (Mesopotamia), Huang Ho (China) and Indus (Pakistan). Dykes dating back to about 2000 B.C. were built in the Indus basin to protect town of Mohenjo-Daro. During Chan dynasty (1120 to 249 B.C.) several dykes were built for irrigation purpose. However, no evidences for the stability of foundation and erosion by floods have been found.

One of the most famous problems related to soil bearing capacity is construction of Pisa tower. The construction began in 1173 A.D. and completed in period of 200 years. Overall weight of structure is 15,700 metric tons and in foundation has a circular base having diameter of 20m. The tower has tilted to east, north, west and south. Studies have revealed that there is a weal layer of clay at depth of 11m below ground surface compression which caused tower to tilt. It has been recently stabilized by excavating soil from the north side of tower. 70 metric tons of earth was removed in 41seperate extractions. As ground gradually settled the tilt of tower eased. The tower now leans 5 degree.

After encountering several foundation related problems during construction over past centuries engineers and scientist began to address the properties and behavior of soils in early part of 18thcentury. Based on nature of study in this specific field time span extending from 1700 to 1927 can be divided into 4 major periods (Skempton, 1985). These as, Pre classical (1700-1776 A.D.), Classical Soil Mechanics-Phase 1 (1776-1856 A.D.), Classical soil Mechanics-Phase 2 (1856-1910 A.D.) and Modern Soil Mechanics (1910-1927 A.D.)

Natural slopes of soils are tipped in a heap for formulating the design procedure of retaining walls. The natural slope is what refers to as angle of repose now. The slope of clean dry sand and ordinary earth is 31˚ and 45˚ respectively. Unit weight of clean sand and ordinary earth is 18.1 kN/m3 and 13.4 kN/m3 (Henri Gautier, 1717). Principles of calculus, maxima and minima can be used to determine the position of sliding surface in solid behind retaining wall (Charles Augustin Coulomb, 1776).

Extended Coulomb's theory can be used providing a graphical method for determining the magnitude of lateral earth pressure on vertical and inclined retaining walls with arbitrarily broken polygonal ground surface. Jean was the first to use symbol ϕ for soil friction angle. He also provided the first ultimate bearing capacity theory for shallow foundations (Jean Victor Poncelet, 1840). Details for deep slips in clay slopes, cutting and embankments were studied by Alexandre Collin, He theorized that all the cases of failure took place when the mobilized cohesion exceeds the existing cohesion of soils. He also observed that actual failure surfaces could be approximated as arcs of cycloids (Alexandre Collin, 1864).

Albert Mauritz Atterberg, Defined clay size in fractions as percentage by weight of particles smaller than 2 microns in size. He also identified the importance of clay particles in a soil and plasticity thereof. Albert Maurtiz Atterberg, explained the consistency of cohesive soils by defining liquid, plastic and shrinkage limits. He also defined plasticity index as the difference between liquid limits and plastic limits (Albert Mauritz Atterberg, 1908). Bell worked on the design and construction of the outer seawall at Rosyth Dockyard. Based on his work, he developed relationships for lateral pressure and resistance in clay as well as bearing capacity of shallow foundations in clay. He also used shear-box tests to measure the undrained shear strength of undisturbed clay specimens (Arthur Langley Bell, 1915). Wolmer developed the stability analysis of saturated clay slopes (ϕ=0 condition) with the assumption that the critical surface of sliding is the arc of a circle. The correct numerical solutions for the stability numbers of circular slip surfaces passing through the toe of the slope (Wolmar Fellenius, 1926). Karl Terzaghi, published the first text in Soil Mechanics (in German). Terzaghi is known as the father of soil mechanics, but also had great interest in geology; Terzaghi considered soil mechanics to be a sub-discipline of engineering geology (Karl Tarzaghi, 1925).

Casgrande made contribution to analysis of soft clay, soil compaction and classification, seepages, earth dams and other topics (Arthur Casgrande, 1932).

Lazarus White, a foundation engineer and builder who developed of design and construction, underpinning and other advances. R.R. Proctor, he made important assessment of compacted fills during construction. Burland and Worth explained that ground improvement includes systems that use the ground or some modification of it to transfer or support loads. Ground improvement can increase soil strength and stiffness or reduce permeability. In many situations ground improvement can be used to support new foundation or increase capacity of existing foundations (Burland and Worth, 1977). Charlie suggested that design of structure on soft compressible soils has created problems for geotechnical engineers. Construction without soil treatment is impractical due to long term settlement (Charlie et al., 1984).

During past century history has noticed major improvement in geotechnical engineering for predicting the behavior of soil and rock. Mathematical equations cannot be a reliable dependency as same model cannot be implemented on diverse site due to distinctive nature of soil. Major mistake is overestimation of geotechnical analysis by the one practicing. In reality actual behavior often varies form predicted behavior by 50% or more. Therefore it is best to perform geotechnical analysis upto 2 or 3 significant figure.

CHAPTER 3
TECTONICS AND STRATIGRAPHY

Among the most dramatic and visible creations of plate-tectonic forces are the lofty Himalayas, which stretch 2,900 km along the border between India and Tibet. This immense mountain range began to form between 40 and 50 million years ago, when two large landmasses, India and Eurasia, driven by plate movement, collided. Because both these continental landmasses have about the same rock density, one plate could not be subducted under the other. The pressure of colliding plates could only be released by thrusting.

About 225 million years ago, India was a large island still situated off the Australian coast, and a vast ocean (called Tethys Sea) separated India from the Asian continent. When Pangaea broke apart about 200 million years ago, India began to forge northward. By studying the history -- and ultimately the closing-- of the Tethys, scientists have reconstructed India's northward journey. About 80 million years ago, India was located roughly 6,400 km south of the Asian continent, moving northward at a rate of about 9 m a century. When India rammed into Asia about 40 to 50 million years ago, its northward advance slowed by about half. The collision and associated decrease in the rate of plate movement are interpreted to mark the beginning of the rapid uplift of the Himalayas (An Yin, T. Mark Harrison, 2000).

The Himalayas and the Tibetan Plateau to the north have risen very rapidly. In just 50 million years, peaks such as Mt. Everest have risen to heights of more than 9 km. The impinging of the two landmasses has yet to end. The Himalayas continue to rise more than 1 cm a year a growth rate of 10 km in a million years! If that is so, why aren't the Himalayas even higher? Scientists believe that the Eurasian Plate may now be stretching out rather than thrusting up, and such stretching would result in some subsidence due to gravity (An Yin, T. Mark Harrison, 2000).

The study area lies in Peshawar Basin and its detailed stratigraphy is explained below.

3.1 Peshawar basin

The Peshawar basin in general consists of Paleozoic stratigraphy overlain by Quaternary alluvium. Quaternary alluvium is valley fill consisting of silt, sand, and gravel. Includes some terrace deposits and glacial drift of Pleistocene age in some areas locally includes hot spring tufa.

An almost complete Paleozoic sequence of sedimentary rocks is exposed in the ranges fringing Peshawar Basin. The Precambrian-Cambrian Tanawal formation forms the base of the sequence and is overlain unconformably by Amber Formation. The Misri Banda Quartzite unconfromably overlies the Amber formation and contains Cruziana ichnofossils which indicate an Early to Middle Ordovician age. The limestone at the base of the Panjpir Fromation contains siluricus zone conodonts and unconformably overlies the Misri Banda Formation. The Early to Late Devonian Nowshera Formation overlies the Panjpir and contains a reef facies. The youngest recognized Paleozoic unit is the Jafar Kandao Formation from which Carboniferous conodonts have been obtained. The section outlined above has possible correlatives in Khyber and Hazara regions, but differs dramatically from the Paleozoic sequence of Salt Range to the south. The tectonic setting of the area is transitional between a sedimentary fold-thrust belt to the south and a metamorphic terrene to the north. The Paleozoic stratigraphic sequence of Peshawar basin is given below (Husain et al, 1991).

3.1.1 Amber Formation

The formation is comprised of dolomite, dolomitic limestone, calcareous quartzite and subordinate argillite. Chert in the form of veinlets and nodules is found at places. The dolomite contains algal laminations and poor development of strornatolites. The lower contact of the formation is unconformable with Tanawal Formation in Swabi area. The contact is locally marked by a conglomerate bed with cobble and pebble size in a matrix of dolomitic quartzite and argillite. South of Swabi, the lower contact of the formation is covered under the alluvium of Peshawar basin. The upper contact of the formation is also unconformable with Misri Banda Quartzite and is marked by 5-10 meters of maroon colored shale. In Chingalai area, however, the unconformity is represented by about 10 meters of conglomerate consisting of pebbles and cobbles of quartzite and dolomite in quartzite matrix (Shah. 2009).

The formation has not revealed any fossils except microscopic shell debris in the interstices of pisoliths from Ambar section. It has been tentatively placed in Cambrian because of its stratigraphic position resting above the Tanawal Formation of Precambrian age and below the Misri Banda Quartzite of Ordovician age. The formation is correlated with Abbottabad Formation in Hazara on the basis of its stratigraphic position and lithological similarity (Shah. 2009).

3.1.2 Misri Banda Formation

The quartzite at Misri Banda is lithologically distinct from the calcareous quartzite overlying Nowshera and also contains Cruziana ichnofossils of Ordovician age.

The quartzite is light grey to pinkish-grey and contains fine to medium-grained quartz and feldspar in siliceous and calcareous matrix. Cross-bedding, ripple marks and graded bedding are commonly found in the quartzite. In some parts of the quartzite sequence vertically oriented tube-shaped burrows are preserved. A dark grey thinly laminated argillite is commonly associated in the upper part of Misri Banda Quartzite. The upper contact of the Misri Banda Quartzite is unconformable with the Panjpir Formation. The unconformity is marked by discontinuous conglomerate bed composed of rounded to sub rounded cobbles and pebbles of quartzite and dolomite in calcareous quartzite matrix (Shah. 2009).

3.1.3 Panjpir Formation

The formation is composed of argillite and phyllite with interbeds of crinoidal limestone, metasiltstone and argillaceous and calcareous quartzite. These rocks are generally dark grey to greenish-grey, silty, fissile and chloritic. The upper part of the formation is characterized by interbedded argillite and crinoidal limestone. The formation has a conformable contact with the overlying NowsheraForrnation. The contact can be placed at the base of massive limestone overlying the interbedded argillite and limestone of the Panjpir Formation.

The samples of crinoidal limestone from the upper part of Panjpir Formation have yielded Late Silurian (Pridolian) conodonts. The fossiliferous horizon has widespread exposure and Late Silurian conodonts have also been reported at the Misri Banda, Ambar, and Panjpir outcrops (Shah. 2009).

3.1.4 Nowshera Formation

The Nowshera Formation is composed of limestone and dolomitic limestone (marble), calcareous quartzite and sandstone, and subordinate argillite. This formation was subdivided into reef core, carbonate containing reef breccia or fossil debris, and carbonate containing fewer or no fossils. It constitutes the youngest Paleozoic sedimentary formation exposed between the Nowshera and Swabi areas (Shah. 2009).

The lower part of the formation in the Nowshera area yielded corals, brachiopods, gastropods, cephalopods, stromatoporoids and conodonts that reveal an Early Devonian (Lochkovian) age. The formation thus ranges in age from Early to early Late Devonian. The Nowshera Formation is correlated with the Ghundai Sar "reef complex" of the Khyber area (Shah. 2009).

3.1.5 Jafar Kandao Formation

The principal lithology of the formation is argillite with subordinate interbeds of limestone, argillaceous quartzite and conglomerate. 0n the basis of lithology the formation is subdivided into lower, middle, and upper parts. The lower part consists of argillite with lenses of limestone, argillaceous quartzite and conglomerate. The conglomerate occurs in channels and contains clasts of granite and quartzite. The middle part is dominated by interbedded argillite, calcareous quartzite, and sandy limestone (Shah. 2009).

The upper part contains argillite with lenses of argillaceous quartzite and conglomerate. The formation is overlain by green schist which is the southern extension of amphibolites (Shah. 2009).

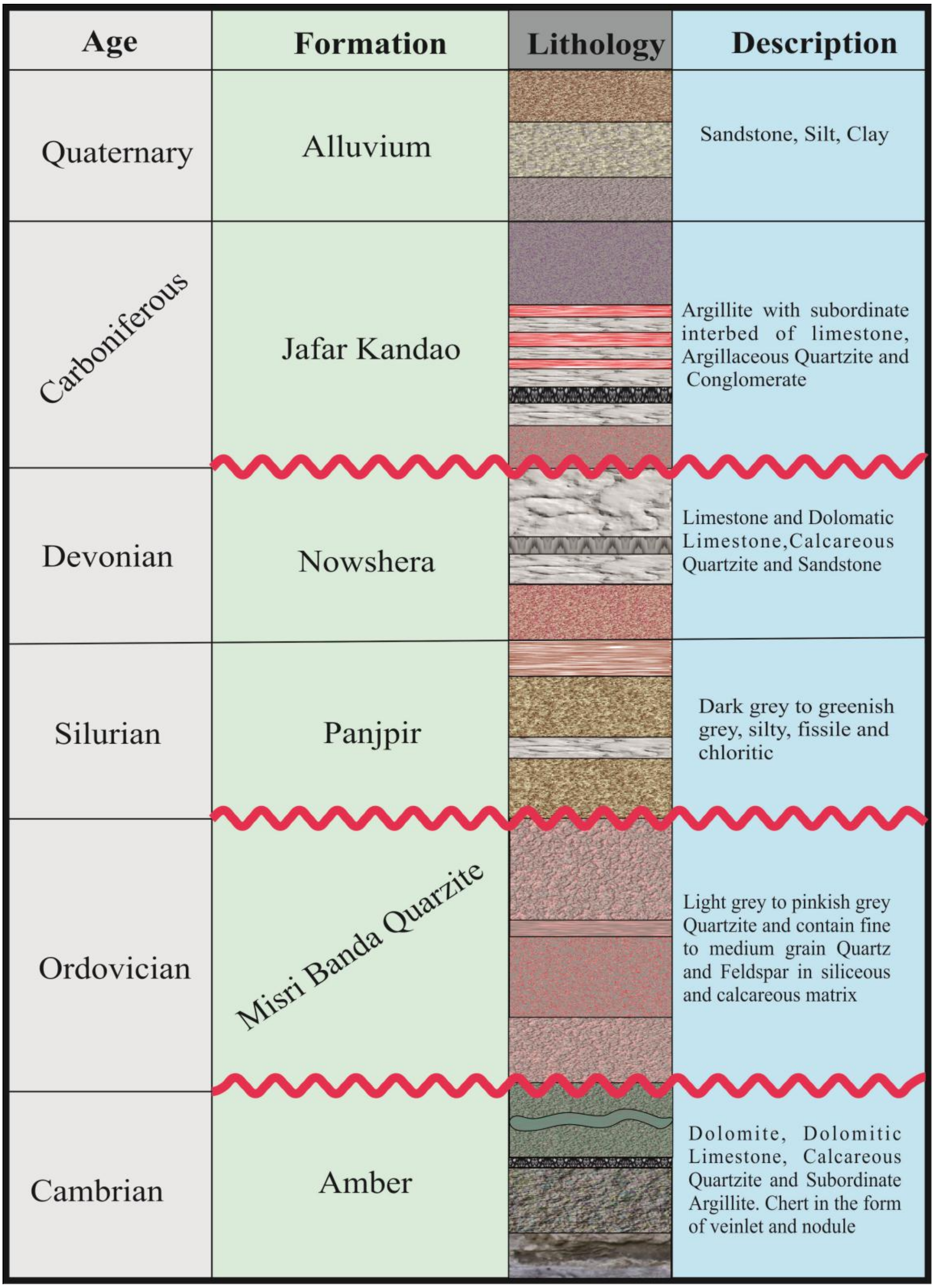

Figure 3.1. Generalized stratigraphic chart of study area (Shah, 2009).

CHAPTER 4
METHODS AND MATERIALS

The geotechnical investigation of proposed site was carried out by drilling boreholes up to depth of 100.0 ft using Light Percussion drilling. Standard Penetration Test was adopted to collect both disturbed and undisturbed samples. Both laboratory and on site testing and analysis were carried out as per requirement.

4.1 Field activity

Field activity was conducted using following tools.

4.1.1 Drilling of boreholes

Seven bore holes were drilled by Light Percussion drilling method. Drilling was done up to maximum depth of 100.0 ft at various locations. The encountered sub-soil stratum is termed as Quaternary alluvium composed of Lean Clay with traces of Sand and Gravel and Silty Sand with traces of Gravels.

4.1.2 Collection of undisturbed / disturbed samples

Samples were obtained at various depths disturbed samples were obtained using SS sampler of SPT and sealed to avoid moisture content loss. The undisturbed soil samples were collected through Shelby tubes at various depths and vexed on either side of tubes as per ASTM standards.

In borehole logs UDS denotes Undisturbed Soil Samples and DS denotes Disturbed Soil Samples.

4.1.3 Standard Penetration Test (ASTM D 1586-84)

SPT is an important field test, which in most cases furnishes a fairly direct correlation with the bearing capacity of the soil layer, especially for materials, which are sandy in nature. In case of cohesive strata, however, the SPT results have to be interpreted by keeping in view the composition of clay, silt and other admixture is present.

4.1.3.1 Apparatus

Hammer, SPT Rods, Guide rod, containers, split spoon sampler, wrenches.

4.1.3.2 Procedure

The boring is done in order to permit continuous sampling. The intervals selected by geologist are 5 ft or less in homogeneous strata with test and sampling locations at every change of strata. After the boring has been advanced to the desired sampling elevation and rock cuttings have been removed, prepare for the following tests.

Attach the split barrel sampler to the sampling rods and lower into the borehole. Do not allow the sampler to drop onto the soil to be sampled. Position the hammer above and attach the anvil to the top of sampling rods. This may be done before the sampler and sampling rods are lowered into the borehole. Rest the dead weight of the sampler, rods, anvil and drive weight on bottom of boring and apply a seating blow. If excessive cuttings are encountered at bottom of boring, remove the sampler and sampling rods from the boring and remove the cuttings.

Mark the drill rods in three successive 6-inch increments so that the advance of the sampler under the impact of the hammer can be easily observed for each 6-in increment. Drive the sampler with blows from the 140-lb hammer and count the number of blows applied in each 6-inch increment. A total of 50 blows have been applied during any one of the three 6-in increments. A total of 100 blows have been applied. There is no observed advance of the sampler during the application of 10 successive blows of hammer. The sampler is advanced the complete 18-in without the limiting blow counts. The first 6-in is considered to be a seating drive. The sum of the number of blows required for the second and third 6-in of penetration is termed the standard penetration resistance or the N-value. Now for each hammer blow for 30-in lift and drop shall be employed by the operator. The operation of drilling and throwing the rope shall be performed rhythmically without holding the rope at top of the stroke. Bring the sample on surface and open. Record the percent recovery or sample length recovered. Describe sample as to composition, color, stratification and condition then place sample into sealable moisture proof containers without ramming or distorting any apparent stratification. Seal each container to prevent evaporation of soil moisture. Fix labels to containers bearing job designation, boring number, sample depth and the blow count per 6-in increment. Protect the samples from extreme temperature changes.

4.1.3.3 Limits

The SPT should not be relied on in soils containing coarse gravel, cobbles or boulders because the sampler can become obstructed giving high and wrong N-values. It should also not be relied on for cohesion less silts because dynamic effects at sampler can lead to erroneous strength and compressibility determination.

The test has meaning with sensitive clays. In such soils the SPT yields results inconsistent with actual in-situ conditions.

Because of the uncertainty of qualitative or quantitative effects of many of the variables which could influence the SPT blow count data, it is recommended that the standard procedures should be followed and practiced.

4.2 Laboratory testing

Soil samples were tested in the laboratory for the index and the properties of the soil strength according to the ASTM standards.

(1) Sieve Analysis

(2) Atterberg Limits

(3) Unconfined Compressive Strength Test

(4) Direct Shear Test

4.2.1 Sieve analysis (ASTM C-136)

Sieve analysis (ASTM C-136) is a test conducted for the classification of soil materials. Sieve analysis consists of shaking the soil sample through a set of sieves that have progressively smaller openings. The sieves used for soil analysis are generally 203mm (8 in.) in diameter. To conduct sieve analysis, one must first oven dry the soil and then break all lumps into small particles. The soil then is shaken through a stack of sieves with openings of decreasing size from top to bottom and pan is placed below the stack.

The smallest sized sieve that should be used for this test is U.S no. 200 sieve. After the soil is shaken, the mass of soil retained on each sieve is determined. Portions retained on each sieve are collected separately and oven dried before the mass retained on each sieve is measured.

Once the percent finer for each sieve is calculated, the calculations are plotted on semi logarithmic graph paper with percent finer as the ordinate and sieve opening size as the abscissa. This plot is referred to as the particle size distribution curve.

4.2.1.1 Apparatus

Stack of Sieves including pan and cover. Balance (with accuracy to 0.01 g). Rubber pestle and Mortar (for crushing the lumped soil). Mechanical sieve shaker. Oven.

4.2.1.2 Procedure

Take a representative oven dried sample of soil that weighs about 300 g. If soil particles are lumped or conglomerated crush the lumped and not the particles using the pestle and mortar. Determine the mass of sample accurately. Wt (g) Prepare a stack of sieves. Sieves having larger opening sizes (i.e. lower numbers) are placed above the ones having smaller opening sizes (i.e. higher numbers). The very last sieve is #200 and a pan is placed under it to collect the portion of soil passing #200 sieve. Make sure sieves are clean; if many soil particles are stuck in the openings try to poke them out using brush. Pour the soil into the stack of sieves from the top and place the cover, put the stack in the sieve shaker and fix the clamps, adjust the time on 10 to 15 minutes and get the shaker going. Stop the sieve shaker and measure the mass of each sieve retained soil.

4.2.1.3 Calculations

The results are presented in a graph of percent passing versus the sieve size. On the graph the sieve size scale is logarithmic. To find the percent of aggregate passing through each sieve, first find the percent retained in each sieve. To do so, the following equation is used.

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

Where w.sieve is the weight of aggregate in the sieve and w.total is the total weight of the aggregate. The next step is to find the cumulative percent of aggregate retained in each sieve. To do so, add up the total amount of aggregate that is retained in each sieve and the amount in the previous sieves. The cumulative percent passing of the aggregate is found by subtracting the percent retained from 100%.

%Cumulative Passing = 100% - %Cumulative Retained.

The values are then plotted on a graph with cumulative percent passing on the y axis and logarithmic sieve size on the x axis.

4.2.1.4 Precautions

(1) Proper care of the sieves is necessary for accurate sieving.

(2) Examine sieves each day for broken wires and solder any breaks.

(3) Discard any sieve that develops a break in the main body of the screen.

(4) Soldering decreases effective sieving area; therefore, sieves with large breaks or several small breaks should be discarded.

(5) Never sieve hot samples, as hot aggregate will distort the fine mesh of the No.100 and No. 200 sieves.

(6) Take care to avoid loss of material during transfer of sample from wash pot to sieves and also during rinsing.

(7) Do not overload sieves.

(8) Take care to avoid loss of material due to volume or pressure of water when rinsing samples through the No. 200 sieve.

4.2.1.5 Limitation

Material that is finer than 100 no mesh, dry sieving can be significantly less accurate. This is because the mechanical energy required to make particles pass through an opening and the surface attraction effects between the particles themselves and between particles and the screen increase as the particle size decreases. Wet sieve analysis can be utilized where the material analyzed is not affected by the liquid except to disperse it.

Sieve analysis assumes that all particles will be round and will pass through the square openings when the particles diameter is less than the square opening in the screen. For elongated and flat particles a sieve analysis will not yield reliable mass based results, as the particle size reported will assume that the particles are spherical, where in fact an elongated particles might pass through the screen end-on, but would be prevented from doing so if presented itself side-on.

The laboratory determination of Sieve analysis is shown in figure 4.1.

Figure 4.1. Sieve analysis in laboratory.

4.2.2 Atterberg limits (ASTM D-4318)

The Atterberg limits are a basic measure of the critical water contents of a fine-grained soil, such as its shrinkage limit, plastic limit, and liquid limit. As a dry, clayey soil takes on increasing amounts of water, it undergoes dramatic and distinct changes in behavior and consistency. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state, the consistency and behavior of a soil is different and consequently so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behavior These tests are mainly used on clayey or silty soils since these are the soils that expand and shrink due to moisture content. This test can be used to find the following limits.

(1) Liquid limit

(2) Plastic limit

4.2.2.1 Liquid Limit

(a) Apparatus

Sieve #40, spatula, grooving tool, containers and balance.

(b) Procedure

Take 3/4 of the soil and place it into the dish. Soil was previously passed through a No. 40 sieve, air-dried, and then pulverized. Thoroughly mix the soil with a small amount of distilled water until it appears as a smooth uniform paste. Cover the dish with cellophane to prevent moisture from escaping. Squeeze the soil down to eliminate air pockets and spread it into the cup to a depth of about 10 mm at its deepest point. The soil pat should form an approximately horizontal surface. Then use the grooving tool and mark straight groove down the center of the cup. Use extreme care to prevent sliding the soil relative to the surface of the cup.

Turn the crank of the apparatus at a rate of approximately two drops per second and count the number of drops, N, it takes to make the two halves of the soil pat come into contact at the bottom of the groove along a distance of 13 mm (1/2 in.). Place the soil into a moisture can cover it. Immediately weigh the moisture can containing the soil and place the can into the oven. Leave the moisture can in the oven for at least 16 hours. Place the soil remaining in the cup into the porcelain dish. Afterwards remix the entire soil specimen in the porcelain dish. Add a small amount of distilled water to increase the water content so that the number of drops required to close the groove decrease. Repeat steps six, seven, and eight for at least two additional trials producing successively lower numbers of drops to close the groove. One of the trials shall be for a closure requiring 25 to 35 drops, one for closure between 20 and 30 drops, and one trial for a closure requiring 15 to 25 drops. Determine the water content from each trial by using the same method used in the first laboratory. Remember to use the same balance for all weighing. The laboratory testing of liquid limit is shown in figure 4.2.

Figure 4.2 Laboratory testing of Liquid Limit.

4.2.2.2 Plastic Limit

(a) Apparatus

Plastic limit plate, steel rods, mixing dish, spatula.

(b) Procedure

Weigh the remaining empty moisture cans with their lids, and record the respective weights and can numbers on the data sheet. Take the remaining 1/4 of the original soil sample and add distilled water until the soil is at a consistency where it can be rolled without sticking to the hands. Then form the soil into an ellipsoidal mass. Roll the mass between the palm or the fingers and the glass plate .Use sufficient pressure to roll the mass into a thread of uniform engineering properties of soils diameter by using about 90 strokes per minute. The thread shall be deformed so that its diameter reaches 3.2 mm (1/8 in.), taking no more than two minutes. When the diameter of the thread reaches the correct diameter, break the thread into several pieces. Knead and reform the pieces into ellipsoidal masses and re-roll them.

Continue this alternate rolling, gathering together, kneading and re-rolling until the thread crumbles under the pressure required for rolling and can no longer be rolled into a 32 mm diameter thread. After that gather the portions of the crumbled thread together and place the soil into a moisture can, then cover it. If the can does not contain at least 6 grams of soil, add soil to the can from the next trial. Immediately weigh the moisture can containing the soil, record it's mass, remove the lid, and place the can into the oven. Leave the moisture can in the oven for at least 16 hours. Determine the water content from each trial by using the same method used in the first laboratory. Remember to use the same balance for all weighing.

(c) Precautions

(1) After performing each test the cup and grooving tool must be cleaned.

(2) The number of blows should be just enough to close the groove.

(3) The number of blows should be between 10 and 40.

(d) Limitation

The one limitation which comes across with the Atterberg limits is that it will give no indication of particle fabric or residual bonds between particles which may have been developed in the natural soil but are destroyed in preparing the specimen for the determination of limits.

(e) Impact on strength

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

Laboratory testing of liquid limit is shown in figure 4.3.

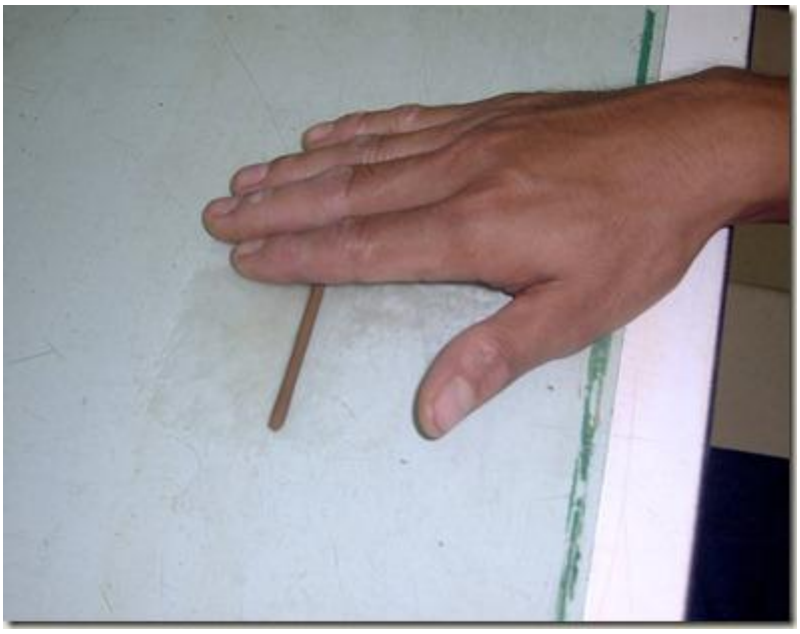

Figure 4.3. Laboratory determination of Plastic Limit.

4.2.3 Unconfined Compression Test (ASTM D-2166)

The purpose of this laboratory is to determine the unconfined compressive strength of a cohesive soil sample. We will measure this with the unconfined compression test, which is an unconsolidated undrained (UU or Q-type) test where the lateral confining pressure is equal to zero (atmospheric pressure).

4.2.3.1 Apparatus

Hand operated or motorized axial compression machine, timer, dial gauge, balance, plastic and metal end caps.

4.2.3.2 Procedure

Measure the initial height and diameter of the soil sample with calipers. Therefore, it will be necessary to find the average height and diameter by taking several measurements in different places along the soil sample. The measurements should be taken two to three times. Record the weight of the soil sample and determine the total (moist) unit weight. Then place the soil sample in the loading frame, seat the proving ring and zero the dials. Record the load applied at specified strain values. It is recommended that readings be taken at strains of 0, 0.1, 0.2, 0.5, 1,2,3,4,5,6,8, 10, 12 14, 16, 18 and 20 percent. With the measured initial height of sample (H^o), the desired percent strain (ε) and the initial dial reading (S^o), calculate the dial readings (S) with the formula

$$S = S^{\circ} + \left(\frac{\varepsilon}{100}\right) H^{\circ}$$

Readings of force (F) are taken from the proving ring dial gauge and the stress applied to the ends of the sample is computed as follows:

$$\sigma 1 = \frac{F}{A}$$

Where, A is the cross-sectional area of the sample. The equivalent or average area (A) at any strain (e) is computed from the initial area (Ac) and the assumption that volume is conserved

$$A = \frac{A^{\circ}}{1 - \varepsilon}$$

The unconfined compressive strength (qu) is the maximum value σ1, which may or may not coincide with the maximum force measurement.

4.2.3.3 Limitation

The limitations of the unconfined compression test is applicable to the fully saturated non-fissured clays, and only the undrained strength C_u can be measured.

4.2.3.4 Impact on strength

This test tells us about the strength of the soil and the amount of stress/strain which it can bear.

Testing apparatus of UCS is shown in figure 4.4.

Figure 4.4. Unified Compression Test apparatus.

4.2.4 Direct shear test (ASTM D-3080)

In many engineering problems such as design of foundation, retaining walls, slab bridges, pipes, sheet piling, the value of the angle of internal friction and cohesion of the soil involved are required for the design. Direct shear test is used to predict these parameters quickly. The laboratory report covers the laboratory procedures for determining these values for cohesion less soils.

4.2.4.1 Apparatus

Direct shear box apparatus, loading frame, dial gauge, proving ring, tamper, straight edge, balance, aluminum container, and spatula.

4.2.4.2 Procedure

Check the inner dimension of the soil container. Put the parts of the soil container together. And calculate the volume of the container. Weigh the container. Then place the soil in smooth layers (approximately 10 mm thick). If a dense sample is desired tamp the soil. Weigh the soil container, the difference of these two is the weight of the soil. After that calculate the density of the soil and make the surface of the soil plane. Then put the upper grating on stone and loading block on top of soil.

Measure the thickness of soil specimen and apply the desired normal load. Remove the shear pin and attach the dial gauge which measures the change of volume. Record the initial reading of the dial gauge and calibration values. Before proceeding to test check all adjustments to see that there is no connection between two parts except sand/soil. Now start the motor. Take the reading of the shear force and record the reading. Take volume change readings till failure. Then add 5 kg normal stress 0.5 kg/cm^2 and continue the experiment till failure. Record carefully all the readings. Set the dial gauges zero, before starting the test. The apparatus used for direct shear test is shown in figure 4.5.

4.2.4.3 Limitation

The practical is not performed on undisturbed soil.

Figure 4.5. Direct Shear Test Apparatus.

CHAPTER 5
RESULTS AND DISCUSSIONS

5.1 Discussions

The geotechnical investigation of proposed site was carried out in the following manners

(1) Drilling of seven boreholes of different depth at different locations.

(2) Performance of Standard Penetration Test (SPT) at 3 and 5 ft intervals.

(3) Determination of bulk density and in-situ moisture content.

(4) Collection of disturbed and undisturbed soil samples for laboratory testing.

The soil starting from surface level to maximum drilling depth of 100.0 ft is composed of Light brown, medium stiff Lean Clay with traces of sand and gravels along with Silty Sand with traces of gravels. In the summary sheets they are presented as

CL : Lean Clay with traces of Sand and Gravels

SM : Silty Sand with traces of Gravels

Light Percussion Machine was used for drilling up to depth of 100.0 ft. ground water table was encountered at depth of 6.0 ft in BH-1, BH-3and BH-5. 9.0 ft in BH-6 and BH-7 and 12.0 ft in BH-2 and BH-4.

Standard Penetration Tests are performed according to ASTM Designation D-1586-84. The SPT blows count values "N" recorded at various depth are shown in bore hole logs and summary of test results.

Disturbed and undisturbed samples were taken at various depth by SS sampler of SPT and sealed to avoid loss of moisture content and undisturbed samples were taken through Shelby tubes from various depth. In the borehole logs DS denotes the disturbed samples and UDS undisturbed soil samples.

Grain size analysis of soil samples indicates the percentage of soil particles; Gravel 0.0 to 11.0%, Sand 4.0 to 68.0% and Lean clay 32.0 to 95.0% has been calculated. Liquid and Plastic limit of clay mix soil vary from 26.0 to 32.0 and 16.0 to 21.0 respectively. The Plastic index value ranges from 8 to 13. According to Unified Soil Classification System (USCS) the soil is predominantly classified as Lean Clay with traces of Sand, Gravel and Silty Sand. The stratum of the soil is approximately uniform and no wide variation in soil characteristics is found. Bulk density varies between 12.5 to 117.5 pcf for Lean Clay with traces of Sand and Gravels. In direct shear test value of internal cohesion varies from 1060 to 1270 psf and angle of internal friction is from 24.5° to 29.9° in sandy clay and angle of friction for Sand ranges from 32.7° to 35.7°. pH test was also conducted the sole purpose of this test is to check if the soil is acidic or alkaline. The cover of concrete and choice of cement is made accordingly. pH of subsurface water at proposed site was determined as 8.0 which is within permissible limits (7.0-8.5).

Soil samples were tested in laboratory for index and strength properties of soil. All the tests were carried out as per ASTM standards.

Bearing capacity of foundation soil can be determined with standard penetration test (SPT) as well as Unconfined Compression Test (UC). Tarzaghi (1943) explained expression for the ultimate bearing capacity for general shear conditions as

Strip footing $qu = c\,Nc + \gamma\,D\,Nq + \frac{1}{2}\gamma\,B\,Nr$

Isolated footing $1.2\,c\,Nc + \gamma\,D\,Nq + 0.4\,\gamma\,B\,Nr$

Assessment of Bearing capacity with SPT blows "N" is calculated using allowable bearing capacity equation developed by Tarzaghi and Peck. Where "N" is the no. of SPT blows. The equation is given as under

$$Qa = 0.72\left(\frac{B+1}{2B}\right) * \frac{1000}{2240} * (N-3)\ \text{TSF}$$

Where

Qa = Allowable Bearing Capacity of Soil

B = Width of footing in feet

N = SPT blows count

Bearing capacity equation is developed by Meyerhof is as under

$$Qa = \frac{N}{4} * \frac{1000}{2240}\ \text{TSF}$$

Where

Qa = Allowable bearing capacity of soil

N = SPT blows count

Similarly for Raft footing following equation was developed by Meyerhof

$$Qa = 360\ (N\text{-}3)\ \text{lbs/Sft}$$

Where

Qa = Allowable bearing Capacity

N = SPT blows count

For calculating allowable bearing capacity for the construction of Khyber Institute of Neuro Science and Clinical research we have adopted Meyerhof's equation (1965).

CONCLUSIONS

On the basis of field and laboratory tests following conclusions can be deduced.

(1) Field and laboratory testing were conducted using ASTM standards and geotechnical interpretations were made.

(2) Most suitable foundation type for proposed site was concluded as Raft foundation.

(3) Consistency of sub-soil stratum of all boreholes is firm up to the depth of 30.0 ft due to high moisture in soil and this area require some improvement. So, it is recommended to provide 3.0 ft granular pad under foundation lean, containing 50% crushed aggregate 3\4 size and 50% coarse sand duly mixed in a mixture machine at optimum moisture contents. The mix should be laid in 6.0 equal layers (lift thickness 6 inches) and each layer should be compacted with vibratory roller to attain 95.0% relative compaction.

(4) Temporary pits\sumps should be made at end of excavation\trenches. Water should be pumped out and then the lying of granular layer. (Gherra) activity should be started on excavated compacted surface. Efficient dewatering is recommended and no accumulation of water in excavation is allowed.

(5) Load calculated was 0.75 TSF.

(6) The allowable bearing capacity for the proposed site at or below 6.0 ft depth (without basement) from existing surface level is 0.75 TSF for Raft footing and below 12.0 ft depth (with single basement) from existing surface level is 0.75 TSF for Raft footing.

RECOMMENDATIONS

(1) If any abnormality in soil strata is noticed during excavation of footings, Site Engineer should bring in our notice for doing the needful.

(2) The foundation bed soil should be protected from ingress of moisture from any source by providing adequate surface drainage system and ensuring leaks proof jointing of sewerage and water supply lines.

(3) It is recommended to provide sufficient ventilation arrangement in the basement to avoid suffocation and dampness. Proper moisture barriers should also be provided against seepage and dampness through use of quality materials available in market and by adopting adequate quality control\assurance during construction of basement.

(4) Safety of workers and adjacent structures should be ensured through adequate measures during all construction activities.

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ANNEXURES

Annexure-1

(SUMMARY OF TEST RESULTS)

Site: Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.

Serial	Borehole	Depth	Bulk Density	IMC	Grain Size Analysis (%age of Material)			Atterberg Limits			Soil	Stratum	SPT Blows	DS Test		UC Test	
No.	No.	(ft)	(pcf)	(%)	Gravel	Sand	Silt & Clay	LL	PL	PI	Type	Condition	"N"	c (psf)	ø°	qu (tsf)	Strain (%)
1	1	3	115.5	14.4	0	5	95	27	16	11	CL	Natural	7	-	-	0.75	6.00
2	"	6	-		0	8	92	Same as above			CL	Natural	7	-	-	-	-
3	"	9	-	G/W	6	9	85	Same as above			CL	Natural	6	1060	29.9	-	-
4	"	12	-	G/W	7	13	80	Same as above			CL	Natural	7	-	-	-	-
5	"	15	-	G/W	1	6	93	Same as above			CL	Natural	8	-	-	-	-
6	"	20	-	G/W	0	20	80	Same as above			CL	Natural	7	-	-	-	-
7	"	25	-	G/W	0	18	82	Same as above			CL	Natural	6	-	-	-	-
8	"	30	-	G/W	1	52	47	Non-Plastic			SM	Natural	8	-	-	-	-
9	"	35	-	G/W	2	50	48	Non-Plastic			SM	Natural	8	-	-	-	-
10	"	40	-	G/W	0	57	43	Non-Plastic			SM	Natural	9	-	-	-	-
11	"	45	-	G/W	1	60	39	Non-Plastic			SM	Natural	10	-	-	-	-
12	"	50	-	G/W	0	59	41	Non-Plastic			SM	Natural	9	-	-	-	-
13	"	55	-	G/W	0	51	49	Non-Plastic			SM	Natural	10	-	-	-	-
14	"	60	-	G/W	0	55	45	Non-Plastic			SM	Natural	11	-	-	-	-
15	"	65	-	G/W	0	59	41	Non-Plastic			SM	Natural	12	-	-	-	-
16	"	70	-	G/W	0	55	45	Non-Plastic			SM	Natural	11	-	-	-	-
17	"	75	-	G/W	0	64	36	Non-Plastic			SM	Natural	13	-	-	-	-
18	"	80	-	G/W	0	54	46	Non-Plastic			SM	Natural	12	-	-	-	-
19	"	85	-	G/W	0	57	43	Non-Plastic			SM	Natural	13	-	-	-	-
20	"	90	-	G/W	0	61	39	Non-Plastic			SM	Natural	14	-	-	-	-
21	"	95	-	G/W	0	65	35	Non-Plastic			SM	Natural	15	-	-	-	-
22	"	100	-	G/W	0	65	35	Non-Plastic			SM	Natural	16	-	-	-	-
-	**Min**	3	-	14.4	0	5	35	-			-	-	5	-	-	-	-
-	**Max**	100	-	G/W	7	60	95	-			-	-	16	-	-	-	-

Annexure-2

(SUMMARY OF TEST RESULTS)

Site: Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.

Serial	Borehole	Depth	Bulk Density	IMC	Grain Size Analysis (%age of Material)			Atterberg Limits			Soil	Stratum	SPT Blows	DS Test		UC Test	
No.	No.	(ft)	(pcf)	(%)	Gravel	Sand	Silt & Clay	LL	PL	PI	Type	Condition	"N"	c (psf)	ø°	qu (tsf)	Strain (%)
1	2	3	113.5	11.4	1	5	94	29	20	9	CL	Natural	4	-	-	-	-
2	"	6	116.5	12.3	0	5	95	Same as above			CL	Natural	8	-	-	-	-
3	"	9	117.5	14.1	1	7	92	Same as above			CL	Natural	9	-	-	1.08	5.30
4	"	12	-		2	5	93	Same as above			CL	Natural	8	-	-	-	-
5	"	15	-	G/W	6	19	75	Same as above			CL	Natural	9	-	-	-	-
6	"	20	-	G/W	3	12	85	Same as above			CL	Natural	10	-	-	-	-
7	"	25	-	G/W	0	11	89	Same as above			CL	Natural	11	-	-	-	-
8	"	30	-	G/W	1	16	83	Same as above			CL	Natural	13	-	-	-	-
9	"	35	-	G/W	2	56	42	Non-Plastic			SM	Natural	12	-	-	-	-
10	"	40	-	G/W	9	51	40	Non-Plastic			SM	Natural	14	-	34.3	-	-
11	"	45	-	G/W	11	52	37	Non-Plastic			SM	Natural	13	-	-	-	-
12	"	50	-	G/W	0	56	44	Non-Plastic			SM	Natural	15	-	-	-	-
13	"	55	-	G/W	5	48	47	Non-Plastic			SM	Natural	14	-	-	-	-
14	"	60	-	G/W	0	65	35	Non-Plastic			SM	Natural	14	-	-	-	-
15	"	65	-	G/W	0	61	39	Non-Plastic			SM	Natural	15	-	-	-	-
16	"	70	-	G/W	7	61	32	Non-Plastic			SM	Natural	16	-	-	-	-
17	"	75	-	G/W	0	54	46	Non-Plastic			SM	Natural	17	-	-	-	-
18	"	80	-	G/W	0	59	41	Non-Plastic			SM	Natural	16	-	-	-	-
19	"	85	-	G/W	5	58	37	Non-Plastic			SM	Natural	18	-	-	-	-
20	"	90	-	G/W	0	57	43	Non-Plastic			SM	Natural	17	-	-	-	-
21	"	95	-	G/W	11	49	40	Non-Plastic			SM	Natural	17	-	-	-	-
22	"	100	-	G/W	0	67	33	Non-Plastic			SM	Natural	19	-	-	-	-
-	**Min**	3	113.5	11.4	0	5	33	-			-	-	4	-	-	-	-
-	**Max**	100	117.5	G/W	11	65	95	-			-	-	19	-	-	-	-

Annexure-3

(SUMMARY OF TEST RESULTS)

Site: Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.

Serial	Borehole	Depth	Bulk Density	IMC	Grain Size Analysis (%age of Material)			Atterberg Limits			Soil	Stratum	SPT Blows	DS Test		UC Test	
No.	No.	(ft)	(pcf)	(%)	Gravel	Sand	Silt & Clay	LL	PL	PI	Type	Condition	"N"	c (psf)	ø°	qu (tsf)	Strain (%)
1	3	3	113.0	15.7	0	6	94	29	18	11	CL	Natural	5	-	-	-	-
2	"	6	-		1	7	92	Same as above			CL	Natural	7	-	-	-	-
3	"	9	-	G/W	0	9	91	Same as above			CL	Natural	8	-	-	-	-
4	"	12	-	G/W	0	7	93	Same as above			CL	Natural	8	-	-	-	-
5	"	15	-	G/W	6	25	69	Same as above			CL	Natural	9	1410	24.5	-	-
6	"	20	-	G/W	2	4	94	Same as above			CL	Natural	10	-	-	-	-
7	"	25	-	G/W	3	18	79	Same as above			CL	Natural	11	-	-	-	-
8	"	30	-	G/W	0	15	85	Same as above			CL	Natural	12	-	-	-	-
9	"	35	-	G/W	2	11	87	Same as above			CL	Natural	13	-	-	-	-
10	"	40	-	G/W	0	59	41	Non-Plastic			SM	Natural	14	-	-	-	-
11	"	45	-	G/W	0	64	36	Non-Plastic			SM	Natural	15	-	-	-	-
12	"	50	-	G/W	0	52	48	Non-Plastic			SM	Natural	14	-	-	-	-
13	"	55	-	G/W	9	50	41	Non-Plastic			SM	Natural	16	-	-	-	-
14	"	60	-	G/W	6	56	38	Non-Plastic			SM	Natural	17	-	-	-	-
15	"	65	-	G/W	0	54	46	Non-Plastic			SM	Natural	18	-	-	-	-
16	"	70	-	G/W	10	49	41	Non-Plastic			SM	Natural	17	-	-	-	-
17	"	75	-	G/W	0	62	38	Non-Plastic			SM	Natural	19	-	-	-	-
18	"	80	-	G/W	7	48	45	Non-Plastic			SM	Natural	18	-	-	-	-
19	"	85	-	G/W	0	65	35	Non-Plastic			SM	Natural	20	-	-	-	-
20	"	90	-	G/W	0	57	43	Non-Plastic			SM	Natural	19	-	-	-	-
21	"	95	-	G/W	9	49	42	Non-Plastic			SM	Natural	21	-	-	-	
22	"	100	-	G/W	0	59	41	Non-Plastic			SM	Natural	21	-	-	-	-
-	**Min**	3	-	15.7	0	4	35	-			-	-	5	-	-	-	-
-	**Max**	100	-	G/W	10	56	94	-			-	-	21	-	-	-	-

Annexure-4

(SUMMARY OF TEST RESULTS)

Site: Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.

Serial	Borehole	Depth	Bulk Density	IMC	Grain Size Analysis (%age of Material)			Atterberg Limits			Soil	Stratum	SPT Blows	DS Test		UC Test	
No.	No.	(ft)	(pcf)	(%)	Gravel	Sand	Silt & Clay	LL	PL	PI	Type	Condition	"N"	c (psf)	ø°	qu (tsf)	Strain (%)
1	4	3	114.5	12.3	0	23	77	32	20	12	CL	Natural	6	-	-	-	-
2	"	6	116.5	13.4	0	28	72	Same as above			CL	Natural	8	-	-	-	-
3	"	9	117.0	16.0	0	30	70	Same as above			CL	Natural	9	-	-	1.07	5.30
4	"	12	-		0	29	71	Same as above			CL	Natural	10	-	-	-	-
5	"	15	-	G/W	0	24	76	Same as above			CL	Natural	11	-	-	-	-
6	"	20	-	G/W	0	27	73	Same as above			CL	Natural	13	-	-	-	-
7	"	25	-	G/W	0	55	45	Non-Plastic			SM	Natural	14	-	35.7	-	-
8	"	30	-	G/W	0	62	38	Non-Plastic			SM	Natural	15	-	-	-	-
9	"	35	-	G/W	0	68	32	Non-Plastic			SM	Natural	16	-	-	-	-
10	"	40	-	G/W	0	60	40	Non-Plastic			SM	Natural	17	-	-	-	-
11	"	45	-	G/W	0	64	36	Non-Plastic			SM	Natural	16	-	-	-	-
12	"	50	-	G/W	0	59	41	Non-Plastic			SM	Natural	18	-	-	-	-
13	"	55	-	G/W	0	61	39	Non-Plastic			SM	Natural	19	-	-	-	-
14	"	60	-	G/W	0	62	38	Non-Plastic			SM	Natural	20	-	-	-	-
15	"	65	-	G/W	0	69	31	Non-Plastic			SM	Natural	19	-	-	-	-
16	"	70	-	G/W	0	65	35	Non-Plastic			SM	Natural	21	-	-	-	-
17	"	75	-	G/W	0	60	40	Non-Plastic			SM	Natural	20	-	-	-	-
18	"	80	-	G/W	0	58	42	Non-Plastic			SM	Natural	22	-	-	-	-
19	"	85	-	G/W	0	64	36	Non-Plastic			SM	Natural	23	-	-	-	-
20	"	90	-	G/W	0	67	33	Non-Plastic			SM	Natural	22	-	-	-	-
21	"	95	-	G/W	0	68	32	Non-Plastic			SM	Natural	22	-	-	-	
22	"	100	-	G/W	0	66	34	Non-Plastic			SM	Natural	24	-	-	-	-
-	**Min**	3	114.5	12.3	-	23	32	-			-	-	6	-	-	-	-
-	**Max**	100	117.0	G/W	-	68	77	-			-	-	24	-	-	-	-

(SUMMARY OF TEST RESULTS)

Site: Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.

Serial	Borehole	Depth	Bulk Density	IMC	Grain Size Analysis (%age of Material)			Atterberg Limits			Soil	Stratum	SPT Blows	DS Test		UC Test	
No.	No.	(ft)	(pcf)	(%)	Gravel	Sand	Silt & Clay	LL	PL	PI	Type	Condition	"N"	c (psf)	ø°	qu (tsf)	Strain (%)
1	5	3	114.0	15.3	0	13	87	31	21	10	CL	Natural	5	-	-	-	-
2	"	6	-		0	30	70	Same as above			CL	Natural	7	-	-	-	-
3	"	9	-	G/W	3	25	72	Same as above			CL	Natural	7	-	-	-	-
4	"	12	-	G/W	0	24	76	Same as above			CL	Natural	8	1270	26.2	-	-
5	"	15	-	G/W	0	26	74	Same as above			CL	Natural	9	-	-	-	-
6	"	20	-	G/W	0	30	70	Same as above			CL	Natural	8	-	-	-	-
7	"	25	-	G/W	8	19	73	Same as above			CL	Natural	10	-	-	-	-
8	"	30	-	G/W	0	29	71	Same as above			CL	Natural	9	-	-	-	-
9	"	35	-	G/W	0	25	75	Same as above			CL	Natural	10	-	-	-	-
10	"	40	-	G/W	0	30	70	Same as above			CL	Natural	11	-	-	-	-
11	"	45	-	G/W	0	54	46	Non-Plastic			SM	Natural	12	-	-	-	-
12	"	50	-	G/W	0	67	33	Non-Plastic			SM	Natural	10	-	-	-	-
13	"	55	-	G/W	0	64	36	Non-Plastic			SM	Natural	12	-	-	-	-
14	"	60	-	G/W	0	61	39	Non-Plastic			SM	Natural	11	-	-	-	-
15	"	65	-	G/W	0	60	40	Non-Plastic			SM	Natural	13	-	-	-	-
-	**Min**	3	-	15.3	0	13	33	-			-	-	5	-	-	-	-
-	**Max**	65	-	G/W	8	67	87	-			-	-	13	-	-	-	-

Annexure-5

(SUMMARY OF TEST RESULTS)

Site: Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.

Serial No.	Borehole No.	Depth (ft)	Bulk Density (pcf)	IMC (%)	Grain Size Analysis (%age of Material) Gravel	Sand	Silt & Clay	Atterberg Limits LL	PL	PI	Soil Type	Stratum Condition	SPT Blows "N"	DS Test c (psf)	ø°	UC Test qu (tsf)	Strain (%)
1	6	3	112.5	13.7	0	4	96	31	18	13	CL	Natural	4	-	-	-	-
2	"	6	113.5	14.9	1	8	91	Same as above			CL	Natural	6	-	-	-	-
3	"	9	-		0	11	89	Same as above			CL	Natural	7	-	-	-	-
4	"	12	-	G/W	0	13	87	Same as above			CL	Natural	8	-	-	-	-
5	"	15	-	G/W	5	25	70	Same as above			CL	Natural	7	-	-	-	-
6	"	20	-	G/W	3	19	78	Same as above			CL	Natural	9	-	-	-	-
7	"	25	-	G/W	0	27	73	Same as above			CL	Natural	8	-	-	-	-
8	"	30	-	G/W	6	50	44	Non-Plastic			SM	Natural	10	-	32.7	-	-
9	"	35	-	G/W	5	48	47	Non-Plastic			SM	Natural	11	-	-	-	-
10	"	40	-	G/W	0	61	39	Non-Plastic			SM	Natural	12	-	-	-	-
11	"	45	-	G/W	9	51	40	Non-Plastic			SM	Natural	12	-	-	-	-
12	"	50	-	G/W	0	56	44	Non-Plastic			SM	Natural	13	-	-	-	-
13	"	55	-	G/W	0	59	41	Non-Plastic			SM	Natural	14	-	-	-	-
14	"	60	-	G/W	8	56	36	Non-Plastic			SM	Natural	13	-	-	-	-
15	"	65	-	G/W	0	54	46	Non-Plastic			SM	Natural	15	-	-	-	-
-	**Min**	3	112.5	13.7	0	4	36	-			-	-	4	-	-	-	-
-	**Max**	65	113.5	G/W	6	61	91	-			-	-	15	-	-	-	-

Annexure-6

Annexure-7

(SUMMARY OF TEST RESULTS)

Site: Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.

Serial	Borehole	Depth	Bulk Density	IMC	Grain Size Analysis (%age of Material)			Atterberg Limits			Soil	Stratum	SPT Blows	DS Test		UC Test	
No.	No.	(ft)	(pcf)	(%)	Gravel	Sand	Silt & Clay	LL	PL	PI	Type	Condition	"N"	c (psf)	ø°	qu (tsf)	Strain (%)
1	7	3	116.5	14.4	0	29	71	26	18	8	CL	Natural	7	-	-	-	-
2	"	6	117.5	16.1	0	30	70	Same as above			CL	Natural	8	-	-	-	-
3	"	9	-		0	28	72	Same as above			CL	Natural	10	-	-	-	-
4	"	12	-	G/W	0	27	73	Same as above			CL	Natural	11	-	-	-	-
5	"	15	-	G/W	0	30	70	Same as above			CL	Natural	12	-	-	-	-
6	"	20	-	G/W	0	29	71	Same as above			CL	Natural	13	-	-	-	-
7	"	25	-	G/W	0	64	36	Non-Plastic			SM	Natural	15	-	-	-	-
8	"	30	-	G/W	0	68	32	Non-Plastic			SM	Natural	16	-	-	-	-
-	**Min**	3	116.5	11.2	-	27	32	-			-	-	7	-	-	-	-
-	**Max**	30	117.5	G/W	-	68	73	-			-	-	16	-	-	-	-

Annexure-8

Bore Hole No	**BH-1**						
Project	**Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.**						
Drilling Method	**Light Percussion Machine**						

Depth (ft)	**Legend**	**SPT "N"**	**Classification**	**Description of Material**	**Sample Type**	**Field Density Test** Bulk Density, γ_b (pcf)	**Field Density Test** Moisture Content (%)
0				EXISTING GROUND LEVEL			
3		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	UDS	115.5	14.4
6		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	
9		6	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
12		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
15		8	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
20		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
25		6	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
30		8	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
35		8	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
40		9	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
45		10	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
50		9	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
55		10	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
60		11	SM	REDDISH BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
65		12	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
70		11	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
75		13	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
80		12	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
85		13	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
90		14	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
95		15	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
100		16	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
				END OF BOREHOLE			

NOTE: DS-Disturbed Sample, UDS-Undisturbed Sample, R-Refusal

Annexure-9

Bore Hole No	**BH-3**						
Project	Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.						
Drilling Method	Light Percussion Machine						

Depth (ft)	Legend	SPT "N"	Classification	Description of Material	Sample Type	Field Density Test: Bulk Density, γ_b (pcf)	Field Density Test: Moisture Content (%)
0				EXISTING GROUND LEVEL			
3		45	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	113.6	15.7
6		87	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	116.5	[illegible].3
9		98	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	UDS	117.5	14.1
12		88	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
15		99	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
20		10	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
25		11	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
30		12	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
35		13	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
40		14	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
45		15	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
50		14	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
55		16	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
60		17	SM	REDDISH BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
65		18	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
70		17	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
75		19	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
80		18	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
85		20	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
90		19	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
95		21	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
100		21	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
END OF BOREHOLE							

NOTE: DS-Disturbed Sample, UDS-Undisturbed Sample, R-Refusal

Annexure-10

Annexure-11

Bore Hole No	BH-4						
Project	Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.						
Drilling Method	Light Percussion Machine						

Depth (ft)	Legend	SPT "N"	Classification	Description of Material	Sample Type	Field Density Test: Bulk Density, γ_b (pcf)	Field Density Test: Moisture Content (%)
0				EXISTING GROUND LEVEL			
3		6	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	114.5	12.3
6		8	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	116.5	13.4
9		9	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	UDS	117.0	16.0
12		10	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	
15		11	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
20		13	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
25		14	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
30		15	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
35		16	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
40		17	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
45		16	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
50		18	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
55		19	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
60		20	SM	REDDISH BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
65		19	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
70		21	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
75		20	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
80		22	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
85		23	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
90		22	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
95		22	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
100		24	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
				END OF BOREHOLE			

NOTE: DS-Disturbed Sample, UDS-Undisturbed Sample, R-Refusal

Annexure-12

Bore Hole No	**BH-5**						
Project	**Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.**						
Drilling Method	**Light Percussion Machine**						

Depth (ft)	**Legend**	**SPT "N"**	**Classification**	**Description of Material**	**Sample Type**	**Field Density Test: Bulk Density, γ_b (pcf)**	**Field Density Test: Moisture Content (%)**
0				EXISTING GROUND LEVEL			
3		5	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	114.0	15.3
6		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	
9		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
12		8	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
15		9	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
20		8	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
25		10	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
30		9	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
35		10	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
40		11	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
45		12	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
50		10	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
55		12	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
60		11	SM	REDDISH BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
65		13	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
				END OF BOREHOLE			

NOTE: DS-Disturbed Sample, UDS-Undisturbed Sample, R-Refusal

Annexure-13

Bore Hole No	BH-6
Project	Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.
Drilling Method	Light Percussion Machine

Depth (ft)	Legend	SPT "N"	Classification	Description of Material	Sample Type	Field Density Test: Bulk Density, γ_b (pcf)	Field Density Test: Moisture Content (%)
0				EXISTING GROUND LEVEL			
3		4	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	112.5	13.7
6		6	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	113.5	14.9
9		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	
12		8	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
15		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
20		9	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
25		8	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
30		10	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
35		11	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
40		12	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
45		12	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
50		13	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
55		14	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
60		13	SM	REDDISH BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
65		15	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS		
				END OF BOREHOLE			

NOTE: DS-Disturbed Sample, UDS-Undisturbed Sample, R-Refusal

Bore Hole No	**BH-7**
Project	**Const. of Khyber Institute of Neuro Sciences & Clinical Research at Mardan.**
Drilling Method	**Light Percussion Machine**

Depth (ft)	**Legend**	**SPT "N"**	**Classification**	**Description of Material**	**Sample Type**	**Field Density Test: Bulk Density, γ_b (pcf)**	**Field Density Test: Moisture Content (%)**
0				EXISTING GROUND LEVEL			
3		7	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	116.5	14.4
6		8	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	117.5	16.1
9		10	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	
12		11	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
15		12	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
20		13	CL	LIGHT BROWN LEAN CLAY WITH TRACES OF SAND & GRAVELS	DS	-	-
25		15	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
30		16	SM	LIGHT BROWN TO GREY SILTY SAND WITH TRACES OF GRAVELS	DS	-	-
END OF BOREHOLE							

NOTE: DS-Disturbed Sample, UDS-Undisturbed Sample, R-Refusal

Annexure-14