GEOTECHNICAL CHARACTERIZATION OF RIVERINE AND COASTAL EMBANKMENT SOIL OF BANGLADESH BASED ON CONE PENETRATION TESTING AND STANDARD PENETRATION TESTING

by

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A thesis submitted to the Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, in partial fulfillment of the degree of Master of Science in Civil and Geotechnical Engineering

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ABSTRACT

The major objective of this research is to advance the state of art interpretation and application of results obtained from in-situ testing of soil, in particular Cone Penetration Test and Standard Penetration Test. CPT-SPT based soil characterization requires reliable empirical correlations among in-situ and laboratory obtained soil parameters. A large part of the research involves development of various CPT-SPT correlations for studied soil type. Correlations have been developed for the use of CPT and SPT data to evaluate shear strength parameters (c-$$\phi$$) as well as settlement parameter ($$c_s$$). $$q_c/N$$ ratio has been determined for different soil types and also as a function of index properties like Soil Behavior Index ($$I_c$$), Mean Grain Size ($$d_{50}$$) and Fines Content ($$f_c$$). New developments have been compared with previously established correlations. CPT based soil classification/stratigraphy has been prepared for riverine and coastal regions of Bangladesh. A new approach has been taken to correlate plasticity characteristics of soil with in-situ test parameters.

For this study, a total of 38 pairs of CPT and SPT have been performed in different locations from northern-most riverine embankment to the southernmost coastal embankment. Each pair of CPT and SPT has been carried out as closely as possible, maximum horizontal distance not being greater than 1m. SPT N value varies from 1-57, CPT cone tip resistance has a variation of 0.4 to 25 MPa, Soil behavior Index ($$I_c$$) varies from 1.5 to 3.5. This research demonstrates that continuous and reliable information on soil strata can be achieved through CPT. Analyses show good agreement between previously established and newly developed correlations for studied soil type. This study also illustrates that in-situ soil characterization aids easy and less conservative structural design for Bangladesh.

Statistical analysis of the data results in the formulation of empirical correlations between the various soil parameters, with often reasonably high correlation coefficients considering the wide variation of soil types tested. However, the confidence limits for these correlations range from 30-60% after performing a data filtering for the regression analysis. Due to the moderate size of the database, the proposed correlations for CPT and SPT can be considered moderately reliable for Bangladeshi embankment soils; the consistent uniformity of testing procedures is also ensured as testing has been carried out using standard techniques.
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<th>Description</th>
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<tr>
<td>$d_{50}$</td>
<td>Mean Grain Size</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Fines Content</td>
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<tr>
<td>LL</td>
<td>Liquid Limit</td>
</tr>
<tr>
<td>PL</td>
<td>Plastic Limit</td>
</tr>
<tr>
<td>PI</td>
<td>Plasticity Index</td>
</tr>
<tr>
<td>$c$</td>
<td>Cohesion Parameter</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Angle of Internal Friction</td>
</tr>
<tr>
<td>$s_u$</td>
<td>Undrained Shear Strength</td>
</tr>
<tr>
<td>$C_c$</td>
<td>Compression Index</td>
</tr>
<tr>
<td>$c_v$</td>
<td>Coefficient of Consolidation</td>
</tr>
<tr>
<td>N</td>
<td>Measured Penetration Number</td>
</tr>
<tr>
<td>$N_{60}$</td>
<td>SPT Blow Count</td>
</tr>
<tr>
<td>$N_{1,60}$</td>
<td>Corrected SPT Blow Count</td>
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<tr>
<td>$q_c$</td>
<td>Cone Tip Resistance</td>
</tr>
<tr>
<td>$q_t$</td>
<td>Corrected Cone tip Resistance</td>
</tr>
<tr>
<td>$f_s$</td>
<td>Sleev Friction</td>
</tr>
<tr>
<td>$R_f$</td>
<td>Friction Ratio</td>
</tr>
<tr>
<td>$F_r$</td>
<td>Normalized Friction Ratio</td>
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<tr>
<td>$I_c$</td>
<td>Soil Behavior Index</td>
</tr>
<tr>
<td>$\sigma_o$</td>
<td>Overburden Stress</td>
</tr>
<tr>
<td>$\sigma'_o$</td>
<td>Effective Overburden Stress</td>
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<tr>
<td>$A_R$</td>
<td>Area Ratio</td>
</tr>
<tr>
<td>$D_o$</td>
<td>Outside Diameter of the Sampling Tube</td>
</tr>
<tr>
<td>$D_i$</td>
<td>Inside Diameter of the Sampling Tube</td>
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<tr>
<td>E</td>
<td>Young’s Modulus</td>
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<td>$E_R$</td>
<td>Energy of Hammer</td>
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<tr>
<td>$R^2$</td>
<td>Coefficient of Determination</td>
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<td>M</td>
<td>Constraint Modulus</td>
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<tr>
<td>$D_R$</td>
<td>Relative Density</td>
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<tr>
<td>k</td>
<td>Permeability</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>$\psi$</td>
<td>State Parameter</td>
</tr>
<tr>
<td>$G_o$</td>
<td>Shear Modulus</td>
</tr>
<tr>
<td>$u_0$</td>
<td>In-situ Static Pore Pressure</td>
</tr>
<tr>
<td>$W$</td>
<td>Weight of Hammer</td>
</tr>
<tr>
<td>$h$</td>
<td>Height of Drop</td>
</tr>
<tr>
<td>$\eta_H$</td>
<td>Hammer efficiency (%)</td>
</tr>
<tr>
<td>$\eta_B$</td>
<td>Correction for Borehole Diameter</td>
</tr>
<tr>
<td>$\eta_S$</td>
<td>Sampler Correction</td>
</tr>
<tr>
<td>$\eta_R$</td>
<td>Correction for Rod Length</td>
</tr>
<tr>
<td>$C_N$</td>
<td>Correction Factor</td>
</tr>
<tr>
<td>$P_a$</td>
<td>Atmospheric Pressure</td>
</tr>
<tr>
<td>CI</td>
<td>Consistency Index</td>
</tr>
<tr>
<td>$\omega$</td>
<td>Natural Moisture Content</td>
</tr>
<tr>
<td>$q_u$</td>
<td>Unconfined Compression Strength</td>
</tr>
<tr>
<td>$C_u$</td>
<td>Uniformity Coefficient of Sand</td>
</tr>
<tr>
<td>$\sigma'_c$</td>
<td>Pre-consolidation Pressure</td>
</tr>
<tr>
<td>$C_p$</td>
<td>Grain Size Correlation Factor</td>
</tr>
<tr>
<td>$C_A$</td>
<td>Correlation Factor for Aging</td>
</tr>
<tr>
<td>$m_v$</td>
<td>Coefficient of Volume Compressibility</td>
</tr>
<tr>
<td>$a$</td>
<td>Cone Area Ratio</td>
</tr>
<tr>
<td>$u_2$</td>
<td>Pore Pressure Acting Behind The Cone</td>
</tr>
<tr>
<td>$A_n$</td>
<td>Cross-Sectional Area Of The Load Cell Or Shaft</td>
</tr>
<tr>
<td>$A_c$</td>
<td>Projected Area Of Cone</td>
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</table>
1.1 General

Soil is a naturally heterogeneous and discontinuous material that is composed of various degraded minerals and organic matter. Soil is deposited by several natural actions such as mechanical and chemical weathering on different kinds of rocks. These actions produce various sedimentation patterns and formations. Soil is composed of different solid minerals of finite gradations, gases and water that give different behaviors when subjected to vertical or horizontal forces.

In-situ testing has a long history in geotechnical engineering. Load bearing tests have been a part of foundation design even prior to modern soil mechanics. The standard penetration test and earlier forms of the cone penetration test were in use before 1930 and represented the main methods for early subsurface exploration, and eventually led to widely used design procedures based on empirical correlations. Evaluation of these tests and development of new, more sophisticated in-situ testing techniques have recently become the subject of renewed interest and research. The continued growth of many cities has led to increased construction of larger, more complex structures on sites with difficult ground conditions. In situations where complex structures are founded on soft, stratified soils there is relatively little evaluated experience. The uncertainties implicit in simplified and highly empirical design methods have become extremely significant. The use of more sophisticated and reliable design procedures, and their continued development, has therefore become increasingly important. This, in turn, necessitates improved capabilities for logging, measurement and selection of soil parameters, increasingly by in-situ techniques.

In situ soil investigations give readily available, relatively economic, and reliable results that can aid an engineer’s decisions and judgments on the subsurface features and his choice of foundation type. The variability of in situ test equipment, procedures, and design guidelines associated with each test, sparks the approach of correlation. Many in situ tests’ parameters have been correlated to many of soil engineering properties by means of indirect measurements. Furthermore, these in situ tests have also been correlated to other tests in order to use them for primary evaluation or initial design purposes.
Up to the present, the Standard Penetration Test (SPT) showed in Figure 1.1(left) is still one of the most commonly used in-situ tests for site investigation. Many empirical relations have been established between the SPT blow count, N-value, and other engineering properties of soils. Although geotechnical engineers use these correlations in foundation design, continued effort has been made recently for the standardization of the SPT. It is believed that the application of a measured energy correction factor will lead to more repeatability and reliability of the SPT N-value in the future (Campanella and Robertson, 1982; Kovacs and Salomone, 1982).

The cone penetration test, CPT showed in Figure 1.1(right), has been used for many years as a standard investigation tool, mainly to determine quickly the soil profile (through the friction ratio) as well as for the estimation of the undrained shear strength, $s_u$, in the case of cohesive soils. The Cone Penetration Test (CPT) is becoming increasingly popular for it’s unequalled ability to delineate soil stratigraphy and measure soil properties rapidly and continuously. Although the CPT is gaining popularity, the older in-situ apparatus like SPT, is still used in almost all geotechnical investigations, during sampling boring. Thus, correlations between SPT and CPT are of practical interest, considering the great number of existing data obtained with SPT. Similar to the use of the SPT results, many correlations need to be established for the direct application of CPT results. Before these relations can be set up, it is very valuable to correlate the cone tip resistance, $q_c$, to SPT N-value, $N_{60}$, so that the available data base of the field performances and property correlations with the N-value could be effectively utilized. Numerous geotechnical researchers have presented relationships between the two most common used in situ soil investigation tests, the SPT and the CPT. These relationships help engineers in adopting empirical methods to evaluate and analyze soil performance by converting the available database of either one of the two tests into the other test’s parameter(s). A significant amount of published literature suggests linear statistical correlations between the two tests variables: N blow count of the SPT, and cone tip resistance ($q_c$) of the CPT.

Shear strength and settlement parameters for soil are used in the design and analysis of foundations, retaining structures, and embankments, and are measured using a variety of laboratory and in-situ tests. To perform laboratory tests, such as triaxial, unconfined compression tests and one-dimensional consolidation test, relatively undisturbed samples must be recovered using an appropriate form of drilling and sampling. The sampling and subsequent laboratory testing can be a time-consuming and expensive process.
Additionally, it is often difficult to recover samples of high enough quality to obtain meaningful laboratory test results. Compared with the traditional technique of drilling, sampling, and laboratory testing, in-situ testing offers the advantage of generally being less expensive and quicker to perform. While in-situ testing is often more convenient and cost effective than laboratory strength testing, appropriate correlations must be developed for each in-situ test in order to successfully apply the results to design.

![Image of Standard Penetration Test and Cone Penetration Test](figure1.png)

**Figure 1.1** Standard Penetration Test (left) and Cone Penetration Test (right)

### 1.2 Overall Geology of Bangladesh and the Research Area

The country occupies a major part of the Bengal delta, one of the largest in the world. The Ganges-Brahmaputra delta basin or the Bengal basin includes part of the Indian state of West Bengal in the west and Tripura in the east. Geological evolution of Bangladesh is basically related to the uplift of the Himalayan Mountains and the outbuilding of deltaic landmass by major river systems originating in the uplifted Himalayas. This geology is mostly characterized by the rapid subsidence and filling of a basin in which a huge thickness of deltaic sediments was deposited as a mega-delta out built and progressed towards the south. The delta building is still continuing into the present bay of Bengal and a broad fluvial front of the Ganges-Brahmaputra-Meghna river system gradually follows it from behind. Only the eastern part of Bangladesh has been uplifted into hilly landform, incorporating itself into the frontal belt of the Indo-Burman range lying to the east. Due to such geology river, embankments are flood prone and coastal polders of Bay of Bengal have high risk of large storm surges. The research area covered the flood prone
embankments of three major rivers, Brahmaputra, Meghna and Jamuna and the coastal polders of Bay of Bengal from southwest to southeast zones.

1.3 **Testing Procedure and Evaluation of Results**

Bangladesh University of Engineering and Technology (BUET) with the financial assistance of Japan International Corporation Agency (JICA), Bangladesh carried out a significant number of Cone Penetration Tests (CPT) and Standard Penetration Tests (SPT) in 2017 as a part of Research on Disaster Prevention/Mitigation Measures against Floods and Storm Surges in Bangladesh within the framework of research project including several geotechnical investigations. The collection and analysis of this information allowed the creation of a data bank that includes the results of in-situ and laboratory tests. Cone penetration tests have been carried out with the help of a Hogentogler type piezocone penetrometer of 200 kN capacity. A mechanical cone with a friction sleeve (Begemann type) has been used having a cone with 60-degree apex angle and a diameter of 35.7mm (10 cm² cross-sectional area). The constant penetration rate was 2 cm/s. Furthermore, laboratory tests have been carried out on undisturbed samples taken from boreholes drilled close to the cone penetration test sites. Tests required for classification have been performed for all the samples in order to determine the soil type, while the undrained shear strengths of cohesive soils have been determined with the aid of triaxial tests. Plasticity characteristics and settlement parameters have been determined through Atterberg limit tests and 1-D consolidation tests respectively.

The data collected have been organized with the aid of the Excel software, so that laboratory test results correspond to in-situ test results from the same depth. The results have been recorded in detail and a geotechnical profile has been compiled for each investigated site. This database includes results from laboratory tests, standard penetration tests (SPT) and cone penetration tests (CPT).

1.4 **Objective**

The present study is aimed to characterize disaster prone soil of flood plains and coastal belt directly from in-situ testing to meet the following objectives.
(i) To develop detailed soil stratigraphy for northern-most riverine embankment to the southernmost coastal embankment through CPT

(ii) To improve the practice of interpretation and application of results obtained from in situ tests for Bangladeshi Soil.

1.5 Outline of the thesis

Following this introductory chapter, Chapter 2 presents the general description of the in-situ tests, test procedure, interpretation of test data and also limitations and advantages of these in-situ tests. An overview of previously established CPT and SPT based correlations for geotechnical parameters are also discussed in this chapter.

Chapter 3 presents the works on this study that have been carried out for the development of the interpretation of the SPT and CPT data incorporating detailed field and laboratory investigation.

Chapter 4 discusses three major analyses in three different sections. The first section discusses the development of CPT based soil stratigraphy for all the studied locations using soil behavior type. The next section illustrates CPT-SPT correlations, several approaches to determine qc/N ratio for different soil types and also as a function of soil behavior index (Ic), mean grain size (d50) and Fines content (fC). The final section demonstrates interpretation of both CPT and SPT based shear strength and settlement parameters, also, plasticity characteristics as well.

Finally, Chapter 5 presents a summary of research, main conclusions, and scope for future studies.
Chapter 2
LITERATURE REVIEW

2.1 General
The use and application of in-situ testing for the characterization of geomaterials have continued to expand over the past few decades, especially in materials that are difficult to sample and test using conventional methods. Table 2.1 and Table 2.2 present a summary of the current perceived applicability of the major in-situ tests. Professor Mitchell (1978) carried out research and published on a wide range of topics ranging from fundamentals of clay behavior to the use and interpretation of in-situ tests. In-situ testing was only a part of his extensive and impressive research record. The tables illustrate that the Standard Penetration Test (SPT), Cone Penetration Test (CPT), and its recent variations (e.g. CPTu and SCPTu) have the widest application for estimating geotechnical parameters over a wide range of materials from very soft soil to weak rock. This explains the continued growth in the use and application of the SPT and CPT worldwide. Hence, much of this paper will focus on the use and interpretation of these in situ tests.

Table 2.1 Perceived applicability of in-situ tests for geotechnical parameters (updated from Mitchell et al., 1978 and Lunne et al, 1997)

<table>
<thead>
<tr>
<th>In situ Test</th>
<th>Soil Type</th>
<th>Profile</th>
<th>u₀</th>
<th>OCR</th>
<th>Dₑ-ψ</th>
<th>φ</th>
<th>sₑ</th>
<th>Gᵦ-E</th>
<th>s-є</th>
<th>M-Cₑ</th>
<th>k</th>
<th>cᵥ</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT</td>
<td>B</td>
<td>B</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CPT</td>
<td>B</td>
<td>A</td>
<td>-</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>CPTu</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>SCPTu</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td>A</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>
Table 2.2 Perceived applicability of in-situ tests for Ground Type (updated from Mitchell et al., 1978 and Lunne et al, 1997)

<table>
<thead>
<tr>
<th>In situ Test</th>
<th>Hard Rock</th>
<th>Soft Rock</th>
<th>Gravel</th>
<th>Sand</th>
<th>Slit/Clay</th>
<th>Peat-Organic</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT</td>
<td>-</td>
<td>C</td>
<td>B</td>
<td>A</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>CPT</td>
<td>-</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>CPTu</td>
<td>-</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>SCPTu</td>
<td>-</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
</tbody>
</table>

[Note for Table 1 & 2:  
A = High, B = Moderate, C = Low, - = None  
u_i=insitu static pore pressure, OCR= Over Consolidation Ratio, D_r= Relative Density and/State Parameter, \( \phi \)=Friction angle, \( u_i \)=Undrained Shear Strength, G_r=E= Small Strain Shear or Youngs Modulus,  
\( \sigma-\varepsilon \) = Stress-Strain Relationship, M-C_c= Constraint Modulus or Compression Index, k= Permeability, c_v  
=Coefficient of Consolidation]

2.2 Role of In-Situ Testing

Before discussing in-situ tests, it is appropriate to discuss briefly the role of in-situ testing in geotechnical practice. Leroueil and Hight (2003) suggested that the appropriate level of sophistication for a site characterization program should be based on the following criteria:

- Precedent and local experience
- Design objectives
- Level of geotechnical risk
- Potential cost savings

The evaluation of geotechnical risk was described by Robertson (1998) and is dependent on the hazards (what can go wrong), probability of occurrence (how likely is it to go wrong) and the consequences (what are the outcomes). Traditional site investigation in many countries typically involves soil borings with intermittent standard penetration test (SPT) N-values at regular depth intervals (typically 1.5m) and occasional thin walled tube samples for subsequent laboratory testing. Increasingly a more efficient and cost-effective
approach is the utilization of direct-push methods using multi measurement in-situ devices, such as the seismic cone penetration test with pore pressure measurements (SCPTu) and the seismic flat dilatometer test (SDMT). Since the CPTu is about 3 to 4 times faster, collects more frequent data and is less expensive than the DMT, the CPTu is increasingly the preferred primary in-situ test.

The continuous nature of the CPTu results provides valuable information about soil variability that is difficult to match with sampling and laboratory testing. Given the above statements, emphasis in this paper will be placed on the interpretation of the CPTu. For low risk projects direct-push logging tests (e.g. CPT and CPTu) and index testing on disturbed samples combined with conservative design criteria are often appropriate. For moderate risk projects, the above can be supplemented with additional specific in-situ testing, such as the SCPTu with pore pressure dissipations combined with selective sampling and laboratory testing to develop site-specific correlations. For high risk projects, the above can be used for screening to identify potentially critical regions/zones appropriate to the design objectives, followed by possible additional in-situ tests, selective high-quality sampling and advanced laboratory testing. The results of the often-limited laboratory testing are correlated to the in-situ test results to extend and apply the results for other regions/zones within the project.

A common complaint with most direct-push in-situ tests, such as the CPT and DMT, is that they do not provide a soil sample. Although it is correct that a soil sample is not obtained during either the CPT or DMT, most commercial operators carry simple direct-push samplers that can be pushed using the same direct-push installation equipment to obtain a small (typically 25 to 50 mm in diameter) disturbed soil sample of similar size to that obtained from the SPT. The preferred approach, and often more cost-effective solution, is to obtain a detailed continuous stratigraphic profile using the CPT, then to move over a short distance (<1m) and push a small diameter soil sampler to obtain discrete selective samples in critical layers/zones identified by the CPT. The push rate to obtain samples can be significantly faster (up to 20 times) than the 20 mm/s used for the CPT and sampling can be rapid and cost effective for a small number of discrete samples.

Many of the recommendations contained in this study are focused on low to moderate risk projects where empirical interpretation tends to dominate. For projects where more advanced methods are more appropriate, the recommendations provided in this research
can be used as a screening to evaluate critical regions/zones where selective additional in-situ testing and sampling may be appropriate.

2.3 Standard Penetration Test

The standard penetration test (SPT) was developed initially by Circa (1927) and is perhaps the most popular field test. According to Sanglerat (1972), the penetrometer test evolved from the need to acquire data on subsurface soils, which could not be obtained by other means. The penetrometer measures the resistance to penetration offered by the soil at any particular depth. The test was originally designed to determine the relative density of cohesionless soils but its use has been extended to include the design of foundations by determining the load and the required embedment of piles into the bearing strata. The standard penetration test is performed by the use of the cable percussion drilling rig and its accessories.

2.3.1 The Percussion Drilling Rig

The machine used for making boreholes commonly is called a drilling rig Figure 2.1. This machine is power driven by gasoline or diesel or compressed air or electricity. There is no universal rig i.e. there is no one type of rig capable of taking every type of sample in every type of subsurface material. The cable percussion rig is used for soil investigation among other uses and is suitable for soil drilling up to a depth of approximately 50 m. It is highly portable and suitable for all terrains. To affect the drilling, some drilling tools are suspended on a cable which is alternately pulled and released to create the up and down motion of the tools. The drill hole is simply sunk by repeated dropping of one of the various tools into the ground. A power winch is used to lift the tool, suspended on a wire, and by releasing the clutch of the winch the tool drops and cuts into the soil. Once a hole is established, it is lined with casing.
2.3.1.1 The Split-spoon Sampler

Split-spoon samplers can be used in the field to obtain soil samples that are generally disturbed, but still representative. A section of a standard split-spoon sampler is shown in Figure 2.1. The tool consists of a steel driving shoe, a steel tube that is split longitudinally in half, and a coupling at the top. The coupling connects the sampler to the drill rod. The standard split tube has an inside diameter of 34.93 mm and an outside diameter of 50.8 mm; however, samplers having inside and outside diameters up to 63.5 mm and 76.2 mm, respectively, are also available. When a borehole is extended to a predetermined depth, the drill tools are removed and the sampler is lowered to the bottom of the hole. The sampler is driven into the soil by hammer blows to the top of the drill rod. The standard weight of the hammer is 622.72 N, and for each blow, the hammer drops a distance of 0.762 m. The number of blows required for a spoon penetration of three 152.4-mm intervals are recorded. The number of blows required for the last two intervals are added to give the standard penetration number, N, at that depth. This number is generally
referred to as the N value (American Society for Testing and Materials, 2018). The sampler is then withdrawn, and the shoe and coupling are removed. Finally, the soil sample recovered from the tube is placed in a glass bottle and transported to the laboratory. This field test is called the standard penetration test (SPT).

(b) 
Figure 2.2 (a) and (b) show a split-spoon sampler unassembled before and after sampling.

The degree of disturbance for a soil sample is usually expressed as

\[ A_R(\%) = \frac{(D_0^2 - D_i^2)}{D_i^2} \times 100 \]  

2.1

Where,
\[ A_R = \text{area ratio (ratio of disturbed area to total area of soil)} \]
\[ D_o = \text{outside diameter of the sampling tube} \]
\[ D_i = \text{inside diameter of the sampling tube} \]

Hence, these samples are highly disturbed. Split-spoon samples generally are taken at intervals of about 1.5 m. When the material encountered in the field is sand (particularly fine sand below the water table), recovery of the sample by a split-spoon sampler may be difficult. In that case, a device such as a spring core catcher may have to be placed inside the split spoon.

**2.3.1.2 Hammer**

Drivage is accomplished by a rope and cathead hammer (Figure 2.3) weighing 64 kg, falling from a distance of 760 mm onto the drive head, which is fitted at the top of the rods (Cernica, 1995). The blow count taken during the hammering provides a rough estimate of (but easily obtainable, very tangible and in many cases sufficiently correct) characterization of the earth material in place (Krynine and Judd, 2001).

**2.3.1.3 Drilling Rod**

A rod enclosed in a tube or sleeve (Figure 2.3) is used as a drive rod to help achieve maximum blow on the sampler. It is attached to the drive head from the top and to the sampler at the bottom. The rod is a solid steel rod, rectangular in section, with circular threaded ends to enable as many lengths to be joined together to reach the bottom of the drill hole to be sampled. The rods used for driving the sampler should have sufficient stiffness. Normally, when sampling is carried out to depths greater than around 15m, 54mm rods are used.
2.3.2 Standard Penetration Test Data Acquisition

The Standard Penetration Test is done to characterize the shear strength of engineering materials by taking note of the number of hammer blows that are required to penetrate a given depth. As the test progresses, soil samples and groundwater information are also collected. A record is made of the number of blows required to drive each 150 mm (6-in) segment into the soil. This is done until 450 mm depth is achieved or otherwise penetration refusal (Figure 2.4). The blows recorded for the first 150 mm are usually discarded because of fall-in and contamination in the hole. The number of hammer blows required to drive the sampler for the last 300 mm (12-in) is an indication of the relative density of the material and is generally referred to as the Standard Penetration Number or SPT blow count Value (N). The word “standard” is a misnomer for the Standard Penetration Test, because several methods are used in different parts of the world to release the hammer. Also, different types of anvils and rod lengths are prevalent (Budhu, 2008). Split-spoon samples (disturbed) of all are generally taken at every change of soil stratum or at specified intervals of depth, usually every 150 mm or at every change of stratum detected by the driller. Data obtained from drilling the boreholes are recorded.
accurately, completely, and at the time when the data is available. In clays and silts relatively, undisturbed samples are taken at depth intervals of 150 mm, this is done by driving thin-walled steel tube into the soil using an Akerman or a U2 hammer to its full length of 45 mm or otherwise penetration refusal. The tube is then pulled to the surface, removed from the sampling hammer, and labeled and waxed top and bottom to prevent natural moisture content from escaping. Groundwater level, where available, is also recorded during the drilling. As the drilling progresses and information regarding the strata becomes available, either through visual observations of the materials taken from samples taken by the split-spoon or Shelby-tube samplers, the information is immediately recorded. Samples that are saved for future evaluations in the laboratory are likewise properly labeled on the container in which they are preserved (a jar, a Shelby tube, or a core box). Simultaneously, that information is also recorded in the boring log shown in Figure 2.4.

![Figure 2.4 A typical Bore Log](image-url)
2.3.3 Standard Penetration Test Data Correction

The quality of test results depends on several factors, such as energy delivered to the head of the drill rod, the dynamic properties (impedance) of the drill rod, the method of drilling and borehole stabilization. At this juncture, it is important to point out that several factors contribute to the variation of the standard penetration number $N$ at a given depth for similar soil profiles. Among these factors are the SPT hammer efficiency, borehole diameter, sampling method, and rod length (Skempton, 1986); (Seed et al., 1985). The SPT hammer energy efficiency can be expressed as in Eq. (2.2).

$$E_R(\%) = \frac{\text{Actual Hammer Energy to the Sampler}}{\text{Input Energy}} \times 100$$ \hspace{1cm} 2.2

Initial Input Energy = Wh

Where,

$W =$ Weight of Hammer

$h =$ Height of Drop

In the field, the magnitude of $E_R$ can vary from 30 to 90%. The standard practice now in the Bangladesh is to express the $N$-value to an average energy ratio of 60% ($\approx N_{60}$). Thus, correcting for field procedures and on the basis of field observations, it appears reasonable to standardize the field penetration number as a function of the input driving energy and its dissipation around the sampler into the surrounding soil, or

$$N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60}$$ \hspace{1cm} 2.3

Where,

$N_{60} =$ Standard Penetration Number, corrected for field condition

$N =$ Measured Penetration Number

$\eta_H =$ Hammer Efficiency (%)

$\eta_B =$ Correction for Borehole Diameter

$\eta_S =$ Sampler Correction

$\eta_R =$ Correction for Rod Length
Variations of $\eta_H$, $\eta_B$, $\eta_S$ and $\eta_R$ based on recommendations by Seed et al. (1985) and Skempton (1986), are summarized in Table 2.3, Table 2.4, Table 2.5 and Table 2.6.

**Table 2.3** Variation of $\eta_H$

<table>
<thead>
<tr>
<th>Country</th>
<th>Hammer Type</th>
<th>Hammer Release</th>
<th>$\eta_H$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Japan</td>
<td>Donut</td>
<td>Free Fall</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>Donut</td>
<td>Rope and Pulley</td>
<td>67</td>
</tr>
<tr>
<td>United States</td>
<td>Safety</td>
<td>Rope and Pulley</td>
<td>60</td>
</tr>
<tr>
<td>Argentina</td>
<td>Donut</td>
<td>Rope and Pulley</td>
<td>45</td>
</tr>
<tr>
<td>China</td>
<td>Donut</td>
<td>Free Fall</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td>Donut</td>
<td>Rope and Pulley</td>
<td>50</td>
</tr>
</tbody>
</table>

**Table 2.4** Variation of $\eta_B$

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>$\eta_B$</th>
</tr>
</thead>
<tbody>
<tr>
<td>60-120</td>
<td>1.0</td>
</tr>
<tr>
<td>150</td>
<td>1.05</td>
</tr>
<tr>
<td>200</td>
<td>1.15</td>
</tr>
</tbody>
</table>

**Table 2.5** Variation of $\eta_S$

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>$\eta_S$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Samplar</td>
<td>1.0</td>
</tr>
<tr>
<td>With liner for dense sand and clay</td>
<td>0.8</td>
</tr>
<tr>
<td>With liner for loose sand</td>
<td>0.9</td>
</tr>
</tbody>
</table>

**Table 2.6** Variation of $\eta_R$

<table>
<thead>
<tr>
<th>Rod Length (m)</th>
<th>$\eta_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;10</td>
<td>1.0</td>
</tr>
<tr>
<td>6-10</td>
<td>0.95</td>
</tr>
<tr>
<td>4-6</td>
<td>0.85</td>
</tr>
<tr>
<td>0-4</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Hossain (2016) conducted a research on hammer efficiency for Bangladeshi context for both auto trip and manual hammer type. For manual hammer,

$$ E_r = 1.5N + 5.1 $$ 2.4
2.3.3.1 Correction for $N_{60}$ in Granular Soil

In granular soils, the value of $N$ is affected by the effective overburden pressure, $\sigma'_0$. For that reason, the value of $N_{60}$ obtained from field exploration under different effective overburden pressures should be changed to correspond to a standard value of $\sigma'_0$. That is,

$$ (N_1)_{60} = C_N N_{60} \quad \text{2.5} $$

Where, $(N_1)_{60} = \text{Standard penetration number, corrected for a standard overburden stress}$

$C_N = \text{Correction Factor}$

$N_{60} = \text{Standard penetration number, corrected for field conditions}$

In the past, a number of empirical relations were proposed for some of the relationships are given next. The most commonly cited relationships are those of Liao and Whitman (1986) and Skempton (1986).

In the following relationships for $C_N$ note that $\sigma'_0$ is the effective overburden pressure and $Pa = \text{Atmospheric pressure} \approx 100 \text{ kN/m}^2$

Liao and Whitman’s relationship (1986):

$$ C_N = \left[ \frac{1}{\left( \frac{\sigma'_0}{Pa} \right)} \right]^{0.5} \quad \text{2.6} $$

Skempton’s relationship (1986):

$$ C_N = \frac{2}{1 + \left( \frac{\sigma'_0}{Pa} \right)} \quad \text{(For normally consolidated fine sand)} \quad \text{2.7} $$

$$ C_N = \frac{3}{2 + \left( \frac{\sigma'_0}{Pa} \right)} \quad \text{(For normally consolidated coarse sand)} \quad \text{2.8} $$

$$ C_N = \frac{1.7}{0.7 + \left( \frac{\sigma'_0}{Pa} \right)} \quad \text{(For over consolidated sand)} \quad \text{2.9} $$
Seed et al.’s relationship (1985):

\[ C_N = 1 - 1.25 \log\left(\frac{\sigma'_0}{p_a}\right) \]  

2.10

Peck et al.’s relationship (1974)

\[ C_N = 0.77 \log\left(\frac{20}{\sigma'_0/p_a}\right) \text{ (for } \sigma'_0/p_a \geq 0.25) \]  

2.11

Bazara (1967):

\[ C_N = \frac{4}{1 + 4\left(\frac{\sigma'_0}{p_a}\right)} \text{ (for } \sigma'_0/p_a \leq 0.75) \]  

2.12

\[ C_N = \frac{4}{3.25 + 4\left(\frac{\sigma'_0}{p_a}\right)} \text{ (for } \sigma'_0/p_a > 0.75) \]  

2.13

2.3.4 Interpretation of Geotechnical Parameters Using SPT Data

Besides compelling the geotechnical engineer to obtain soil samples, standard penetration tests provide several useful correlations.

2.3.4.1 Soil Consistency

The consistency of clay soils can be estimated from the standard penetration number \( N_{60} \), In order to achieve that, Szechy and Varga (1978) calculated the consistency index (CI) as,

\[ CI = \frac{LL - \omega}{LL - PL} \]  

2.14

Where,

LL = Liquid limit
PL = Plastic limit

ω = Natural moisture content

The approximate correlation between CI, N$_{60}$, and the unconfined compression strength $(q_u)$ is given in Table 2.7.

Table 2.7 Approximate correlation between CI, N$_{60}$, and $q_u$

<table>
<thead>
<tr>
<th>Standard Penetration Number, N$_{60}$</th>
<th>Consistency</th>
<th>CI</th>
<th>Unconfine Compression Strength $q_u$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;2</td>
<td>Very Soft</td>
<td>&lt;0.5</td>
<td>&lt;25</td>
</tr>
<tr>
<td>2-8</td>
<td>Soft To Medium</td>
<td>0.5-0.75</td>
<td>25-80</td>
</tr>
<tr>
<td>8-15</td>
<td>Stiff</td>
<td>0.75-1.0</td>
<td>80-150</td>
</tr>
<tr>
<td>15-30</td>
<td>Very Stiff</td>
<td>1.0-1.5</td>
<td>150-400</td>
</tr>
<tr>
<td>&gt;30</td>
<td>Hard</td>
<td>&gt;1.5</td>
<td>&gt;400</td>
</tr>
</tbody>
</table>

2.3.4.2 Relative Density

Kulhawy and Mayne (1990) modified an empirical relationship for relative density that was given by Marcuson and Bieganousky (1977), which can be expressed as,

$$D_r(\%) = 12.2 + 0.75 \left[ 222N_{60} + 2311 - 711OCR - 779 \left( \frac{\sigma'_c}{p_a} \right) - 50C_u^2 \right]^{0.5} \tag{2.15}$$

Where,

$D_r$ = Relative density

$\sigma'_o$ = Effective overburden stress

$C_u$ = Uniformity coefficient of sand

$OCR = \frac{preconsolidation~pressure, \sigma'_c}{effective~overburden~pressure, \sigma'_o}$

$p_a$ = Atmospheric pressure

Meyerhof (1957) developed a correlation between $D_r$ and N$_{60}$ as,
\[ N_{60} = \left[ 17 + 24 \left( \frac{\sigma'_0}{p_a} \right) \right] D_r^2 \]  \hspace{1cm} (2.16)

\[ D_r = \left\{ \frac{N_{60}}{17 + 24 \left( \frac{\sigma'_0}{p_a} \right)} \right\}^{0.5} \]  \hspace{1cm} (2.17)

Equation (2.20) provides a reasonable estimate only for clean, medium fine sand.

Cubrinovski and Ishihara (1999) also proposed a correlation between \( N_{60} \) and the relative density of sand \( (D_r) \) that can be expressed as,

\[ D_r(\%) = \left[ \frac{N_{60} \left( 0.23 + \frac{0.06}{D_{50}} \right)^{1.7}}{9} \left( \frac{1}{\sigma'_0} \right) \right]^{0.5} \times 100 \]  \hspace{1cm} (2.18)

Where,

\[ p_a = \text{Atmospheric pressure} \approx 100kN/m^2 \]

\[ D_{50} = \text{Sieve size through which 50\% of the soil will pass (mm)} \]

Kulhawy and Mayne (1990) correlated the corrected standard penetration number and the relative density of sand in the form,

\[ D_r(\%) = \left[ \frac{(N_1)_{60}}{C_p C_A C_{OCR}} \right]^{0.5} \times 100 \]  \hspace{1cm} (2.19)

Where,

\[ C_p = \text{Grain-size correlations factor} = 60 + 25 \log D_{50} \]

\[ C_A = \text{Correlation factor for aging} = 1.2 + 0.05 \log \left( \frac{t}{100} \right) \]

\[ C_{OCR} = \text{Correlation factor for overconsolidation} = OCR^{0.18} \]
D$_{50}$ = Diameter through which 50% soil will pass through (mm)

t = Age of soil since deposition (years)

OCR = Overconsolidation ratio

Correlation between SPT N-value and relative density or compactness of sand is given in Table 2.8 (Terzaghi & Peck, 1967).

**Table 2.8 Correlation between SPT N-value and relative density**

<table>
<thead>
<tr>
<th>N-value</th>
<th>Relative Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 4</td>
<td>Very loose</td>
</tr>
<tr>
<td>4 – 10</td>
<td>Loose</td>
</tr>
<tr>
<td>10 – 30</td>
<td>Medium dense</td>
</tr>
<tr>
<td>30 – 50</td>
<td>Dense</td>
</tr>
<tr>
<td>Over 50</td>
<td>Very dense</td>
</tr>
</tbody>
</table>

2.3.4.3 Friction Angle

The peak friction angle, $\phi'$, of granular soil has also been correlated with $N_{60}$ or $(N_{i})_{60}$ by several investigators. Some of these correlations are as follows:

1. Peck et al. (1974) gave a correlation between $N_{60}$ and $\phi'$ in a graphical form, which can be approximated as Wolff (1989),

$$\varphi'(\text{deg}) = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$$

2.20
2. Schmertmann (1975) provided the correlation between $N_{60}$, $\sigma'$ and $\phi'$. Mathematically, the correlation can be approximated as Kulhawy and Mayne (1990),

$$\phi' = \tan^{-1} \left[ \frac{N_{60}}{12.2 + 20.3 \left( \frac{\sigma'_0}{p_a} \right)} \right]^{0.34} \tag{2.21}$$

Where,

$N_{60}$ = Field standard penetration number

$\sigma'_0$ = Effective overburden pressure

$p_a$ = Atmospheric pressure in the same unit as $\sigma'_0$

$\phi'$ = Soil friction angle

3. Hatanaka and Uchida (1996) provided a simple correlation between $\phi'$ and $(N_1)_{60}$ that can be expressed as,

$$\phi' = \sqrt{15.4(N_1)_{60}} + 20 \tag{2.22}$$
The following qualifications should be noted when standard penetration resistance values are used in the preceding correlations to estimate soil parameters:

i. The equations are approximate.
ii. Because the soil is not homogeneous, the values of obtained from a given borehole vary widely.
iii. In soil deposits that contain large boulders and gravel, standard penetration numbers may be erratic and unreliable.

Although approximate, with correct interpretation the standard penetration test provides a good evaluation of soil properties. The primary sources of error in standard penetration tests are inadequate cleaning of the borehole, careless measurement of the blow count, eccentric hammer strikes on the drill rod, and inadequate maintenance of water head in the borehole.

**Figure 2.6** Effective friction angle of undisturbed sands vs. normalized SPT resistance in comparison with Hatanaka and Uchida (1996) relationship
2.3.4.4 Undrained Shear Strength

Efforts have been made to find a general relationship between $s_u$ and $N_{60}$. Many such relations suggest that $c_u$ is proportional to $N_{60}$ (Terzaghi and Peck, 1967). (Stroud, 1974). Terzaghi and Peck published an approximate relationship between unconfined compressive strength $s_u$ and $N_{60}$ and stated that the scattering of $s_u$ could be very large at a given $N_{60}$. This approximate relationship was later modified by Kulhawy and Mayne (1990) to Eq. (2.23) to estimate $s_u$ of clay.

$$\frac{s_u}{p_a} = 0.06N_{60}$$ \hspace{2cm} 2.23

Some believe, it is unlikely that a generally accepted relationship between $c_u$ and $N_{60}$ will be found and a realistic correlation may be possible for clays within the same geology. Eq. (2.24) by Hara et al (1971) is an example for one such effort.

$$\frac{s_u}{p_a} = 0.29(N_{60})^{0.72}$$ \hspace{2cm} 2.24

![Figure 2.7 Relationship for undrained shear strength with N value (Hara et al. 1971)](image-url)
2.3.4.5 Over Consolidation Ratio

The over consolidation ratio, OCR, of a natural clay deposit can also be correlated with the standard penetration number. On the basis of the regression analysis of 110 data points, Mayne and Kemper (1988) obtained the relationship,

\[
OCR = 0.193 \left( \frac{N_{60}}{\sigma'_0} \right)^{0.698}
\]

Where,
\[
\sigma'_0 = \text{Effective vertical stress in MN/m}^2
\]

It is important to point out that any correlation between \(s_u\), OCR and \(N_{60}\) is only approximate.

2.3.4.6 Modulus of Elasticity

The modulus of elasticity of granular soils (\(E_s\)) is an important parameter in estimating the elastic settlement of foundations. Kulhawy and Mayne (1990) gave a first order estimation for \(E_s\) as,

\[
\frac{E_s}{p_a} = \alpha N_{60}
\]

Where,
\[
p_a = \text{Atmospheric pressure (same unit as } E_s)\]
\[
\alpha = 5 \text{ for sands with fines}
\]
\[
= 10 \text{ for clean normally consolidated sand}
\]
\[
= 15 \text{ for clean over consolidated sand}
\]

2.3.4.7 Coefficient of Volume Compressibility

The SPT is a normal site investigation method to determine soil stiffness in a lot of countries. In order to approximately calculate the coefficient of volume compressibility, \(m_v\), of stiff over consolidated clays, various empirical correlations have been introduced and are recently selected for engineering practice (Schnaid, 2009). The coefficient of
volume compressibility, $m_v$, is employed to calculate settlements for clays soils and can be gained through below equations (Stroud and Butler, 1975):

$$m_v = \frac{1}{f_2 \times N} \left( \frac{m^2}{MN} \right)$$

$f_2$ is gained from below Figure 2.8. When N-values decrease, $m_v$ values have tendency to increase.

**Figure 2.8** Correlation between coefficient $f_2$ and plasticity index (Stroud, and Butler, 1975)
2.4 Cone Penetration Test

Cone penetration test is a widely used in-situ test for the purpose of soil investigation. In the cone penetration test (CPT), a cone penetrometer attached to the ends of rods is pushed vertically into the ground at a constant rate of penetration (2 cm/sec) and the resistance of the soil at the tip and sleeve of the penetrometer is recorded. The cone penetrometer consists of the cone, friction sleeve, the sensors and the data acquisition test system, as well as the connections to the push rods. CPT is carried out by advancing a 60° apex angle cone with a diameter of 35.7 mm (10 cm² cross-sectional area) into the soil. Figure 2.9 presents the sections of cone and detailed terminology.

During the penetration, resistance to the penetration of the cone and the surface of the sleeve are recorded. The values are represented as cone tip resistance \( q_c \) and friction sleeve resistance \( f_s \). Additionally, pore water pressure, verticality and shear wave velocity can be measured by attaching additional sensors to the CPT system. The number of the readings taken must be adequate to obtain data to give a detailed picture of the variation of the measured parameters with the penetration depth. The results obtained from cone penetration test are used to evaluate the nature and sequence of soil, groundwater conditions, physical and mechanical properties of the soil. These results are used to estimate valuable parameters for geotechnical design.

![Schematic view of a cone penetrometer probe](image)

**Figure 2.9** Schematic view of a cone penetrometer probe
2.4.1 Test procedure

In order to obtain reliable and robust data from field tests with the electric penetrometers, well-trained operators, good technical back-up facilities for calibration and maintenance of the equipment used are required. During the cone penetration tests, there are certain details that must be taken care of. Criteria that must be checked during the test are given in detail following sections.

2.4.1.1 Pre-drilling, on Land Testing

In order to avoid overloading or disturbing the cone penetrometer, predrilling gain importance for performing test in fills or hard soils. Fills with coarser particles such as stones are always predrilled.

In tests in soft soils, this procedure should be done through the dry crust. The hole is filled with water to at least water table level to guarantee the maintenance of piezo element’s saturation.

In certain cases, the pre-drilling can be replaced with pre-forming hole with a solid dummy cone whose diameter is slightly larger than cone penetrometer (about 45-50 mm) through the stiffer layers (Lunne et al., 1997).

2.4.1.2 Verticality

Verticality of thrust and straightness of pushing rods are the two aspects that should be checked during penetration. The thrust machine shall be set up as to obtain vertical thrust direction. Minimum 2° exceedance can be acceptable as the deviation of the initial thrust direction from the vertical. The other point checked is the coincidence of the axis of push rods with the direction of thrust.

Once the penetrometer deflects, it continues along a path with consistent radius of curvature. 1° of deflection per meter length of standard push rods does not cause noticeable damage. On the other hand, a sudden deflection in excess of 5° over one-meter cause damage to the penetrometer end rods as a consequence of bending. Slope sensor is attached to the electric cone system; hence deflection can be monitored. When a sudden
deflection occurs, penetration should be stopped to avoid the probable damage (Lunne et al., 1997).

### 2.4.1.3 Rate of Penetration

The other point considered during cone penetration tests is the rate of penetration due to its significant effect on the measurements of pore pressure. In cone penetration tests without pore pressure measurements, the rate of penetration may be within 20 mm/s ± 25%. In the tests with pore pressure measurement, the rate of penetration is narrowed to 20 mm/s ± 10%. Excess pore pressure will start to dissipate, if any pause occurs during the penetration. To avoid the effect of dissipation on the CPT measurements, soundings are performed as continuous as possible (Lunne et al., 1997).

### 2.4.1.4 Frequency Rate

Continuous analogue data is obtained by electric cone penetrometer that converts the data to digital format in the selected interval. The chosen interval depends on the project demand. Collecting data at close intervals is preferable if information of thin layers is required. In general, the depth interval readings will be in the range of 10-50 mm. (Lunne et al., 1997).

### 2.4.2 Advantages and Limitations of CPT

The CPT has several advantages over other exploration methods, thus, the popularity of it continues to increase. The cone penetration test (CPT) is a widely used sounding procedure that has valuable outcomes to classify the materials in a soil profile and to estimate their engineering properties. Besides these main advantages, CPT test is one of the most rapid and economical exploration forms of in-situ testing. The test can be performed in a wide range of soils, although very hard soils or gravel cannot be penetrated with current technology. Significant advantage of electric cone penetration is that, it provides continuous or near continuous data.

The CPT has also a number of disadvantages. The significant limitation is that no sample is obtained during the testing process. The penetrometer cannot penetrate very dense soils
or soils contain boulder or cobbles because of the excessive force required to penetrate in these materials.

### 2.4.3 CPT Corrections

Various factors may influence the results of cone penetration test results; hence the outcomes should be corrected to obtain more accurate ones. The pore pressure around the penetrating cone is an important factor that must be taken into consideration. Cone resistance and sleeve friction measurements are influenced by the pore pressure around the penetrating cone.

Other factors that influence the measurements are the temperature changes, inclination, calibration errors and wear of the cone. In this study only, pore pressure effect is considered.

Because of the inner geometry of cone penetrometer, the ambient pore water pressure will act on the shoulder area behind the cone and on the ends of the friction sleeve. This phenomenon occurred during the cone penetration tests are defined as unequal area effect (Campanella et al., 1982). This concept has influence on the measured total stresses determined from cone and friction sleeve.

The corrected total cone resistance, $q_t$, is given in Eq. (2.28),

$$q_t = q_c + u_2 (1 - a)$$  \hspace{1cm} (2.28)

Where $u_2$ is the pore pressure acting behind the cone, $q_c$ is the measured tip resistance and $a$ is the cone area ratio. Cone area ratio, $a$, can be found by equation,

$$a = \frac{A_n}{A_c}$$  \hspace{1cm} (2.29)

where $A_n$ is the cross-sectional area of the load cell or shaft and $A_c$ is the projected area of cone, as shown in Figure 2.10.
Figure 2.10 Pore water pressure effects on measured parameters

The pore water pressure gains more importance in soft-fine grained saturated soils because of the fact that the ratio of the pore pressure to the cone resistance can be higher in these soils.

2.4.4 Soil classification based on CPT

One of the major applications of the CPT has been the determination of soil stratigraphy and the identification of soil type. This has been accomplished using charts that link cone measurements to soil type. The early charts developed in the Netherlands were based on measured cone resistance, \( q_c \), and sleeve resistance, \( f_s \), using a mechanical cone (Begemann, 1953) and showed that there is an approximate linear link between \( q_c \) and \( f_s \) for a given soil type. Early charts using \( q_c \) and friction ratio (\( R_f = f_s/q_c \) in percent) were proposed by Douglas and Olsen (1981), but the charts proposed by Robertson et al. (1986), Robertson (1990), Robertson (2009) have become very popular. Robertson stressed that the charts were predictive of soil behavior type (SBT), since the cone responds to the in situ mechanical behavior of the soil (e.g., strength, stiffness,
compressibility, and drainage) and not directly to classification criteria based on physical characteristics, such as grain-size distribution and soil plasticity.

The CPT-based normalized soil behaviour type (SBTn) method suggested by Robertson (1990) was based on the following normalized parameters:

\[ Q = \left( \frac{q_c - \sigma_{vo}}{P_{a2}} \right) \]

\[ F = \left( \frac{f_s}{(q_c - \sigma_{vo})} \right) \times 100\% \]

\[ B_q = \left( \frac{u_2 - u_1}{q_c - \sigma_{vo}} \right) \]

where \( q_t \) is the cone resistance corrected for water effects, where \( q_t = q_c + u_2(1 - a) \); \( a \) is the cone area ratio, typically around 0.8; \( \sigma_{vo} \) is the current in situ total vertical stress; \( P_{a2} \) is the current in situ effective vertical stress; \( u_2 \) is the penetration pore pressure (immediately behind cone tip); \( u_0 \) is the current in situ equilibrium water pressure; and \( u_2 - u_1 = \Delta u \) is the excess penetration pore pressure.

Robertson (1990) suggested two charts based on either \( Q - F \) and \( Q - B_q \) but recommended that the \( Q - F \) chart (illustrated in Figure 2.11) was generally more reliable, since the CPT penetration pore pressures \( (u_2) \) can suffer from lack of repeatability due to loss of saturation, especially when performed onshore at locations where the water table is deep and (or) in very stiff soils. The sleeve resistance (\( f_s \)) is often considered less reliable than the cone resistance \( (q_c) \) due to variations in cone design (e.g., Lunne et al. 1997). However, Boggess and Robertson (2010) provided recommendations on methods to improve the repeatability and reliability of sleeve resistance measurements by using cone designs with separate load cells, equal end-area sleeves, attention to tolerance requirements, and careful test procedures. Robertson (2009) also showed that, in softer soils, the SBTn charts are not overly sensitive to variations in \( f_s \). The chart shown in Figure 2.11 is based on the corrected cone resistance \( (q_t) \) that requires pore-pressure measurements to make the correction. However, the difference between \( q_c \) and \( q_t \) is generally small, except in very soft fine-grained soils. Hence, the chart in Figure 2.11 is often used successfully with the basic CPT data of \( q_c \) and \( f_s \) in most soils (i.e., \( q_c \) used in
Eq. 2.30). Since soils are essentially frictional and both strength and stiffness increase with depth, normalized parameters are more consistent with in situ soil behavior. The chart in Figure 2.11 is often referred to as the Robertson SBTn chart.

Since 1990, there have been other CPT soil behavior type charts developed [e.g., (Jeffries and Davies, 1993); (Olsen and Mitchell, 1995); (Fellenius and Eslami, 2000); (Ramsey, 2002); (Schneider et al., 2008), (Hotstream et al., 2012)]. Each chart tends to have advantages and limitations, some of which were briefly discussed by Robertson (2009). A common feature in many of these CPT-based methods is that the classification system uses groupings based on traditional physical descriptions (e.g., sand and clay) even though the methods are based on behavior measurements (e.g., either $q_c$ or $q_t$ and $f_s$). This has resulted in some confusion in geotechnical practice.

Jeffries (1993) identified that a soil behaviour type index, $I_c$, could represent the SBTn zones in the $Q_t$–$F_r$ chart where $I_c$ is essentially the radius of concentric circles that define the boundaries of soil type. Robertson and Wride (1998) modified the definition of $I_c$ to apply to the previous $Q_t$–$F_r$ chart (Robertson, 1990) as defined by,

$$I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5} \tag{2.33}$$

The contours of $I_c$ can be used to approximate the SBTn boundaries. The circular shape of the $I_c$ boundaries provides a reasonable fit to the SBTn boundaries in the center of the chart where much of the data exists for most normally to lightly over consolidated ideal soils. For some soils, the circular shape is a less effective fit to the original SBT boundaries, as illustrated in Figure 2.11 for $I_c = 2.6$ and discussed by Robertson (2009).

Robertson and Wride (1998) had suggested that $I_c = 2.6$ was an approximate boundary between soils that were either more sand like or more clay like, based on cyclic liquefaction case histories that were limited to predominately silica-based ideal soils that were essentially normally consolidated. However, experience has shown that the $I_c = 2.6$ boundary is not always effective in soils with significant microstructure. Robertson (2009) updated the normalized cone resistance and the associated SBTn chart, using a normalization with a variable stress exponent, $n$, where

$$Q_{tn} = \left(\frac{q_c - \sigma_{vo}}{P_{n2}}\right) \times \left(\frac{P_a}{\sigma'_{vo1}}\right)^n \tag{2.34}$$
Where, \((q_t - \sigma_{vo})/p_{a2}\) is the dimensionless net cone resistance; \((p_a/\sigma_{vo})^n\) is the stress normalization factor; \(p_a\) is the atmospheric reference pressure in the same units as \(q_t\) and \(\sigma_{vo}\); and \(n\) is the stress exponent that varies with SBTn, and defined by

\[
  n = 0.381(I_c) + 0.05 \left( \frac{\sigma'_{vo}}{P_a} \right) - 0.15
\]

where \(n \leq 1.0\)

The chart shown in Figure 2.11 uses \(Q_{tn}\), instead of the original \(Q_t\) suggested by Robertson (1990), and where \(I_c\) is also determined using \(Q_{tn}\) (Robertson, 2009). In most fine-gained soils, when \(I_c > 2.6\), \(Q_t \sim Q_{tn}\), since \(n \sim 1.0\). Likewise, when \(\sigma'_{vo} = 1\) atm (100 kPa) and \(\sigma'_{vo} > 10\) atm (1 MPa), \(Q_t \approx Q_{tn}\). The largest difference between \(Q_t\) and \(Q_{tn}\) occurs in coarse-grained soils at shallow depth (\(\sigma'_{vo} < 1\) atm), when \(Q_t > Q_{tn}\), (since \(n < 1.0\)). Jefferies and Been (Jefferies & Been, 2006) had suggested that the stress exponent in Eq. 2.35 should always be \(n = 1.0\). However, due to the nonlinear variation of shear stiffness \((G)\) with depth, the effective stress exponent in coarse-grained soil can be less than 1.0, as indicated by Eq. 2.35. The original method suggested by Robertson (1990) included a chart based on \(Q_t\) and \(B_q\). However, Schneider et al. (2008) showed \(Q\) and \(F\) are the normalized CPT penetration resistance and normalized friction ratio, respectively. \(Q\) is a dimensionless parameter and \(F\) is in percent. \(q_c\) and \(f_s\) are the cone tip resistance and friction sleeve resistance, respectively and are recorded during penetration. \(\sigma_{vo}\) and \(\sigma'_{vo}\) are the total and effective overburden stresses and \(P_{a2}\) and \(P_a\) are the reference pressures that have the same units as \(q_c\), \(\sigma_{vo}\) and \(\sigma'_{vo}\) respectively.

The original method suggested by Robertson (1990) included a chart based on \(Q_t\) and \(B_q\). However, Schneider et al (2008) showed that \(B_q\) may not be the best form of normalized CPT pore pressure to identify soil type and suggested a chart based on \(Q_t\) and \(U_2\) (where \(U_2 = \Delta u_2/\sigma'_{vo}\)). The Schneider et al. (2008) pore-pressure chart applies mostly to claylike soils, since it requires a measured excess penetration pore pressure \((\Delta u_2)\) and was developed primarily to aid in separating whether CPT penetration is drained, undrained, or partially drained. The Schneider et al. (2008) \(Q_t-U_2\) chart uses slightly different grouping of soil type (and description terms) compared to the Robertson (1990) chart, which can also lead to some confusion when applying both.
Robertson (2010) provided an update to utilize non-normalized parameters with a non-normalized SBT chart shown in Figure 2.12 where the boundaries are also essentially concentric circles and a non-normalized Soil Behavior Type Index, $I_{SBT}$ can also be defined by:

$$I_{SBT} = \left[\left((3.47 - \log(q_t/p_a))\right)^2 + \left(\log R_f + 1.22\right)^2\right]^{0.5}$$ \hspace{1cm} 2.36

where:

$q_c =$ CPT cone resistance (or corrected cone resistance, $q_t$)

$R_f =$ friction ratio = $(f_s/q_c)\times100\%$

$f_s =$ CPT sleeve friction

The non-normalized SBT index ($I_{SBT}$) is essentially the same as the normalized SBTn index ($I_c$) but only uses the basic CPT measurements. In general, the normalized $I_c$ provides more reliable identification of SBT than the non-normalized $I_{SBT}$, but when the in-situ vertical effective stress is between 50 kPa to 150 kPa there is often little difference between normalized and non-normalized SBT.

There is now more than 25 years of experience using the Robertson SBTn chart, and in general, the chart provides good agreement between USCS-based classification and CPT-based SBTn [e.g., (Molle, 2005)] for most soils with little microstructure. Robertson (2009) discussed several examples where there can be observed differences. The Robertson SBTn chart tends to work well in ideal soils (i.e., unstructured soils) but can be less effective in structured soils. That leads to an updated SBTn chart which includes a method to identify the existence of microstructure in soils (Robertson, 2016).
Figure 2.11 CPT-based SBTn chart suggested by Robertson (1990)
Figure 2.12 Non-normalized SBT chart updated by Robertson (2010)
In this paper soil classification based on CPT has been done with the non-normalized SBT index to use non-normalized CPT results according to Roberson (2010). As mentioned before, the boundaries between soil behavior type zones 2 to 7 can be approximated as concentric circles 4. The radius of each circle can be defined as a soil behavior type index, $I_c$. Hence, the soil behavior type index does not apply to zones 1, 8 and 9. The variation of soil behavior type index, $I_c$ with soil behavior type zones are given in Table 2.9.

**Table 2.9** Boundaries of soil behavior type (Robertson, 1990)

<table>
<thead>
<tr>
<th>Soil Behavior Type Index, $I_c$</th>
<th>Zone</th>
<th>Soil Behavior Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I_c&lt;1.31$</td>
<td>7</td>
<td>Gravelly sand to dense sand</td>
</tr>
<tr>
<td>$1.31&lt;I_c&lt;2.05$</td>
<td>6</td>
<td>Sands: clean sand to silty sand</td>
</tr>
<tr>
<td>$1.05&lt;I_c&lt;2.60$</td>
<td>5</td>
<td>Sand Mixtures: silty sand to sandy silt</td>
</tr>
<tr>
<td>$2.60&lt;I_c&lt;2.95$</td>
<td>4</td>
<td>Silt Mixtures: clayey silt to silty clay</td>
</tr>
<tr>
<td>$2.95&lt;I_c&lt;3.60$</td>
<td>3</td>
<td>Clays: silty clay to clay</td>
</tr>
<tr>
<td>$3.60&lt;I_c$</td>
<td>2</td>
<td>Organic soils: peats</td>
</tr>
</tbody>
</table>

2.4.5 **Interpretation of Shear Strength Parameters Using CPT Data**

2.4.5.1 **Commonly used $q_c$–$\phi$ Correlations**

Significant advances have been made in the development of theories to model the cone penetration process in sands (Yu and Mitchell, 1998). Cavity expansion models show the most promise since they are relatively simple and can incorporate many of the important features of soil response. However, empirical correlations based on calibration chamber test results and field results are still the most commonly used.

Robertson and Campanella (1983) suggested a correlation to estimate the peak friction angle ($\phi'$) for uncemented, unaged, moderately compressible, predominantly quartz sands based on calibration chamber test results. For sands of higher compressibility (i.e. carbonate sands or sands with high mica content), the method will tend to predict low friction angles.

$$
tan\phi' = \frac{1}{2.68} \left[ \log \left( \frac{q_c}{\sigma'_{vo}} \right) + 0.29 \right]
$$

2.37
Kulhawy and Mayne (1990) suggested an alternate relationship for clean, rounded, uncemented quartz sands, and evaluated the relationship using high quality field data:

\[
\phi' = 17.6 + 11\log(Q_{tn})
\]

Jefferies and Been (2006) showed a strong link between state parameter (\(\psi\)) and the peak friction angle (\(\phi'\)) for a wide range of sands. Using this link, it is possible to link \(Q_{tn,cs}\) with \(\phi'\), using:

\[
\phi' = \phi'_{cv} - 48\psi
\]

Where \(\phi'_{cv}\) = constant volume (or critical state) friction angle depending on mineralogy (Bolton, 1986), typically about 33 degrees for quartz sands but can be as high as 40 degrees for felspathic and carbonate sands.

Hence, the relationship between normalized clean sand equivalent cone resistance, \(Q_{tn,cs}\) and \(\phi'\) becomes:

\[
\phi' = \phi'_{cv} + 15.84\log(Q_{tn,cs}) - 26.88
\]

The above relationship produces estimates of peak friction angle for clean quartz sands that are like those by Kulhawy and Mayne (1990). However, the above relationship based on state parameter has the advantage that it includes the importance of grain characteristics and mineralogy that are reflected in both \(\phi'_{cv}\), as well as soil type through \(Q_{tn,cs}\). The above relationship tends to predict \(\phi'\) values closer to measured values in calcareous sands where the CPT tip resistance can be low for high values of \(\phi'\).

For fine-grained soils, the best means for defining the effective stress peak friction angle is from consolidated triaxial tests on high quality undisturbed samples. An assumed value of \(\phi' = 28^\circ\) for clays and \(32^\circ\) for silts is often sufficient for many low-risk projects. Alternatively, an effective stress limit plasticity solution for undrained cone penetration developed at the Norwegian Institute of Technology (Senneset et al, 1989) allows the approximate evaluation of effective stress parameters (\(c'\) and \(\phi'\)) from piezocone (\(u_2\)) measurements. In a simplified approach for normally- to lightly over consolidated clays and silts (\(c' = 0\)), the NTH solution can be approximated for the following ranges of parameters: \(20^\circ \leq \psi \leq 40^\circ\) and \(0.1 \leq Bq \leq 1.0\) (Mayne, 2006):
\[
\varphi'(\text{deg}) = 29.5^\circ \times B_q^{0.121}[0.256 + 0.336B_q + \log Q_t]
\]

2.41

For heavily over consolidated soils, fissured geomaterials, and highly cemented or structured clays, the above will not provide reliable results and should be determined by laboratory testing on high quality undisturbed samples. The above approach is only valid when positive \(u_2\) pore pressures are recorded (i.e. \(B_q > 0.1\))

Minmura (2003) did CIDC Triaxial Results on Frozen Sand Samples and modified the correlation provided by Kulhawy and Mayne (1990) as bellow:

\[
\varphi'(\text{deg}) = 17.6 + 11\log\left(\frac{q_t}{\sigma_{vp} \times \sigma_{atm}}\right)^{0.5}
\]

2.42

2.4.5.2 Commonly used \(q_c−s_u\) Correlations

No single value of undrained shear strength, \(s_u\), exists, since the undrained response of soil depends on the direction of loading, soil anisotropy, strain rate, and stress history. Typically, the undrained strength in tri-axial compression is larger than in simple shear that is larger than tri-axial extension \(s_u\text{TC} > s_u\text{SS} > s_u\text{TE}\). The value of \(s_u\) to be used in analysis therefore depends on the design problem. In general, the simple shear direction of loading often represents the average undrained strength \(s_u\text{SS} \sim s_u(\text{ave})\).

Since anisotropy and strain rate will inevitably influence the results of all in-situ tests, their interpretation will necessarily require some empirical content to account for these factors, as well as possible effects of sample disturbance.

Theoretical solutions have provided valuable insight into the form of the relationship between cone resistance and \(s_u\). All theories result in a relationship between corrected cone resistance, \(q_t\), and \(s_u\) of the form,

\[
s_u = \frac{(q_t - \sigma_v)}{N_{kt}}
\]

2.43

Typically, \(N_{kt}\) varies from 10 to 18, with 14 as an average for \(s_u(\text{ave})\). \(N_{kt}\) tends to increase with increasing plasticity and decrease with increasing soil sensitivity. Lunne et al. (1997)
showed that $N_{kt}$ decreases as $B_q$ increases. In very sensitive fine-grained soil, where $B_q \sim 1.0$, $N_{kt}$ can be as low as 6.

For deposits where little experience is available, estimate $s_u$ using the corrected cone resistance ($q_t$) and preliminary cone factor values ($N_{kt}$) from 14 to 16. For a more conservative estimate, select a value close to the upper limit.

In very soft clays, where there may be some uncertainty with the accuracy in $q_t$, estimates of $s_u$ can be made from the excess pore pressure ($\Delta u$) measured behind the cone ($u_2$) using the following:

$$s_u = \frac{\Delta u}{N_{\Delta u}}$$  \hspace{1cm} (2.44)

Where $N_{\Delta u}$ varies from 4 to 10. For a more conservative estimate, select a value close to the upper limit. Note that $N_{\Delta u}$ is linked to $N_{kt}$, via $B_q$, where:

$$N_{\Delta u} = B_q \times N_{kt}$$  \hspace{1cm} (2.45)

If previous experience is available in the same deposit, the values suggested above should be adjusted to reflect this experience.

For larger, moderate to high risk projects, where high quality field and laboratory data may be available, site-specific correlations should be developed based on appropriate and reliable values of $s_u$.

$N_{kt}$ is the cone factor that depends on soil stiffness, OCR and soil sensitivity, but experience has shown that soil sensitivity has the largest influence. $N_{kt}$ can be linked to soil sensitivity via the normalized friction ratio, $F_r$, and can be represented approximately by Robertson (2012),

$$N_{kt} = 10.5 + 7 \log(F_r)$$  \hspace{1cm} (2.46)
2.4.6 Correlations between CPT Data and Compressibility of Soils

Deformation characteristics of soils are generally expressed by one dimensional constrained modulus, \( M \), undrained Young’s modulus, in compression loading, \( E_u \) and small-strain shear modulus, \( G_o \). One-dimensional consolidation settlement is based on the assumption that the lateral strain is equal to zero. Hence, the appropriate parameter to define the deformation characteristics of soils in the consolidation process is the one-dimensional constrained modulus, \( M \). One-dimensional constrained modulus, \( M \) is defined as follows;

\[
M = \frac{d\sigma'_V}{d\varepsilon_V} \quad 2.47
\]

\[
M = \frac{1}{m_V} \quad 2.48
\]

In which \( \sigma'_V \) is the vertical effective stress, \( \varepsilon_V \) is the vertical strain, and \( m_V \) is the coefficient of volume compressibility.

After an extensive literature survey, available methods based on the CPT data for determining the compressibility characteristics of soils have been complied. Sanglerat (1972), Jones and Rust (1995), Senneset et al. [(1982), (1989)] and Kulhawy and Mayne (1990) suggested a model for estimating constrained modulus of clays. Moreover, Manas and Linaresh (1995), Furmonavicius and Dagys (1995) and Mahesh and Vikash (1995) also proposed a correlation for clays. Lunne and Christophersen (1983) suggested a method for over consolidated and normally consolidated sands. All these methods will be discussed next. The 1-D constrained modulus is expressed in terms of the cone tip resistance and coefficient, \( m_m \) as given in Eq. (2.49). Sanglerat (1972) proposed a comprehensive relationship for the coefficient, \( m_m \) for a wide range of the cone tip resistance \( (q_c) \) and type of the soils. This correlation is presented in Table 2.10.

\[
M = a_m q_c \quad 2.49
\]
**Table 2.10** Estimations of coefficient, $\alpha_m$ to find the constrained modulus, $M$, for clays
(Sanglerat, 1972)

<table>
<thead>
<tr>
<th>Cone Tip Resistance $q_c$ (MPa)</th>
<th>The range of Coefficient $\alpha_m$</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_c&lt;0.7$</td>
<td>$3.0&lt;\alpha_m&lt;8.0$</td>
<td>Clay of low plasticity (CL)</td>
</tr>
<tr>
<td>$0.7&lt;q_c&lt;2.0$</td>
<td>$2.0&lt;\alpha_m&lt;5.0$</td>
<td></td>
</tr>
<tr>
<td>$q_c&gt;2.0$</td>
<td>$1.0&lt;\alpha_m&lt;2.5$</td>
<td></td>
</tr>
<tr>
<td>$q_c&gt;2.0$</td>
<td>$3.0&lt;\alpha_m&lt;6.0$</td>
<td>Silts of low plasticity (ML)</td>
</tr>
<tr>
<td>$q_c&lt;2.0$</td>
<td>$1.0&lt;\alpha_m&lt;3.0$</td>
<td></td>
</tr>
<tr>
<td>$q_c&lt;2.0$</td>
<td>$2.0&lt;\alpha_m&lt;6.0$</td>
<td>High plastic silts and clays (MH, CH)</td>
</tr>
<tr>
<td>$q_c&gt;1.2$</td>
<td>$2.0&lt;\alpha_m&lt;8.0$</td>
<td>Organic silts (OL)</td>
</tr>
</tbody>
</table>

After the data in Table 2.10 were studied, it was concluded that the value of coefficient, $\alpha_m$ gets smaller with the increase in the tip resistance. This observation is compatible with the observation that compressibility decreases with increased effective stress and increased strength.

Another correlation developed by Kulhawy and Mayne (1990) has a more general form as shown in Eq. (2.50) and does not depend on the soil type or the amount of the tip resistance. General relationship is expressed as follows:

$$ M = 8.25 \ (q_t - \sigma_o) \quad 2.50 $$

The above correlation is also presented in the graphical form in Figure 2.13

An alternative correlation for the determination of constrained modulus is given by Senneset et al. [(1982), (1989)]. Two different relations were developed for normally consolidated and over consolidated clays. He proposed a linear interpretation model for over consolidated clays, in the form of:

$$ M_i = \alpha_i(q_t - \sigma_o) \quad 2.51 $$

The coefficient, $\alpha_i$ ranges from 5 to 15 for over consolidated clays.
For normally consolidated clays the following relationship is recommended:

\[ M_n = \alpha_n (q_t - \sigma_o) \]  

2.52

According to the methods of Senneset et al. (1989) the coefficient, \( \alpha_n \) is in the range of 4 to 8 for normally consolidated clays. Manas and Lineresh (1995) suggested that the value of coefficient of one-dimensional constrained modulus, \( \alpha_n \) is equal to 0.8 for soft clays. On the other hand, Furmonavicius and Dagys (1995) has indicated that, a representative value of 10 can be used for the coefficient, \( \alpha_n \) for clays. Mahesh et al (1995) proposed another relationship for the coefficient, \( \alpha_n \) for a wide range of the cone tip resistance (\( q_c \)) and type of clays. This correlation is presented in Table 2.11.
2.5 SPT-CPT Correlation

A considerable number of studies have taken place over the years to quantify the relationship between SPT N value and CPT cone bearing resistance, $q_c$. A wide range of $q_c/N$ ratios has been published leading to much confusion. Most of the empirical correlations considered a constant value ($n$) of $n=q_c/N$ and some other researchers proposed $n=(q_c+f_s)/N$ for different soil types as shown in Table 2.12. Where $q_c$ is the cone tip resistance, $f_s$ is the frictional resistance, and $N$ is the SPT blow count.

Sanglerat (1972) cites Meyerhof (1965) who suggested a relationship $n = (q_c/N) = 0.4$ ( $q_c$ in MPa); but further, Meigh and Nixon (1961) showed that this simple relationship did not take into account the effect of grain size and made comparative tests in sand and gravel. They have recommended $n$-values as 0.2 MPa for coarse sand and 0.3 –0.4 MPa for gravelly sand. Engineers of Franki Piles have found the $n$-values as 1.0 MPa for sand, 0.6 MPa for clayey sand, 0.5 MPa for silty sand, 0.4 MPa for sandy clay, 0.3 MPa for silty clay and 0.2 MPa for clays. De Alencar Velloso (1959) gave $n$ ratios for different types of soils and found as 0.35 MPa for clay and silty clay, 0.2 MPa for sandy clay and silty sand, 0.35 MPa for sandy silt, 0.6 MPa for fine sand and 1.0 MPa for sand (De Alencar Velloso, 1959).

Schmertmann (1970) pointed out that if only SPT data are available, these can be converted to cone penetrometer values. He suggested 0.2 MPa for silt, sandy silt and slightly cohesive silt – sand mixture, 0.3– 0.4 MPa for clean, fine to medium sand and slightly silty sand, 0.5 – 0.6 MPa for coarse sand and sand with little gravel and 0.8– 1.0 MPa for sandy gravel and gravel. He also suggested that instead of correlation of $q_c$ with

<table>
<thead>
<tr>
<th>Cone Tip Resistance $q_c$(MPa)</th>
<th>The range of Coefficient $\alpha_m$</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5&lt;$q_c$&lt;6.0</td>
<td>$\alpha_n$=1.37</td>
<td>Clay of low plasticity (CL)</td>
</tr>
<tr>
<td>4.0&lt;$q_c$&lt;4.5</td>
<td>$\alpha_n$=1.67</td>
<td></td>
</tr>
<tr>
<td>2.5&lt;$q_c$&lt;4.0</td>
<td>1.7&lt;$\alpha_m$&lt;2.8</td>
<td></td>
</tr>
<tr>
<td>0.5&lt;$q_c$&lt;2.5</td>
<td>2.8&lt;$\alpha_m$&lt;6.1</td>
<td>Clays of high plasticity</td>
</tr>
</tbody>
</table>

Table 2.11 Estimations of coefficient, $\alpha_n$ to find the constrained modulus, $M$, for clays Mahesh et al. (1995)
N-value, better correlation with $f_s$ (sleeve friction) and N-value can be established, especially for cohesive soils.

Robertson and Campanella (1983) presented the $qc/N$ ratio as a function of mean grain size, ‘$D_{50}$’. They proposed a soil behavior-type classification, giving $qc/N$ ratio for each soil classification zone based on cone penetration test with pore pressure measurement tests (CPTU, piezocone).

**Table 2.12 Previous Literature Summary**

<table>
<thead>
<tr>
<th>Researcher [s]</th>
<th>Soil type</th>
<th>Proposed relationship</th>
</tr>
</thead>
<tbody>
<tr>
<td>De Alencar Vellso (1959)</td>
<td>Clay and silty day</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.35$</td>
</tr>
<tr>
<td></td>
<td>Sandy clay and silty sand</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sandy silt</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.2$</td>
</tr>
<tr>
<td></td>
<td>Fine sand</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.35$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$n = \left( \frac{qc}{N} \right) = 0.6$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$n = \left( \frac{qc}{N} \right) = 1.0$</td>
</tr>
<tr>
<td>Meigh and Nixon(1961)</td>
<td>Coarse sand</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.2$</td>
</tr>
<tr>
<td></td>
<td>Gravelly sand</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.3 - 0.4$</td>
</tr>
<tr>
<td>Engineers Franki Piles (1960)</td>
<td>Sand</td>
<td>$n = \left( \frac{qc}{N} \right) = 1.0$</td>
</tr>
<tr>
<td>Schmertmann (1970)</td>
<td>Silt, sandy silt and silt-sand mix.</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.2$</td>
</tr>
<tr>
<td></td>
<td>Fine to medium sand, silty sand</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.3 - 0.4$</td>
</tr>
<tr>
<td></td>
<td>Coarse sand, sand with gravel</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.5 - 0.6$</td>
</tr>
<tr>
<td></td>
<td>Sandy gravel and gravel</td>
<td>$n = \left( \frac{qc}{N} \right) = 0.8 - 0.1$</td>
</tr>
<tr>
<td>Barata et al., (1975)</td>
<td>Sandy sky clay</td>
<td>$n = \left( \frac{qc}{N} \right) = 1.5 - 2.5$</td>
</tr>
<tr>
<td>(qc/N) in bar/30cm</td>
<td>Clayey silty sand</td>
<td>$n = \left( \frac{qc}{N} \right) = 2.0 - 3.5$</td>
</tr>
<tr>
<td>Researcher [s]</td>
<td>Soil type</td>
<td>Proposed relationship</td>
</tr>
<tr>
<td>----------------</td>
<td>-----------</td>
<td>-----------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( n = \left(\frac{q_c}{N}\right) = 1.5 - 2.5 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( n = \left(\frac{q_c}{N}\right) = f(D_{50}) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( n = \left(\frac{q_c}{N}\right) = f(F_c) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( n = \left(\frac{q_c}{N}\right) = 3.2 ) ( n = \left(\frac{q_c}{N}\right) = 4.2 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( n = \left(\frac{q_c}{N}\right) = 2.1 ) ( n = \left(\frac{q_c}{N}\right) = 1.8 )</td>
</tr>
<tr>
<td></td>
<td>Silt, sandy silt and silt-sand</td>
<td>( n = \left[\frac{q_c + f_s}{N}\right] = 0.2 ) ( n = \left[\frac{q_c + f_s}{N}\right] = 0.3 - 0.4 ) ( n = \left[\frac{q_c + f_s}{N}\right] = 0.5 - 0.6 ) ( n = \left[\frac{q_c + f_s}{N}\right] = 0.8 - 1.0 ) ( n = \left[\frac{q_c + f_s}{N}\right] = 7.0 )</td>
</tr>
<tr>
<td></td>
<td>Fine to medium sand, silty sand</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Coarse sand, sand with gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sandy gravel and gravel</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Silty sand</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sand</td>
<td>( n = \left(\frac{q_c}{N}\right) = 5.7 ) ( n = \left(\frac{q_c}{N}\right) = 5.0 - 6.4 ) ( n = \left(\frac{q_c}{N}\right) = 3.1 ) ( n = \left(\frac{q_c}{N}\right) = 1.0 - 3.5 ) ( n = \left(\frac{q_c}{N}\right) = 4.6 - 5.3 ) ( n = \left(\frac{q_c}{N}\right) = 1.8 - 3.5 ) ( n = \left(\frac{q_c}{N}\right) = 4.5 )</td>
</tr>
<tr>
<td></td>
<td>Silty sand, Silty clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Clayey silt</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Clay, silt and sand mixtures</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Clayey sand and silty clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sandy clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Clay</td>
<td></td>
</tr>
<tr>
<td></td>
<td>All soil</td>
<td>( n = \left(\frac{q_c}{N}\right) = f(I_c) )</td>
</tr>
<tr>
<td>Researcher [s]</td>
<td>Soil type</td>
<td>Proposed relationship</td>
</tr>
<tr>
<td>------------------------</td>
<td>--------------------------</td>
<td>-----------------------------------------------------------</td>
</tr>
<tr>
<td>Emrem and Durgunoglu</td>
<td>Turkey soils</td>
<td>( n = \left( \frac{q_c}{N} \right) = f(D_{50}) )</td>
</tr>
<tr>
<td>(2000)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chin et al (1988)</td>
<td>Granular soil</td>
<td>( n = \left( \frac{q_c}{N} \right) = f(fc) )</td>
</tr>
<tr>
<td>Acka (2003)</td>
<td>Sand</td>
<td>( n = \left( \frac{q_c}{N} \right) = 0.77 )</td>
</tr>
<tr>
<td></td>
<td>Silty sand</td>
<td>( n = \left( \frac{q_c}{N} \right) = 0.70 )</td>
</tr>
<tr>
<td></td>
<td>Sandy-sift</td>
<td>( n = \left( \frac{q_c}{N} \right) = 0.58 )</td>
</tr>
</tbody>
</table>

Robertson and Campanella suggested that the variations in published \( q_c/N \) ratio can be rationalized somewhat by reviewing the derived \( q_c/N \) ratios as a function of mean grain size \( (D_{50}) \), as shown in Figure 2.14. It is clear that the \( q_c/N \) ratio increases with increasing grain size. The scatter in results appears to increase with increasing grain size. This is not surprising since penetration in gravelly sand \( (D_{50} = 1.0 \text{ mm}) \) is significantly influenced by the larger gravel sized particles, not to mention the variability of delivered energy in the SM. data. Also sand deposits in general are usually stratified or non-homogeneous causing rapid variations in CPT tip resistance.

Jefferies and Davies (1993) correctly suggested that the most reliable way to obtain SPT \( N \) values was to perform a CPT and convert the CPT to an equivalent SPT. They suggested a method to convert the CPT cone resistance, \( q_c \), to an equivalent SPT \( N \) value at 60% energy, \( N_{60} \), using a soil behavior type index, \( I_c \). The method was modified slightly by Lunne et al. (1997), based on the simpler soil behavior type index defined by Robertson and Wride (1998), as follows:

\[
\left( \frac{q_1}{p_a} \right)_{N_{60}} = 8.5 \left[ 1 - \frac{I_c}{4.6} \right]
\]

2.53

Where \( q_1 \) is the corrected cone tip resistance and \( I_c \) is the soil behavior type index defined by Robertson and Wride (1998). The above method has been shown to work effectively in a wide range of soils, although recent experience in North America has shown that Eq. (2.53) tends to under predict the \( N_{60} \) values in some clays.
Figure 2.14 Mean grain size based CPT-SPT correlation, Robertson (1982)

Figure 2.15 compares various relationships of \( \frac{q_t/p_a}{N_{60}} \) as a function of \( I_c \) and presents a suggested updated relationship (Robertson, 2012) that can be defined by the following:

\[
\left( \frac{q_t}{p_a} \right)_{N_{60}} = 10\{1.1268 - 0.2817I_c\}
\]

Eq. 2.54 produces slightly larger \( N_{60} \) values in fine-grained soils than the previous Jefferies and Davies (1993) method. In fine-grained soils with high sensitivity, Eq. (2.54) may overestimate the equivalent \( N_{60} \).
Kasim et al. (1986) and Chin et al. (1988) suggested that $q_c/N$ for granular soils (except gravelly sand) may be better reflected by the fines content rather than mean grain size. They emphasize that $q_c/N$ ratio for fines content is more convenient to use rather than mean grain size.

Although many authors proposed different correlations, it is quite recognisable that authors did not indicate the geology and geomorphology in their correlative works. The only indication of geology was given by Robertson et al. (1983), where they mentioned over consolidation. Ismael and Jeragh (1986) made a correlation on calcareous desert sands in Kuwait and compared it with the value of Schmertmann (1970) for clean, fine to medium sands and slightly silty sands. Their test value was higher than what Schmertmann suggested for clean, fine to medium sands and slightly silty sands. By comparing test results with the historical data of Robertson et al. (1983), they also found close agreement in the form of $q_c/N$ versus mean grain size ‘$D_{50}$’. Danziger and de Velloso (1995) made a correlation between CPT and SPT for some Brazilian soils. Values found were in the same range obtained by Schmertmann (1970).
Different types of correlation were tested, and a linear correlation was found better suited for practical applications. A general trend was obtained in a similar pattern of Robertson’s curve (increasing n-values with increasing grain size). Most authors were apparently satisfied with an approximate relationship, indicating more or less an average. None of them mentioned a standard deviation or correlation coefficient. Much recent work has demonstrated that use of statistical methods would greatly assist in not only improving correlations but also indicate the dependability of the correlations derived. It is significant that such techniques have not been tried in improving correlation between SPT and CPT. A possible reason may be that along a vertical line, each 0.05 m, a reading of an electrical CPT is available, whereas for only each 0.5 m, an SPT result is determined. So over a depth of 1 m, there are 20 readings of qc and at maximum two N values.

(Acka, 2003) proposed SPT-CPT correlation for United Arab Emirates Soils. Results of his study showed higher values of \( n = \frac{q_c}{N} \) when compared to values found in the literature. He compared the R value for acceptability of the correlations using both arithmetic and statistical method. He explained that higher values are due to cementation, densification and Shelly structure or gravel layers in the United Arab Emirates soils.

Shahri et al. (2014) proposed a correlation between \( q_c \) and N-value for various soil layers, particularly in clayey soils with significant clay content in an area in southwest Sweden.

**Figure 2.16** Correlation Proposed by Chin et al (1988)
They proposed linear and power relationships to predict $qc$ using $N$-value. The results of their study showed a good agreement with previous work by other researchers.

2.6 Summary

From the literature three things can be concluded. The first one is, there is well defined methods to interpret CPT data for soil classification as per Robertson (2009, 2010) using soil behavior type/index. The second is, there is several approaches to correlate CPT with SPT depending on different physical properties of soil e.g. soil type [Danziger (1995, 1998), Acka (2003)] or soil behavior type [Jefferies and Davis (1993)], grain size [Robertson (1982)] or percent fines [Chin et al (1988)]. And, finally, mechanical properties of soil like strength [Robertson and Campenella(1983), Robertson (2010), Kulhawy and Mayne (1990), Hatanaka and Uchida (1996)] or settlement parameters [Sanglaret (1972)] can be estimated from in-situ test results.
Chapter 3
GEOTECHNICAL INVESTIGATION

3.1 General

This chapter presents the details of the geotechnical investigation that had been carried out to generate the data bank to develop empirical correlations for the CPT-SPT based geotechnical characterization. Bangladesh University of Engineering and Technology (BUET) with the financial assistance of Japan International Corporation Agency (JICA) Bangladesh, carried out a significant number of Cone Penetration Tests (CPT) and Standard Penetration Tests (SPT) in 2017 as a part of Research on Disaster Prevention/Mitigation Measures against Floods and Storm Surges in Bangladesh which provided the field investigation data for this thesis. Necessary laboratory tests on the sample retrieved from those investigations have been performed by the author in the geotechnical laboratory in the Civil Engineering Department of Bangladesh University of Engineering and Technology. Field and laboratory investigations including relative locations of the investigated areas, geologic formation of the research sites, standards and specifications for all the test performed are discussed elaborately in the following section.

3.2 Field Investigation:

Field investigations were carried out mainly through the application of cone penetration testing (CPT) and standard penetration testing (SPT). Thirty-eight (38) pairs of CPT and SPT were performed up to 30m depth or to the depth of maximum resistance whichever occurred first. The research area covered 11 districts (38 locations) out of which 4 districts (15 locations) represents riverine site and 6 districts (21 locations) covered the coastal region and rest 2 locations falls in both categories. Each pair of CPT and SPT has been carried out as close as possible, maximum horizontal distance is not greater than 1m.

3.2.1 Locations of the study

Figure 3.1 shows a map showing the relative locations of the study. Table 3.1 illustrates the detail information regarding the locations. As it can be observed from the map that the study for the riverine areas starts in the Kurigram district where Brahmaputra enters in Bangladesh from Dhubri, India. But the study locations of the Kurigram district mainly covers the bank of Tista river. Next study locations were in Gaibandha on the banks of Brahmaputra river. Following is Sirajganj on the banks of Jamuna river. Then, in Dhaka
where study has been carried out on the embankment soil of Turag river. After that, the study then advances towards the banks of Meghna river in Laxmipur where Meghna meets the Bay of Bengal. Study for the coastal region started with Satkhira and ended in Cox’s Bazar passing through Barguna, Bhola, Laxmipur, Noakhali and Chittagong.

**Figure 3.1** Study location
### Table 3.1 Locations of the research area

<table>
<thead>
<tr>
<th>Districts</th>
<th>Locations</th>
<th>No of Boring</th>
<th>Boring Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kurigram</td>
<td>Chilmari and Romna Khamar</td>
<td>3</td>
<td>30.0</td>
</tr>
<tr>
<td>Gaibandha</td>
<td>Horiup ghat and Bojari ghat</td>
<td>2</td>
<td>25.0-30.0</td>
</tr>
<tr>
<td>Sirajganj</td>
<td>Hard Point</td>
<td>4</td>
<td>30.0</td>
</tr>
<tr>
<td>Dhaka</td>
<td>Uttara 3rd Phase</td>
<td>6</td>
<td>14.0-28.0</td>
</tr>
<tr>
<td>Satkhira</td>
<td>Shamnagar &amp; Kaliganj</td>
<td>3</td>
<td>30.0</td>
</tr>
<tr>
<td>Barguna</td>
<td>Gab bari Natunbad, Puratanbad and Patharghata Chotobagi</td>
<td>6</td>
<td>30.0</td>
</tr>
<tr>
<td>Bhola</td>
<td>Char fasson</td>
<td>4</td>
<td>30.0</td>
</tr>
<tr>
<td>Laxmipur</td>
<td>Ramgoti</td>
<td>2</td>
<td>30.0</td>
</tr>
<tr>
<td>Chittagong</td>
<td>Anwara and Sitakund</td>
<td>4</td>
<td>22.0-27.0</td>
</tr>
<tr>
<td>Noakhali</td>
<td>Tankir Bazar</td>
<td>2</td>
<td>30.0</td>
</tr>
<tr>
<td>Cox’s Bazar</td>
<td>Maheshkhali</td>
<td>2</td>
<td>10.0</td>
</tr>
</tbody>
</table>

3.2.2 Geologic Formation of Research Site

Geologic formation of research sites have been analyzed according to several soil maps formulated by several researches in the past [(FAO/ UNDP, 1986), (Food and Agriculture Organisation, FAO, 1988), (Alam et al 1990), (Soil Resource Development Institute, SRDI, 1997), (Manwar et al 2001), (Islam et al 2017)]. Location wise surface geology and soil texture according to different researchers are presented in Table 3.2.

### Table 3.2 Geologic formation of research sites as per previous literature

<table>
<thead>
<tr>
<th>Locations</th>
<th>Surface Geology</th>
<th>Soil Texture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chilmari and Romna Khamar</td>
<td>Non Calcareous Gray Flood Plain Soil</td>
<td>Non Calcareous Alluvium</td>
</tr>
<tr>
<td>Horiup ghat and Bojari ghat</td>
<td>Non Calcareous Gray Flood Plain Soil</td>
<td>Gray Flood Plain Soil</td>
</tr>
<tr>
<td>Locations</td>
<td>Surface Geology</td>
<td>Soil Texture</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>-----------------------------------------------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td><strong>FAO 1988</strong></td>
<td><strong>SRDI 1997</strong></td>
<td><strong>Manwar 2001</strong></td>
</tr>
<tr>
<td>Hard Point</td>
<td>Non Calcareous Gray Flood Plain Soil</td>
<td>Alluvial Sand Deposit</td>
</tr>
<tr>
<td><em>SRDI 1997</em></td>
<td>Non Calcareous Alluvium</td>
<td></td>
</tr>
<tr>
<td><strong>Islam 2017</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uttara 3rd Phase</td>
<td>Non Calcareous and Calcareous Brown Flood Plain Soil</td>
<td>Mixed Gray, Dark gray and Brown Flood Plain Soil</td>
</tr>
<tr>
<td><em>SRDI 1997</em></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shamnagar &amp; Kaliganj</td>
<td>Non Calcareous Gray Flood Plain and Acid Sulphate Soil</td>
<td>Saline Tidal Flood Plain Soil</td>
</tr>
<tr>
<td>Gab bari Natonbad, Puratanbad and Patharghata Chotobagi</td>
<td>Non Calcareous Gray Flood Plain and Acid Sulphate Soil</td>
<td>Saline Tidal Flood Plain Soil</td>
</tr>
<tr>
<td>Char fasson</td>
<td>Calcareous Alluvium (Saline Soil)</td>
<td>Tidal Mud</td>
</tr>
<tr>
<td>Ramgoti</td>
<td>Calcareous Alluvium (Saline Soil)</td>
<td>Tidal Deltaic Deposits</td>
</tr>
<tr>
<td>Anwara and Sitakund</td>
<td>Calcareous Alluvium (Saline Soil)</td>
<td>Tidal Deltaic Deposits</td>
</tr>
<tr>
<td>Tankir Bazar</td>
<td>Non Calcareous and Calcareous Brown Flood Plain Soil</td>
<td>Gray Piedmont Soil</td>
</tr>
<tr>
<td>Maheshkhalī</td>
<td>Acid Sulphate Soil</td>
<td>Acid Basin Deposit</td>
</tr>
<tr>
<td><strong>SRDI 1997</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Islam 2017</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.2.3 Cone Penetration Test

CPT soundings were advanced using a Hogentogler type piezocone penetrometer with a cross sectional area of 10 cm$^2$ and which can measure the pore water pressure ($u_2$), as well as the cone tip resistance ($q_c$) and sleeve friction ($f_s$). To perform the tests, the cone was pushed vertically into the ground at a constant rate of approximately 2cm/sec. During the advancement, measurements of dynamic pore water pressure, tip resistance and sleeve friction were recorded continuously at 10 mm depth increments. The typical penetration depth for this study was approximately 30 m below from ground surface in most of the sites. CPTu dissipation test were done for each borehole. Cone penetration resistance has been corrected for pore pressure according to Eq. (2.29) referred in chapter two. Corrected cone resistance ($q_t$) varied from 0.14 to 35MPa shown in Figure 3.3.

![Figure 3.2 Carrying out CPT test on the bank of Tista River, Kurigram](image)
3.2.4 Standard Penetration Testing (SPT)

SPT were conducted as per ASTM D1586. Boreholes for the SPT were advanced by wash boring. The split spoon sampling method has been used to obtain soil samples from boreholes and disturbed representative samples were collected. Samples recovered from boreholes have been stored in plastic bags. Undisturbed samples have been collected using Shelby Tube through percussion drilling. Potential source of uncertainty which may affect SPT N-value has been carefully considered. Borehole drilling, soil sampling and SPT N-value recording procedures have been observed by experienced geologist during the entire test program and this individual provided visual descriptions of the collected samples. The SPT N-value and samples have been taken at every 1.5 m intervals. Rope and cathead SPT hammer-release has been used and the efficiency of hammer has been calculated as per Eq. (2.4). Necessary corrections have been done to obtain normalized SPT blow count as per Eq (2.6). It has been observed that SPT N-value has a range of 1
to 52, that is, the consistency varies from very soft to hard in case of cohesive soils or very loose to very dense in case of cohesion less soil as per article 2.3.4.1 and 2.3.4.2.

![Figure 3.4 Typical SPT profile (Kurigram BH-1)](image)

3.3 Laboratory Investigations

3.3.1 Tests and Guidelines

A laboratory test program was carried out in order to determine the engineering properties of the ground materials. Where applicable, laboratory tests have been performed according to American Society for Testing and Materials (ASTM) Standard as shown in Table 3.3.
<table>
<thead>
<tr>
<th>Test/Procedure Name</th>
<th>Description</th>
<th>Outcome</th>
<th>Code Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet Sieve Analysis</td>
<td>This is a procedure to estimate the percentage of sand and fines in a soil sample. This process is applied when the soil sample seems clayey or silty but liquid limit and plastic limit test cannot be performed.</td>
<td>Index properties of soil</td>
<td>ASTM D 1140</td>
</tr>
<tr>
<td>Grain Size (Sieve and Hydrometer) Analysis</td>
<td>This test method covers the quantitative determination of the distribution of particle sizes in soils. The distribution of particle sizes larger than 75 μm (retained on No. 200 sieve) is determined by sieving, while the distribution of particle sizes smaller than 75 μm is determined by a sedimentation process, using a hydrometer.</td>
<td>Index properties of soil</td>
<td>ASTM D 422</td>
</tr>
<tr>
<td>Atterberg Limit Test</td>
<td>This test method is used as an integral part of several engineering classification systems to characterize the fine-grained fraction of soil. The result of this test method is also used extensively with other soil properties to correlate with engineering behaviour.</td>
<td>Index properties of soil</td>
<td>ASTM D 4318</td>
</tr>
<tr>
<td>Determination of Moisture Content</td>
<td>This test method covers the laboratory determination of the moisture content by mass of soil with and without moisture.</td>
<td>Index properties of soil</td>
<td>ASTM D 2216</td>
</tr>
<tr>
<td>Determination of Density (Unit Weight)</td>
<td>This test method describes two ways of determining the moist and dry densities (unit weights) of intact, disturbed, remoulded, and reconstituted (compacted) soil specimens.</td>
<td>Index properties of soil</td>
<td>ASTM D 7263</td>
</tr>
<tr>
<td>Determination of Specific gravity</td>
<td>This test method covers the determination of the specific gravity of soil solid by means of a water pycnometer. Soil solid for this test method does not include solid which can be altered by this method, contaminated with a substance that prohibits the use of this method, or is highly organic such as fibrous matter which floats in water. Procedures for specimens such as organic</td>
<td>Index properties of soil</td>
<td>ASTM D 854</td>
</tr>
<tr>
<td>Test/Procedure Name</td>
<td>Description</td>
<td>Outcome</td>
<td>Code Specification</td>
</tr>
<tr>
<td>--------------------</td>
<td>-------------</td>
<td>---------</td>
<td>--------------------</td>
</tr>
<tr>
<td>Classification</td>
<td>USCS classification system has been used to correlate in a general way with the engineering behaviour of soil. ASTM D 2487 standard provides guidelines for USCS classification for engineering purposes.</td>
<td>Index properties of soil</td>
<td>ASTM D 2487</td>
</tr>
<tr>
<td>One Dimensional Consolidation Test</td>
<td>The data from consolidation test are used to estimate the magnitude and rate of both differential and total settlement of a structure or earth fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.</td>
<td>Settlement properties of cohesive soil</td>
<td>ASTM D 2435</td>
</tr>
<tr>
<td>Direct Shear Test</td>
<td>This test method covers the determination of the consolidated drained shear strength of a soil specimen in direct shear. The test is performed by deforming a specimen at a controlled strain rate on or near a single shear plane determined by the configuration of the apparatus. Generally, three or more specimens are tested, each under a different normal load, to determine the effects upon shear resistance and displacement, and strength properties such as angle of internal friction, cohesion etc.</td>
<td>Drained Shear Strength parameters c’ and φ’</td>
<td>ASTM D 3080</td>
</tr>
<tr>
<td>Consolidated Undrained Triaxial Test</td>
<td>This test method covers the determination of strength and stress-strain relationships of a cylindrical specimen of either an undisturbed or remolded saturated cohesive soil. Specimens are isotropically consolidated and sheared in compression without drainage at a constant rate of axial deformation (strain controlled). This test method provides for the calculation of total and effective stresses, and axial compression by measurement of axial load, axial deformation, and</td>
<td>Undrained Shear Strength parameters</td>
<td>ASTM D 4767</td>
</tr>
<tr>
<td>Test/Procedure Name</td>
<td>Description</td>
<td>Outcome</td>
<td>Code Specification</td>
</tr>
<tr>
<td>---------------------</td>
<td>-------------</td>
<td>---------</td>
<td>--------------------</td>
</tr>
<tr>
<td>pore-water pressure. This test method provides data useful in determining strength and deformation properties of cohesive soils such as Mohr strength envelopes and Young’s modulus. Generally, three specimens are tested at different effective consolidation stresses to define a strength envelope</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.3.2 Physical and Index Properties of Soil Samples

The detailed results of the laboratory tests are attached in the Appendix. Summary of different test results are presented below.

3.3.2.1 Moisture Content

The location wise range for natural moisture content obtained from the undisturbed samples collected from all the boreholes are shown in Table 3.4. The values of natural moisture content of undisturbed sample had a range of 8-37%.

**Table 3.4 Moisture content of different samples**

<table>
<thead>
<tr>
<th>Districts</th>
<th>Locations</th>
<th>Sample Depth (m)</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kurigram</td>
<td>Chilmari and Romna Khamar</td>
<td>2.10-3.60</td>
<td>17-30</td>
</tr>
<tr>
<td>Gaibandha</td>
<td>Horipur ghat and Bojari ghat</td>
<td>2.10-2.55</td>
<td>8-35</td>
</tr>
<tr>
<td>Sirajganj</td>
<td>Hard Point</td>
<td>2.10-2.55</td>
<td>19-22</td>
</tr>
<tr>
<td>Dhaka</td>
<td>Uttara 3rd Phase</td>
<td>4.00-6.00</td>
<td>20-35</td>
</tr>
<tr>
<td>Satkhira</td>
<td>Shannagar &amp; Kaliganj</td>
<td>2.10-7.05</td>
<td>30-35</td>
</tr>
<tr>
<td>Barguna</td>
<td>Gab bari Natunbad, Puratanbad and Patharghata Chotobagi</td>
<td>2.10-7.05</td>
<td>15-35</td>
</tr>
<tr>
<td>Bhola</td>
<td>Char fasson</td>
<td>2.10-7.05</td>
<td>17-30</td>
</tr>
<tr>
<td>Laxmipur</td>
<td>Ramgoti</td>
<td>2.10-3.60</td>
<td>15-30</td>
</tr>
</tbody>
</table>
### Districts

<table>
<thead>
<tr>
<th>Locations</th>
<th>Sample Depth (m)</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chittagong Anwara and Sitakund</td>
<td>2.10-5.55</td>
<td>22-30</td>
</tr>
<tr>
<td>Noakhali Tankir Bazar</td>
<td>2.10-5.55</td>
<td>14-37</td>
</tr>
<tr>
<td>Cox’s Bazar Maheshkhali</td>
<td>2.10-3.60</td>
<td>16-25</td>
</tr>
</tbody>
</table>

### 3.3.2.2 Unit Weight

The values of bulk and dry unit weight of the undisturbed samples obtained from all the boreholes are shown in Table 3.5. The values of bulk and dry unit weight of the undisturbed samples have been found to vary from 12.0-21.0 kN/m$^3$ and 10.0-18.0 kN/m$^3$ respectively.

**Table 3.5** Unit weight of different samples

<table>
<thead>
<tr>
<th>Locations</th>
<th>Sample Depth (m)</th>
<th>Bulk Unit Weight (kN/m$^3$)</th>
<th>Dry Unit Weight (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chilmari and Romma Khamar, Kurigram</td>
<td>2.10-3.60</td>
<td>14-20</td>
<td>12-16</td>
</tr>
<tr>
<td>Horipur Ghat and Bojari Ghat, Gaibandha</td>
<td>2.10-2.55</td>
<td>12-18</td>
<td>10-14</td>
</tr>
<tr>
<td>Hard Point, Sirajganj</td>
<td>2.10-2.55</td>
<td>19-20</td>
<td>16-17</td>
</tr>
<tr>
<td>Uttara 3$^{rd}$ Phase, Dhaka</td>
<td>4.00-6.00</td>
<td>16-19</td>
<td>12-16</td>
</tr>
<tr>
<td>Shamnagar &amp; Kaliganj, Satkhira</td>
<td>2.10-7.05</td>
<td>17-19</td>
<td>13-14</td>
</tr>
<tr>
<td>Gab Bari Natunbad, Puratanbad and Patharghata Chotobagi, Barguna</td>
<td>2.10-7.05</td>
<td>15-19</td>
<td>11-15</td>
</tr>
<tr>
<td>Char fasson, Bhola</td>
<td>2.10-7.05</td>
<td>14-19</td>
<td>10-15</td>
</tr>
<tr>
<td>Ramgoti, Laxmipur</td>
<td>2.10-3.60</td>
<td>14-20</td>
<td>11-15</td>
</tr>
<tr>
<td>Anwara and Sitakund, Chittagong</td>
<td>2.10-5.55</td>
<td>18-21</td>
<td>14-18</td>
</tr>
<tr>
<td>Tankir Bazar, Noakhali</td>
<td>2.10-5.55</td>
<td>14-20</td>
<td>10-17</td>
</tr>
<tr>
<td>Maheshkhali, Cox’s Bazar</td>
<td>2.10-3.60</td>
<td>14-16</td>
<td>11-14</td>
</tr>
</tbody>
</table>
3.3.2.3 Specific Gravity

Specific gravity values of the solid constituents ($G_s$) of different samples, several from each borehole were determined. Values of specific gravity of the solid constituents ($G_s$) of inorganic samples have been found to vary between 2.60-2.80.

3.3.2.4 Plasticity Characteristics

Liquid limit (LL), plastic limit (PL) and plasticity index (PI) of samples retrieved from cohesive samples have been determined. Plasticity characteristics of 17 inorganic samples were determined. The numerical results obtained from carrying out index property tests on selected samples are presented in Table 3.6. The flow curves for determination of liquid limit of these samples are presented in Appendix C. The values of liquid limit of the samples varied between 27 and 43 with plasticity index varying from 6 to 25.

Using the results of index property tests, soil samples exhibiting plasticity properties have been classified according to Unified Soil Classification System (USCS) as outlined in ASTM D2487 (ASTM, 2017). Classifications of these soil samples are also shown in Table 3.6. Positions of the inorganic samples on the Casagrande’s Plasticity Chart are shown in Figure 3.5. It can be seen from the figure that the positions of most of the samples tested lie above A-line and have liquid limits less than 50, indicating the samples being low plastic clays (CL). Three samples lie below A line which are silts (ML).

Table 3.6 Atterberg limits for different samples

<table>
<thead>
<tr>
<th>Location</th>
<th>BH No</th>
<th>Sample Depth (m)</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>USCS Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shyammagar, Satkhira</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>39</td>
<td>19</td>
<td>20</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>39</td>
<td>27</td>
<td>12</td>
<td>ML</td>
</tr>
<tr>
<td>Kaliganj, Satkhira</td>
<td>BH3</td>
<td>3.60-4.05</td>
<td>31</td>
<td>22</td>
<td>10</td>
<td>CL</td>
</tr>
<tr>
<td>Tankibazar, Noakhali</td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>33</td>
<td>20</td>
<td>13</td>
<td>CL</td>
</tr>
<tr>
<td>Charfasson, Bhola</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>37</td>
<td>30</td>
<td>7</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>36</td>
<td>23</td>
<td>13</td>
<td>CL</td>
</tr>
<tr>
<td>Location</td>
<td>BH No</td>
<td>Sample Depth (m)</td>
<td>LL</td>
<td>PL</td>
<td>PI</td>
<td>USCS Symbol</td>
</tr>
<tr>
<td>-----------------------</td>
<td>-------</td>
<td>------------------</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td>--------------</td>
</tr>
<tr>
<td>Pratonbad, Borguna</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>32</td>
<td>14</td>
<td>18</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>40</td>
<td>21</td>
<td>19</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>BH4</td>
<td>3.60-4.05</td>
<td>39</td>
<td>20</td>
<td>19</td>
<td>CL</td>
</tr>
<tr>
<td>Patharghata, Barguna</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>32</td>
<td>21</td>
<td>11</td>
<td>CL</td>
</tr>
<tr>
<td>Ramgoti, Laxmipur</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>33</td>
<td>21</td>
<td>12</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>37</td>
<td>26</td>
<td>11</td>
<td>ML</td>
</tr>
<tr>
<td>Anwara, Chittagong</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>43</td>
<td>18</td>
<td>25</td>
<td>CL</td>
</tr>
<tr>
<td>Maheshkhal, Cox’s Bazar</td>
<td></td>
<td>2.10-2.55</td>
<td>42</td>
<td>25</td>
<td>17</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>32</td>
<td>18</td>
<td>14</td>
<td>CL</td>
</tr>
</tbody>
</table>

Note:  
LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index  
CL = Lean Clay (Inorganic clay of low to medium plasticity)  
CH = Fat Clay (Inorganic clay of high plasticity)  
CL-ML = Silty Clay (Inorganic silty clay of low plasticity)  
USCS = Unified Soil Classification System
3.3.2.5 Particle Size Distribution

The grain size distribution analyses of 650 selected samples have been carried out. The grain size distribution curves of these samples are shown in Appendix B. From these particle size distribution curves, clay and silt size fraction (< 0.075 mm) and sand size fraction (> 0.075 mm and < 4.75 mm) of the samples have been determined using USCS Classification System. A summary of the grain size distribution of the samples on each borehole is shown in Table 3.7. Fines Content varied between 1-99%. A variation of Fineness Modulus has been observed within the range of 1.08-2.04 in granular soils.

Table 3.7 Summary of grain size distribution of selected samples

<table>
<thead>
<tr>
<th>Districts</th>
<th>% Sand (0.075-4.75mm)</th>
<th>% Fines (&lt; 0.075mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chilmari and Romna Khamar, Kurigram</td>
<td>1.4-98.9</td>
<td>1.1-98.6</td>
</tr>
<tr>
<td>Horipur ghat and Bojari ghat, Gaibandha</td>
<td>5.6-89.3</td>
<td>10.7-94.4</td>
</tr>
<tr>
<td>Hard Point, Sirajganj</td>
<td>24.1-99.1</td>
<td>0.9-75.9</td>
</tr>
<tr>
<td>Uttara 3rd Phase, Dhaka</td>
<td>5.6-97.1</td>
<td>2.9-96.4</td>
</tr>
<tr>
<td>Shamnagar &amp; Kaliganj, Satkhira</td>
<td>1-99.1</td>
<td>0.9-99.0</td>
</tr>
<tr>
<td>Gab bari Natunbad, Puratanbad and Patharghata</td>
<td>29.5-94.9</td>
<td>5.1-69.5</td>
</tr>
<tr>
<td>Chotobagi, Barguna</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Char fasson, Bhola</td>
<td>74-97.2</td>
<td>2.8-26.0</td>
</tr>
<tr>
<td>Ramgoti, Laxmipur</td>
<td>2-36</td>
<td>64.0-98.0</td>
</tr>
<tr>
<td>Anwara and Sitakund, Chittagong</td>
<td>2-74.8</td>
<td>25.2-98.0</td>
</tr>
<tr>
<td>Tankir Bazar, Noakhali</td>
<td>2.6-69.1</td>
<td>30.9-97.4</td>
</tr>
<tr>
<td>Maheshkhali, Cox’s Bazar</td>
<td>15.2-89.5</td>
<td>10.5-84.8</td>
</tr>
</tbody>
</table>

3.3.2.6 Consolidated Drained Direct Shear Test

Consolidated drained direct shear tests were carried out on 25 undisturbed samples. Cylindrical specimens of 63.5 mm diameter by 25 mm high were initially consolidated using three different normal loads and subsequently sheared under drained condition. The shear stress versus shear deformation plots and failure envelope of the sample tested are
presented in Appendix. From the failure envelopes of the samples, the values of effective cohesion ($c'$) and effective angle of internal friction ($\phi'$) of the samples have been determined. A summary of the direct shear test results is presented in Table 3.8. The values of $c'$ and $\phi'$ have been found to vary in between 0-10 kN/m$^2$ and 23-36°, respectively. It appears from the test results that the sample tested was normally consolidated. Detailed results are provided in Appendix D.

**Table 3.8 Summary of consolidated drained direct shear test results**

<table>
<thead>
<tr>
<th>Location</th>
<th>BH No</th>
<th>Depth</th>
<th>Effective Angle of Internal Friction ($\phi'$)</th>
<th>Effective Cohesion ($c'$) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chilmari and Romna Khamar, Kurigram</td>
<td>BH1</td>
<td>2.10-2.54</td>
<td>28.2</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>28.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Horipur Ghat and Bojarighat, Gaibandha</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>24.5</td>
<td>9.3</td>
</tr>
<tr>
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<td>BH2</td>
<td>2.10-2.56</td>
<td>31.3</td>
<td>3.6</td>
</tr>
<tr>
<td>Hard point, Sirajganj</td>
<td>BH4</td>
<td>3.60-4.05</td>
<td>38.1</td>
<td>0.0</td>
</tr>
<tr>
<td>Shyam Nagar, Shatkhira</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>23.9</td>
<td>8.1</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>23.6</td>
<td>6.0</td>
</tr>
<tr>
<td>Tankir Bazar, Noakhali</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>33.7</td>
<td>2.4</td>
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<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>29.6</td>
<td>0.0</td>
</tr>
<tr>
<td>Char fasson, Bhola</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>31.2</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>30.2</td>
<td>1.6</td>
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<tr>
<td></td>
<td>BH3</td>
<td>3.60-4.05</td>
<td>31.2</td>
<td>7.3</td>
</tr>
<tr>
<td>Puratonbad and Natunbad, Borguna</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>28.0</td>
<td>2.5</td>
</tr>
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<td>BH2</td>
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<td></td>
<td>BH3</td>
<td>3.60-4.05</td>
<td>20.8</td>
<td>9.1</td>
</tr>
<tr>
<td></td>
<td>BH4</td>
<td>3.60-4.05</td>
<td>27.8</td>
<td>4.0</td>
</tr>
<tr>
<td>Patharghata, Barguna</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>36.0</td>
<td>4.5</td>
</tr>
<tr>
<td>Ramgoti, Laxmipur</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>32.9</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>27.6</td>
<td>5.1</td>
</tr>
<tr>
<td>Anwara, Chittagong</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>28.5</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>29.5</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>BH1</td>
<td>2.10-2.56</td>
<td>28.9</td>
<td>0.6</td>
</tr>
</tbody>
</table>
### 3.3.2.7 One Dimensional Consolidation Test

Compressibility and expansibility property of 17 samples have been determined from incremental loading in one-dimensional consolidation test according to ASTM D-2435. Consolidation test has been carried out on sample of 61.1 mm diameter and 17.8 mm thickness using stress increment ratio of 1, (i.e. load increment ratio of 2). The vertical stresses applied during consolidation have been 25 kPa, 50 kPa, 100 kPa, 200 kPa, 400 kPa, 800 kPa. The samples have been also allowed to swell under the same series of stresses. Duration of each loading step has been twenty-four hours. During the test, drainage has been permitted from top and bottom of the sample. For each loading step dial gauge has been used to record deformations at specified intervals of time.

Time-deformation curves have been plotted for each pressure increment and from these plots’ times corresponding to 50% and 90% consolidation, i.e., $t_{50}$ and $t_{90}$ have been determined according to ASTM D-2435 guideline. Coefficient of consolidation for vertical flow, $c_v$ has been calculated for stress increment of 50 kPa, 100 kPa, 200 kPa, 400 kPa and 800 kPa using $t_{50}$ and $t_{90}$.

Void ratio versus log of effective vertical pressure plots, deformation versus log of time plots and deformation versus square root of time plots for the samples are presented in Appendix. Compression index ($C_c$) and swelling index ($C_s$) of the samples have been determined from the slopes of the loading and unloading portion, respectively, of the void ratio versus log of effective vertical pressure curves shown in Appendix E. A summary of one-dimensional consolidation test result is presented in Table 3.9. The value of $C_c$ has been found to vary in between 0.120 to 0.259 while the value of $C_s$ of the sample has been found to vary within the range of 0.003 to 0.040. Initial void ratio ($e_0$) of the samples have varied from 0.645 to 0.945. Depending on the stress range, the values of coefficient of consolidation for vertical flow ($c_v$) of the sample have varied between 1.2-25.8 m$^2$/year from $t_{50}$ and 4.4-30.5 m$^2$/year from $t_{90}$.

<table>
<thead>
<tr>
<th>Location</th>
<th>BH No</th>
<th>Depth</th>
<th>Effective Angle of Internal Friction ($\phi'$)</th>
<th>Effective Cohesion ($c'$) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maheshkhali, Cox’s Bazar</td>
<td>BH2</td>
<td>2.10-2.57</td>
<td>29.7</td>
<td>2.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>BH No</th>
<th>Depth</th>
<th>Effective Angle of Internal Friction ($\phi'$)</th>
<th>Effective Cohesion ($c'$) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maheshkhali, Cox’s Bazar</td>
<td>BH2</td>
<td>2.10-2.57</td>
<td>29.7</td>
<td>2.6</td>
</tr>
</tbody>
</table>
Table 3.9 Summary of one-dimensional consolidation test result

<table>
<thead>
<tr>
<th>Location</th>
<th>Borehole</th>
<th>Depth</th>
<th>Initial Void Ratio, $e_o$</th>
<th>Compression Index, $C_c$</th>
<th>Swell Index, $C_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shyam Nagar and Kaliganj Satkhira</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>0.945</td>
<td>0.248</td>
<td>0.017</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>0.849</td>
<td>0.200</td>
<td>0.009</td>
</tr>
<tr>
<td></td>
<td>BH3</td>
<td>3.60-4.05</td>
<td>0.645</td>
<td>0.157</td>
<td>0.029</td>
</tr>
<tr>
<td>Tankibazar, Noakhali</td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>0.664</td>
<td>0.170</td>
<td>0.004</td>
</tr>
<tr>
<td>Charfasson, Bhola</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>0.692</td>
<td>0.145</td>
<td>0.031</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>0.891</td>
<td>0.199</td>
<td>0.044</td>
</tr>
<tr>
<td></td>
<td>BH3</td>
<td>3.60-4.05</td>
<td>0.891</td>
<td>0.206</td>
<td>0.003</td>
</tr>
<tr>
<td>Natunbad and Puratonbad, Borguna</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>0.748</td>
<td>0.182</td>
<td>0.029</td>
</tr>
<tr>
<td></td>
<td>BH4</td>
<td>3.60-4.05</td>
<td>0.845</td>
<td>0.217</td>
<td>0.017</td>
</tr>
<tr>
<td>Patharghata, Barguna</td>
<td>BH1</td>
<td>3.60-4.05</td>
<td>0.793</td>
<td>0.193</td>
<td>0.025</td>
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<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.05</td>
<td>0.869</td>
<td>0.202</td>
<td>0.016</td>
</tr>
<tr>
<td>Ramgoti, Laxmipur</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>0.869</td>
<td>0.186</td>
<td>0.016</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>0.805</td>
<td>0.188</td>
<td>0.032</td>
</tr>
<tr>
<td>Anwara, Chittagong</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>0.920</td>
<td>0.259</td>
<td>0.016</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>0.658</td>
<td>0.120</td>
<td>0.015</td>
</tr>
<tr>
<td>Maheshkhali, Cox’s Bazar</td>
<td>BH1</td>
<td>2.10-2.55</td>
<td>0.923</td>
<td>0.239</td>
<td>0.015</td>
</tr>
<tr>
<td></td>
<td>BH2</td>
<td>2.10-2.55</td>
<td>0.739</td>
<td>0.188</td>
<td>0.014</td>
</tr>
</tbody>
</table>

3.3.2.8 Consolidated Undrained Triaxial Compression Test

Consolidated undrained triaxial compression tests have been carried out on 24 undisturbed samples. Mohr circles have been drawn in terms of effective stresses from which the effective shear strength parameters, e.g., undrained cohesion ($c_u$) and angle of internal friction ($\phi_u$) of the sample tested have been determined. Deviator stress versus axial strain curve, volumetric strain versus axial strain curve and Mohr circles in terms of effective stresses for the consolidated undrained triaxial compression test are presented in Appendix F. Summary of the consolidated undrained triaxial compression test results
is shown in Table 10. The values of $c_u$ and $\phi_u$ has been found to vary in between 10-78 kN/m$^2$ and 23-39$^\circ$, respectively. It appears from the test results that the sample tested was over consolidated.

**Table 3.10** Summary of Consolidated Undrained Triaxial Test

<table>
<thead>
<tr>
<th>Location</th>
<th>Borehole</th>
<th>Depth (m)</th>
<th>$\phi_u$</th>
<th>Undrained Cohesion $c_u$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chilmari, Kurigram</td>
<td>BH2</td>
<td>3.60-4.06</td>
<td>35.0</td>
<td>50</td>
</tr>
<tr>
<td>Horipur ghat and Bojari ghat, Gaibandha</td>
<td>BH1, BH2</td>
<td>3.60-4.07, 3.60-4.08</td>
<td>32.2, 38.5</td>
<td>52, 75</td>
</tr>
<tr>
<td>Hard point, Sirajganj</td>
<td>BH4</td>
<td>5.10-5.59</td>
<td>39.0</td>
<td>38</td>
</tr>
<tr>
<td>Shyam Nagar and Kaliganj Shatkhira</td>
<td>BH1, BH2</td>
<td>5.10-5.59, 5.10-5.59</td>
<td>24.3, 25.4</td>
<td>10, 32</td>
</tr>
<tr>
<td>BH3</td>
<td>3.60-4.05</td>
<td>22.9</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>Tankibazar, Noakhali</td>
<td>BH1, BH2</td>
<td>5.10-5.59, 5.10-5.59</td>
<td>39.0, 28.2</td>
<td>40, 46</td>
</tr>
<tr>
<td>Char fasson, Bhola</td>
<td>BH1, BH2, BH3</td>
<td>5.10-5.59, 5.10-5.59, 5.10-5.59</td>
<td>29.7, 32.2, 33.7</td>
<td>50, 50, 60</td>
</tr>
<tr>
<td>Natunbad and Puratonbad, Borguna</td>
<td>BH1, BH2, BH3, BH4</td>
<td>5.10-5.59, 5.10-5.59, 5.10-5.59, 5.10-5.59</td>
<td>23.3, 26.9, 24.9, 24.3</td>
<td>36, 29, 40, 26</td>
</tr>
<tr>
<td>Patharghata, Barguna</td>
<td>BH1, BH2</td>
<td>5.10-5.59, 5.10-5.59</td>
<td>22.9, 33.7</td>
<td>30, 46</td>
</tr>
<tr>
<td>BH1</td>
<td>3.60-4.05</td>
<td>35.6</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Borehole</td>
<td>Depth (m)</td>
<td>Angle of Internal Friction ($\phi$)°</td>
<td>Undrained Cohesion $c_u$ (kPa)</td>
</tr>
<tr>
<td>---------------------------</td>
<td>----------</td>
<td>-----------</td>
<td>-------------------------------------</td>
<td>-------------------------------</td>
</tr>
<tr>
<td>Ramgoti, Laxmipur</td>
<td>BH2</td>
<td>3.60-4.06</td>
<td>25.9</td>
<td>45</td>
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<td>Anwara, Chittagong</td>
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<td>3.60-4.07</td>
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<td>70</td>
</tr>
<tr>
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<td>BH2</td>
<td>3.60-4.08</td>
<td>28.3</td>
<td>10</td>
</tr>
<tr>
<td>Maheshkhali, Cox’s Bazar</td>
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<td>23.4</td>
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</tr>
<tr>
<td></td>
<td>BH2</td>
<td>3.60-4.10</td>
<td>31.9</td>
<td>78</td>
</tr>
</tbody>
</table>

### 3.4 Summary

Lab test results are as follows:

- The values of natural moisture content of undisturbed sample have a range of 8-37%.
- The values of bulk and dry unit weight of the undisturbed samples have been found to vary from 12.0-21.0 kN/m³ and 10.0-18.0 kN/m³ respectively.
- Values of specific gravity of the solid constituents ($G_s$) of inorganic samples have been found to vary between 2.60-2.80.
- The values of liquid limit of the samples have varied between 27 and 43 with plasticity index varying from 6 to 25. Fines Content varied between 1-99%. A variation of Fineness Modulus has been observed within the range of 1.08-2.04 in granular soils.
- The values of $c'$ and $\phi'$ have been found to vary in between 0-10 kN/m² and 23-36° as per consolidated drained direct shear test
- The values of $c_u$ and $\phi_u$ have varied from 10-78 kN/m² and 23-39°, respectively for consolidated undrained triaxial test.
- The value of $C_c$ has been found to vary in between 0.120 to 0.259 while the value of $C_s$ of the sample was found to vary within the range of 0.003 to 0.040. Initial void ratio ($e_0$) of the samples varied from 0.645 to 0.945. Depending on the stress range, the values of coefficient of consolidation for vertical flow ($c_v$) of the sample have varied between 1.2-25.8 m²/year for $t_{50}$ and 4.4-30.5 m²/year for $t_{90}$. 
4.1 General

This chapter has been organized in four sections. The first section illustrates interpretation of CPTu data for soil classification and preparation of soil stratigraphy. Comparison of soil stratigraphy from CPT and USCS classification system has been shown in here. Following section describes several direct and indirect approaches for CPT-SPT correlation. The next section describes the interpretation of strength, settlement and plasticity index parameters from CPT and SPT data. Characterization of riverine and coastal soil has been illustrated in the final section.

4.2 CPT based Soil Classification

CPT based classification method proposed by Robertson (Robertson, 1990), described in detail in chapter 2 has been followed here. Firstly, friction ratios (Fr) and soil behavior indexes (Ic) have been determined using Eq. (2.31), then CPT based classification have been done with the help of different Ic ranges for different soil type as illustrated in Table 2.9. Each soil type has been color coded for better understanding. Soil stratigraphy prepared for all the boreholes have been attached in the Appendix.

An example for the preparation of CPT based soil stratigraphy is described below for bore hole 1 of Anwara, Chittagong.

Step 1:

Determination of normalized and non-normalized parameters like Qt, Fr, Rf, qf/pa, ISBTn and ISBT as per Eq (2.31), (2.32), (2.34) and (2.36) mentioned in chapter two.

It can be observed from the depth wise profiles that normalized parameters have slightly higher values than non-normalized parameters. Where non normalized parameters can be obtained directly from in-situ tests, normalized parameters requires determination of total
and effective overburden stress that requires unit weight of soil. CPT based unit weight have been determined with the following equation provided by Robertson and Cabal (2010),

\[
\gamma / \gamma_w = 0.27 \log(R_f) + 0.36 \log(q_t / p_a) + 1.236
\]  

Figure 4.1 Normalized and non-normalized friction ratio and SBT index

Where,

\(\gamma\) and \(\gamma_w\) is the unit weight of soil and water respectively, \(R_f\) is the friction ratio \(\left(\frac{f_s}{q_c}\right)\), \(q_t\) is the corrected cone resistance.

**Step 2 :**

Plotting SBT (Robertson, 2010) and SBTn (Robertson, 1990) charts for sorting soil properties of different depths into the dedicated zones for different soil types for classification.

It is observed from Figure 4.2 that soil type varies from zone 3 to 6 which is clays, silt mixtures, sand mixtures and sand.

With normalized parameters the results are a little different where some data have fallen under zone 8 and 9 which represent very stiff sand to clayey sand and also very stiff fine
grained soils. There are also some data falling under the category of organic soil within zone 2.

**Figure 4.2** Chart for soil behavior type with non-normalized parameters

SBT charts shows the variation of soil type for all the soils that has been encountered during soil investigation but do not show the depth wise variation of soil types that is soil stratigraphy.

**Step 3:**
Using limiting values for soil behavior index (ISBT or Ic) of the concentric circles to prepare soil stratigraphy. All five different ranges have been color coded with dense sand to gravelly sand being orange, sand being gray, sand mixtures being yellow, silt mixtures being blue and clays being green.

**Figure 4.3** Chart for soil behavior type with normalized parameters

It is observed from Figure 4.4 that soils are mostly clay mixtures to clay up to 6m depth, the next 3m are mostly silt mixtures, then comes a layer of clays to the depth of 14m, next 5m are silt mixtures to clay mixtures, then a layer of clay mixtures is encountered up to the depth of 22.5m, following comes the final layer of sand mixtures to silt mixtures.
Step 4:

For easy and clear understanding of soil layers an average value of soil behavior indexes have been taken on every 1.5m interval as shown in Figure 4.5.

**Figure 4.4** Soil stratigraphy through soil behavior index (I_{SBT})

There are 9 primary soil layers with alternative silty clay to clay, clayey silt to silty and silty sand to sandy silt. From the previous literature the soil was expected to be silty loam and the results agrees with the statement.
Step 5:

Final soil stratigraphy with detailed soil strata is presented on Figure 4.6.

**Figure 4.5** Simplified soil stratigraphy from average $I_c$ values
<table>
<thead>
<tr>
<th>Depth of Borehole (m)</th>
<th>Layer Thickness (m)</th>
<th>Description (Soil Strata Encountered)</th>
<th>CPT $q_c$ Profile</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>3.0</td>
<td>Clayey Silt to Silty Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td>4.5</td>
<td>Silty Clay to Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>4.5</td>
<td>Silty Sand to Sandy Silt</td>
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<td></td>
</tr>
<tr>
<td>6.0</td>
<td>3.0</td>
<td>Silty Clay to Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td>3.0</td>
<td>Silty Sand to Sandy Silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.0</td>
<td>6.0</td>
<td>Silty Clay to Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.5</td>
<td>3.0</td>
<td>Silty Sand to Sandy Silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12.0</td>
<td>3.0</td>
<td>Clayey Silt to Silty Clay</td>
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</tr>
<tr>
<td>13.5</td>
<td>1.5</td>
<td>Silty Clay to Clay</td>
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<td></td>
</tr>
<tr>
<td>15.0</td>
<td>1.5</td>
<td>Silty Sand to Sandy Silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16.5</td>
<td>3.0</td>
<td>Clayey Silt to Silty Clay</td>
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</tr>
<tr>
<td>18.0</td>
<td>3.0</td>
<td>Silty Clay to Clay</td>
<td></td>
<td></td>
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<tr>
<td>19.5</td>
<td>1.5</td>
<td>Clayey Silt to Silty Clay</td>
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</tr>
<tr>
<td>21.0</td>
<td>1.5</td>
<td>Silty Clay to Clay</td>
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<td></td>
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<tr>
<td>22.5</td>
<td>1.5</td>
<td>Clayey Silt to Silty Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>24.0</td>
<td>1.5</td>
<td>Silty Sand to Sandy Silt</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 4.6** CPT based soil profile (Anwara BH-1)
4.2.1 Soil Profile CPT Vs SPT

Wash sieve, sieve analysis and Atterberg limit tests have been done on 20 disturbed sample obtained from standard penetration testing done at Anwara BH1 at every 1.5m interval. Classification based on soil tests is presented below in the summary Table 4.1.

**Table 4.1 Index properties of soil samples of Anwara, BH-1**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Sample Depth (m)</th>
<th>Sand (%)</th>
<th>Fine (%)</th>
<th>Atterberg Limits</th>
<th>USCS Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>LL</td>
<td>PL</td>
</tr>
<tr>
<td>1</td>
<td>1.5</td>
<td>2.7</td>
<td>97.3</td>
<td>34</td>
<td>17</td>
</tr>
<tr>
<td>2</td>
<td>3.0</td>
<td>7.1</td>
<td>92.9</td>
<td>35</td>
<td>21</td>
</tr>
<tr>
<td>3</td>
<td>4.5</td>
<td>2.4</td>
<td>97.6</td>
<td>38</td>
<td>24</td>
</tr>
<tr>
<td>4</td>
<td>6.0</td>
<td>17.8</td>
<td>82.2</td>
<td>44</td>
<td>34</td>
</tr>
<tr>
<td>5</td>
<td>7.5</td>
<td>38.6</td>
<td>61.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>9.0</td>
<td>50.8</td>
<td>49.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>10.5</td>
<td>11.7</td>
<td>88.3</td>
<td>39</td>
<td>24</td>
</tr>
<tr>
<td>8</td>
<td>12.0</td>
<td>19.3</td>
<td>80.7</td>
<td>42</td>
<td>26</td>
</tr>
<tr>
<td>9</td>
<td>13.5</td>
<td>21.2</td>
<td>78.8</td>
<td>41</td>
<td>22</td>
</tr>
<tr>
<td>10</td>
<td>15.0</td>
<td>22.6</td>
<td>77.4</td>
<td>42</td>
<td>25</td>
</tr>
<tr>
<td>11</td>
<td>16.5</td>
<td>30.6</td>
<td>69.4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>18.0</td>
<td>59.9</td>
<td>40.1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>19.5</td>
<td>18</td>
<td>82</td>
<td>40</td>
<td>23</td>
</tr>
<tr>
<td>14</td>
<td>21.0</td>
<td>15.8</td>
<td>84.2</td>
<td>38</td>
<td>24</td>
</tr>
<tr>
<td>15</td>
<td>22.5</td>
<td>16.8</td>
<td>83.2</td>
<td>43</td>
<td>22</td>
</tr>
<tr>
<td>16</td>
<td>24.0</td>
<td>59.8</td>
<td>40.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>17</td>
<td>25.5</td>
<td>48</td>
<td>52</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>18</td>
<td>27.0</td>
<td>41</td>
<td>59</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>19</td>
<td>28.5</td>
<td>35</td>
<td>65</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>30.0</td>
<td>38</td>
<td>62</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Soil profile has been prepared with the information obtained from laboratory investigations with the disturbed samples mentioned in the above summery table. The SPT-based soil profile of Anwara, BH1 is shown in Figure 4.7.
Figure 4.7 SPT based soil profile (Anwara BH-1)
To compare SPT and CPT based soil classification soil profiles obtained from both methods are shown side by side. It is observed from Figure 4.8 that soil profiles shows significant similarity.

<table>
<thead>
<tr>
<th>SPT BASED (USCS)</th>
<th>CPT BASED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lean CLAY</td>
<td>Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>SILT with Sand</td>
<td>Silty Clay to Clay</td>
</tr>
<tr>
<td>Sandy SILT</td>
<td>Silty Sand to Sandy Silt</td>
</tr>
<tr>
<td>Silty SAND</td>
<td>Silty Sand to Sandy Silt</td>
</tr>
<tr>
<td>Lean CLAY</td>
<td>Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>CLAY with Sand</td>
<td>Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>Sandy SILT</td>
<td>Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>Silty SAND</td>
<td>Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>CLAY with Sand</td>
<td>Silty Clay to Clay</td>
</tr>
<tr>
<td>Silty SAND</td>
<td>Clayey Silt to Silty Clay</td>
</tr>
<tr>
<td>Sandy SILT</td>
<td>Silty Sand to Sandy Silt</td>
</tr>
</tbody>
</table>

**Figure 4.8** CPT vs SPT soil profile (Anwara BH1)
Though CPT based soil classification is less specific than unified soil classification system, it is instant and less laborious.

4.3 CPT-SPT Correlation

4.3.1 $q_t/N$ Ratio for Different Soil Types

Correlation between cone tip resistance ($q_t$) and standard penetration blow count ($N_{60}$) has found to be linear as per the previous literatures discussed in chapter 2. The ratio changes as per variation in soil types. For this study, initially a generalized $q_t/N$ ratio has been determined for all 38 pairs of CPT-SPT with 650 data points illustrated in Figure 4.9 and Figure 4.10.

![Corrected Cone Tip Resistance ($q_t$) Vs Corrected SPT Blow Count ($N_{1,60}$)](image)

**Figure 4.9** Standard deviation of all the data points from the linear trend line

It is observed from the figure that, $N_{1,60}$ and $q_t$ has a fairly good linear correlation with a correlation factor of $R=0.59$. The $q_t/N_{1,60}$ was found to be 0.3 with a coefficient of determination $R^2=0.35$.

After that, $q_t/N$ ratio have been determined for following 2 soil types.

1) Sand to silty sand (sand mixture)
2) Silt,silty clay to clay (silt mixture)
Figure 4.10 A generalized CPT-SPT correlation for all soil types

Figure 4.11 CPT-SPT correlation for sand to silty Sand
It can be observed from the above figures that, qt/N ratio for sand mixtures is found to be 0.31 whereas it is 0.21 for silt mixtures. That is, the ratio decreases as the grain size gets finer. The result is quite similar with the observation found from Schmertzmann (1970).

CPT-SPT ratio have also been determined for different soil region as mentioned in Table 3.2 in the previous Chapter. Regions have been classified as per FAO (1988) and SRDI (1997).

It can be observed from Figure 4.13, Figure 4.14, Figure 4.15 and Figure 4.16 that qt/N varies from 0.10–0.45 for various soil types for studied regions which has been summarized in Table 4.2.

Figure 4.12 CPT-SPT correlation for silty clay to clay
Figure 4.13 CPT-SPT correlation for calcareous alluvium

Figure 4.14 CPT-SPT correlation for non calcareous flood plain soil
Summary of the $q_t/N$ ratio for different soil types are presented in Table 4.2.
### Table 4.2 Findings for $q_t/N$ ratio for different soil types

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Correlation</th>
<th>$q_t/N$</th>
<th>n</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>All types</td>
<td>$q_t = 0.2972N_{1,60}$</td>
<td>0.297</td>
<td>650</td>
<td>0.35</td>
</tr>
<tr>
<td>Sand Mixtures</td>
<td>$q_t = 0.3133N_{1,60}$</td>
<td>0.313</td>
<td>503</td>
<td>0.32</td>
</tr>
<tr>
<td>Silt Mixture</td>
<td>$q_t = 0.2093N_{1,60}$</td>
<td>0.209</td>
<td>150</td>
<td>0.39</td>
</tr>
<tr>
<td>Calcereous Alluvium</td>
<td>$q_t = 0.1685N_{1,60} + 1.0721$</td>
<td>0.168</td>
<td>115</td>
<td>0.32</td>
</tr>
<tr>
<td>Non Calcareous Flood Plain Soil</td>
<td>$q_t = 0.2204N_{1,60} + 2.5894$</td>
<td>0.220</td>
<td>215</td>
<td>0.28</td>
</tr>
<tr>
<td>Saline Tidal Flood Plain</td>
<td>$q_t = 0.4453N_{1,60}$</td>
<td>0.445</td>
<td>194</td>
<td>0.36</td>
</tr>
<tr>
<td>Mixed Flood Plain Soil</td>
<td>$q_t = 0.1126N_{1,60} + 0.7872$</td>
<td>0.113</td>
<td>61</td>
<td>0.21</td>
</tr>
</tbody>
</table>

#### 4.3.2 $q_{t}/N$ Ratio based on Soil Behavior Index

Available literature shows both linear and logarithmic correlation between CPT and SPT based on soil behavior index. For this study soil behavior index, $I_c$ has been determined using Eq. (2.36) for all the boreholes. All 650 data points was plotted against $(q_{t}/p_a)/N_{60}$ and compared with previous literatures. It can be observed from Figure 4.17 data points fall close to the range for available literature and shows somewhat a linear pattern. $q_{t}/N$ ratio decreases with increase in behavior index value. As we know behavior index increases with decreasing grain size, it implies $q_{t}/N$ ratio decreases with decreasing grain size.

To come up with a correlation for the studied data points initially different trend line was formulated and the value of coefficient of determination, $R^2$ was observed for each trend line. It was found that exponential trend line had the maximum $R^2$ value.

To increase the confidence level of the correlation, data were filtered within the range of mean ± standard deviation while formulation of the correlation. It is clearly visible that $R^2$ value increased from 0.37 to 0.40 due to the filter.
Figure 4.17 Comparison of study points with previous behavior index based CPT-SPT correlation

Figure 4.18 Comparison of different trend lines with $R^2$ values
4.3.3 $q_t/N$ Ratio based on Mean Grain Size

Available literature shows a wide range of correlation between CPT and SPT based on mean grain size. Total of 530 data points have been plotted to formulate a correlation. Though the data agree with the pattern available for the previous correlations that is $q_t/N$ ratio increases with increasing grain size, they are very scattered to formulate a reliable correlation. Soil for the studied region is mostly fine sand with mean grain size 0.1-0.2. If equal amount of data could be availed from all mean grain size ranges from .001 to 1, the correlation could have been better understood. Besides the available literature shows that for the same mean grain size $q_t/N$ has an upper and a lower boundary. So an upper and lower boundary for the plotted data set have been determined with a probabilistic approach where upper boundary was defined by “mean+sigma” and lower boundary was defined by “mean –sigma”. But the lower boundary results in negative $D_{50}$ values, so the lower boundary can not be showed on the logarithmic graph in Figure 4.21. Trend line for all the data points with the maximum $R^2$ value has been found to be $(qc/pa)/N_{60} = 7.2\ D_{50}^{0.07}$ and is the proposed correlation for studied soil type as per Figure 4.22.

---

**Figure 4.19** Proposed behavior index based CPT-SPT correlation

<table>
<thead>
<tr>
<th>Study point</th>
<th>Filtered</th>
<th>Proposed Correlation</th>
<th>Mean $+\sigma$</th>
<th>Mean $-\sigma$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
(qc/pa)/N_{60} = 17.8e^{-0.82\ I_c} \\
R^2 = 0.4064 \\
n=525
\]
Figure 4.20 Comparison of study points with previous mean grain size based CPT-SPT correlation

Figure 4.21 Upper boundary for mean grain size based \((q_c/p_a)/N_{60}\)
4.3.4 $q_u/N$ Ratio based on Fines Content

A total of 554 data points of fines content have been plotted against $(q_u/p_a)/N_{60}$ and some data are found to be very scattered. Previous correlations have been plotted on the same graph for better understanding. It is observed that most data were in proximity with the previous correlations as shown in Figure 4.23. Data have been filtered within the range of standard deviation with a probabilistic approach as shown in Figure 4.24. 66 data points have been excluded in the filtering process. It can be observed that $q_u/N$ ratio decreases with increasing fines content. Linear trend line has slightly low $R^2$ value than exponential trend line but the linear correlation $(q_u/p_a)/N_{60}= 3.3785-0.0128 f_c$ is quite similar with the correlation $(q_u/p_a)/N_{60}= 4.25-0.0242 f_c$ provided by Kulhaway & Mayne (1990).

Figure 4.22 Proposed mean grain size based CPT-SPT correlation
The summary of the obtained results from the discussion of article 4.3.2, 4.3.3 and 4.3.4 are presented on Table 4.3.
Table 4.3 Findings for $q_t/N$ ratio based on index properties of soil

<table>
<thead>
<tr>
<th>Base for $q_t/N$ ratio</th>
<th>Correlation</th>
<th>n</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Behavior Index, $I_c$</td>
<td>$(q_c/p_a)/N_{60} = 17.8e^{-0.82 I_c}$</td>
<td>525</td>
<td>0.41</td>
</tr>
<tr>
<td>Mean Grain Size, $D_{50}$</td>
<td>$(q_c/p_a)/N_{60} = 7.2 D_{50}^{0.07}$ (Upper Boundary)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>$(q_c/p_a)/N_{60} = 3.7967D_{50}^{0.1753}$</td>
<td>530</td>
<td>0.04</td>
</tr>
<tr>
<td>Fines Content, $f_c$</td>
<td>$(q_c/p_a)/N_{60} = 3.2509e^{-0.007 f_c}$</td>
<td>488</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>$(q_c/p_a)/N_{60} = 3.3785-0.0128 f_c$</td>
<td>488</td>
<td>0.102</td>
</tr>
</tbody>
</table>

All the correlations provide lower $q_t/N$ ratio for finer particles. So it can be concluded that the difference between CPT and SPT value is smaller as grain size gets finer.

4.4 Interpretation of Strength Parameters from SPT

Data obtained from direct shear and triaxial test both are used to establish correlation to interpret angle of internal friction angle from SPT. The results obtained from triaxial test in consolidated undrained condition are only used to establish SPT correlation with undrained shear strength.

4.4.1 $N_{60}-\phi$ Correlation:

Friction angles have been plotted against SPT blow count $N_{60}$. Plotted data points are in well proximity of available correlations. All the available correlations give higher friction angles for higher SPT values. The plotted data pattern is similar to the previous correlations. The data set is very small to obtain a reliable relationship but it clearly indicates the applicability of available correlations to the local soil. Correlations provided by Hatanaka and Uchida (Hatanaka & Uchida, 1996) and Wolf (Wolff, 1989) slightly over predicts the friction angle for lower SPT values.
Figure 4.25 Comparison of study points with previous N₆₀-ϕ correlations

\[ \phi' = -0.1309N_{60}^2 + 3.1234N_{60} + 19.467 \]
\[ R^2 = 0.4416 \]
\[ n=25 \]

Figure 4.26 Proposed N₆₀-ϕ' correlation for direct shear test
Figure 4.27 Proposed $N_{60}$-$\phi_u$ correlation for triaxial test

Observations of article 4.4.1 is summarized in Table 4.4.

Table 4.4 Observations for friction angle from SPT value

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Correlation</th>
<th>n</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direct Shear</td>
<td>$\phi_d^o = -0.1309N_{60}^2 + 3.1234N_{60} + 19.467$</td>
<td>25</td>
<td>0.44</td>
</tr>
<tr>
<td>Triaxial</td>
<td>$\phi_u^o = -0.0393N_{60}^2 + 2.1062N_{60} + 20.304$</td>
<td>24</td>
<td>0.54</td>
</tr>
</tbody>
</table>

4.4.2 $N_{60}$-$c_u$ Correlation:

Undrained cohesion ($c_u$) obtained from triaxial tests were plotted against SPT blow count. It was observed that $N_{60}$-$c_u$ correlation provided by Hara et al (1971) over predicts the undrained cohesion but correlation provided by Kulhawy and Mayne (1990) predicts the cohesion very close to the values obtained from laboratory tests. The coefficient of determination $R^2$ is very high for proposed correlation, though a small data set is not an appropriate measure to formulate a correlation. Basically the study proves the
applicability of the correlation provided by Kulhawy and Mayne to estimate undrained cohesion from SPT N value.

Observation from Figure 4.28 is summarized in the following table.

Table 4.5 Observations for undrained cohesion from SPT value

<table>
<thead>
<tr>
<th>Correlation</th>
<th>n</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>$c_u = 7.0511N_{60} + 4.6518$</td>
<td>23</td>
<td>0.886</td>
</tr>
</tbody>
</table>

Figure 4.28 Comparison of study points with previous $N_{60}$-$c_u$ correlations

Figure 4.29 Proposed $N_{60}$-$c_u$ correlation for triaxial test
4.5 Interpretation of Strength Parameters from CPT

Previous literature shows logarithmic relationship between friction angle and cone tip resistance and linear correlation between undrained shear strength and cone tip resistance. Laboratory test results were plotted along with previous correlation to observe the behavior of studied soils.

4.5.1 \( q_t-\phi \) Correlation:

Angle of internal friction obtained from the laboratory tests are mostly higher than friction angles obtained by \( q_t-\phi \) correlation provided by Robertson (1983) and Kulhawy (1990) but very close to Minmura (2003). The data points also seem to formulate a logarithmic pattern. Limited data set could be effective for the judgement of the applicability of previous correlations but a reliable new correlation cannot be formulated with it. The value of the coefficient of determination is 40% for triaxial test and 20% for direct shear test. Such \( R^2 \) could be significant for a large data set but is very low for a limited data set.

It can be observed that friction angle slightly increases with increasing cone tip resistance. Results obtained from triaxial test form better correlation than direct shear test.

![Friction Angle Vs Cone Tip Resistance](image)

**Figure 4.30** Comparison of study points with previous \( q_t-\phi \) correlations
Observation from Figure 4.31 and Figure 4.32 is summarized in the following table.

**Table 4.6 Observations for friction angle from CPT value**

<table>
<thead>
<tr>
<th>Test</th>
<th>Correlation</th>
<th>n</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direct Shear Test</td>
<td>$\phi' = 2.4778\ln(q_t) + 28.68$</td>
<td>22</td>
<td>0.186</td>
</tr>
<tr>
<td>Triaxial Test</td>
<td>$\phi_u = 3.5129\ln(q_t) + 28.06$</td>
<td>23</td>
<td>0.399</td>
</tr>
</tbody>
</table>

**Figure 4.31** Proposed $q_t$-$\phi'$ correlation for the results obtained from direct shear test

**Figure 4.32** Proposed $q_t$-$\phi_u$ correlation for the results obtained from triaxial test
4.5.2 \( q_t-c_u \) Correlation:

Undrained cohesion obtained from triaxial test were plotted against cone tip resistance to compare with previously established correlation provided by Robertson (2012). It was observed that the correlation significantly over estimates undrained cohesion. The logarithmic trend of the plotted data resulted in the highest \( \text{R}^2 \) value of 0.4.

**Figure 4.33** Comparison of study points with previous \( q_t-c_u \) correlation

\[
s_u = 18.34 \ln(q_t) + 40.411 \\
\text{R}^2 = 0.3938
\]

**Figure 4.34** Proposed \( q_t-c_u \) correlation for the results obtained from triaxial test
Observation from Figure 4.34 is summarized in the following table.

<table>
<thead>
<tr>
<th>Correlation</th>
<th>n</th>
<th>( \text{R}^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( s_u = 18.34 \ln(q_t) + 40.411 )</td>
<td>24</td>
<td>0.394</td>
</tr>
</tbody>
</table>

### 4.6 Interpretation of Compression Index, \( C_c \) from CPT and SPT

Laboratory obtained compressibility index have been plotted against corrected SPT blow count (\( N_{1,60} \)), Cone tip resistance (\( q_t \)) and \( (q_t/p_a)/N_{1,60} \) as illustrated in Figure 4.35, Figure 4.36 and Figure 4.37. The curves show that higher SPT values give higher \( C_c \) value whereas in case of CPT or CPT-SPT ratio an increase causes decrease in the compressibility index which is quite contrary to the linear proportional CPT-SPT correlation. Though the correlation between \( C_c \) and SPT appears to be linearly proportional like existing correlation with index properties, the \( \text{R}^2 \) value is much less than \( C_c \) vs CPT or \( (q_t/p_a)/N_{1,60} \) curves.

**Figure 4.35** Proposed correlation between compression index (\( C_c \)) and standard penetration resistance (\( N_{60} \))
Figure 4.36 Proposed correlation between compression index \((C_c)\) and cone tip resistance \((q_t)\)

\[
C_c = -0.0333q_t + 0.2336 \\
R^2 = 0.4142
\]

\[
C_c = 0.24e^{-0.19qt} \\
R^2 = 0.4458
\]

Figure 4.37 Proposed correlation between compression index \((C_c)\) and \((q_t/p_a)/N_{1,60}\)

\[
C_c = 0.2231e^{-0.086(qt/pq)/N1,60} \\
R^2 = 0.6453
\]

\[
C_c = -0.0145(q_t/p_a)/N_{1,60} + 0.2198 \\
R^2 = 0.5575
\]
The observations have been summarized in Table 4.8. In the cases of CPT and CPT-SPT ratio-based correlations, exponential curves give slightly higher determinant values than that of the linear curves.

**Table 4.8** Observations for compressibility index from CPT-SPT

<table>
<thead>
<tr>
<th>Correlation with</th>
<th>Proposed Equation</th>
<th>Coefficient of determinant, R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPT blow count N₁,₆₀</td>
<td>(C_c=0.01(0.63N₁,₆₀+14.56))</td>
<td>0.27</td>
</tr>
<tr>
<td>Cone tip resistance (q_c)</td>
<td>(C_c=0.24e^{-0.19q_c})</td>
<td>0.45</td>
</tr>
<tr>
<td>Cone tip resistance (q_c)</td>
<td>(C_c=0.2336-0.0333q_c)</td>
<td>0.41</td>
</tr>
<tr>
<td>CPT-SPT ratio ((q_c/p_a)/N₁,₆₀)</td>
<td>(C_c=0.2231e^{-0.086(q_c/p_a)/N₁,₆₀})</td>
<td>0.65</td>
</tr>
<tr>
<td>CPT-SPT ratio ((q_c/p_a)/N₁,₆₀)</td>
<td>(C_c=0.2198-0.0145(q_c/p_a)/N₁,₆₀)</td>
<td>0.56</td>
</tr>
</tbody>
</table>

It is clearly visible from the study that the new correlations \(R² > 60\%\) provide useful guidance for preliminary assessments of the compressibility of Bangladeshi soils.

### 4.7 Interpretation of Plasticity Characteristics from CPT and SPT

Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) has been plotted against standard penetration resistance, \(N₁,₆₀\) and cone penetration resistance \(q_c\) to understand the effect of penetration resistance on plasticity characteristics of soil. As plasticity characteristics of soil have dependency on soil type and penetration resistance is also influenced by soil type, it was reasonable to assume that both parameters could be inter-related.

It was observed from Plastic limit vs SPT graph that plastic limit seems to linearly decrease with increasing SPT blow count while plastic limit increases with increasing cone penetration resistance. This is quite contrary to the CPT-SPT correlation.

Again, liquid limits seem to increases with increasing cone penetration resistance unlike plasticity index which seem to decrease with increasing cone tip resistance.

But it can be observed from Figure 4.38, Figure 4.39, Figure 4.40 and Figure 4.41 that the coefficient of determination of the linear trend line is too low to formulate any correlation between these parameters.
The observations have been summarized in Table 4.9.

**Table 4.9** Observations for plasticity properties from CPT-SPT

<table>
<thead>
<tr>
<th>Proposed Equation</th>
<th>Coefficient of determinant, $R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$PL = -0.5275N_{1,60} + 27.465$</td>
<td>0.12</td>
</tr>
<tr>
<td>$LL = 2.7156q_t + 32.733$</td>
<td>0.18</td>
</tr>
<tr>
<td>$PL = 4.0412q_t + 18.376$</td>
<td>0.36</td>
</tr>
<tr>
<td>$PI = -1.6835q_t + 15.523$</td>
<td>0.10</td>
</tr>
</tbody>
</table>

**Figure 4.38** Correlation between plastic limit and standard penetration resistance
Figure 4.39 Correlation between liquid limit and cone penetration resistance

\[ \text{LL} = 2.7156q_t + 32.733 \]
\[ R^2 = 0.1883 \]

Figure 4.40 Correlation between plastic limit and cone penetration resistance

\[ \text{PL} = 4.0412q_t + 18.376 \]
\[ R^2 = 0.3601 \]
Figure 4.41 Correlation between plastic limit and cone penetration resistance

4.8 Summary

CPT based soil classification has been compared with SPT based classification and CPT based classification is faster, continuous and agreed with the conventional classification in most cases. $q_t/N_{60}$ ratio for different soil types have been determined with $R^2$ value varying from 0.1 to 0.35. It has also been observed that $q_t/N$ ratio decreases with increase in behavior index value and fines content whereas $q_t/N$ ratio increases with increasing mean grain size. Strength parameters increases in value with increasing SPT blow count and cone tip resistance. Compression index decreases with increasing cone tip resistance and $q_t/N$ ratio. Plastic limits seem to increase with cone tip resistance.
5.1 Conclusions

This chapter summarizes the conclusion of the research results and major findings. The major objective of the research is the characterization of soil from in-situ test parameters. For this purpose, soil classification has been done using CPT test data directly; correlations have been proposed for index parameters, strength parameter, compressibility characteristics and plasticity properties. Moreover, CPT-SPT correlations for different soil types have been determined so that performing any of these two tests can serve for soil characterization.

The main constraint is the limited number of laboratory tests. Strength parameters and compressibility properties have been determined with undisturbed samples for data accuracy but there have been a limited number of undisturbed samples for a very small range of depth. Modern technology and recent development of sampling allow for collection of undisturbed sandy samples, but for this research no undisturbed sandy sample has been collected. Another limitation is the use of manual machines rather than modern automated machines.

The proposed correlations are as follows:

- Proposed CPT-SPT correlations are shown below:

<table>
<thead>
<tr>
<th>Correlation</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_t = 0.2972N_{1,60}$</td>
<td>All types</td>
</tr>
<tr>
<td>$q_t = 0.3133N_{1,60}$</td>
<td>Sand Mixtures</td>
</tr>
<tr>
<td>$q_t = 0.2093N_{1,60}$</td>
<td>Silt Mixture</td>
</tr>
<tr>
<td>$q_t = 0.1685N_{1,60} + 1.0721$</td>
<td>Calcareous Alluvium</td>
</tr>
<tr>
<td>$q_t = 0.2204N_{1,60} + 2.5894$</td>
<td>Non Calcareous Flood Plain Soil</td>
</tr>
<tr>
<td>$q_t = 0.4453N_{1,60}$</td>
<td>Saline Tidal Flood Plain</td>
</tr>
<tr>
<td>$q_t = 0.1126N_{1,60} + 0.7872$</td>
<td>Mixed Flood Plain Soil</td>
</tr>
</tbody>
</table>
- Proposed correlations to determine soil behavior index ($I_c$), mean grain size ($d_{50}$) and fines content ($f_c$) are provided below:

<table>
<thead>
<tr>
<th>Index Properties</th>
<th>Correlation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Behavior Index, $I_c$</td>
<td>$(q_c/p_a)/N_{60} = 17.8e^{-0.82I_c}$</td>
</tr>
<tr>
<td>Mean Grain Size, $d_{50}$</td>
<td>$(q_c/p_a)/N_{60} = 7.2 d_{50}^{0.07}$ (Upper Boundary)</td>
</tr>
<tr>
<td></td>
<td>$(q_c/p_a)/N_{60} = 3.7967d_{50}^{0.1753}$</td>
</tr>
<tr>
<td>Fines Content, $f_c$</td>
<td>$(q_c/p_a)/N_{60} = 3.2509e^{-0.007f_c}$</td>
</tr>
<tr>
<td></td>
<td>$(q_c/p_a)/N_{60} = 3.3785-0.0128 f_c$</td>
</tr>
</tbody>
</table>

- Correlation of both SPT blow count ($N_{60}$) and CPT cone tip resistance ($q_t$) to estimate undrained shear strength ($s_u$) and angle of internal friction ($\phi^o$) are given below:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Correlation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angle of Internal friction, $\phi^o$</td>
<td>$\phi_d^o = 2.4778\ln(q_t) + 28.7$</td>
</tr>
<tr>
<td></td>
<td>$\phi_u^o = 3.5129\ln(q_t) + 28.0$</td>
</tr>
<tr>
<td></td>
<td>$\phi_d^o = -0.1309N_{60}^2 + 3.1234N_{60} + 19.5$</td>
</tr>
<tr>
<td></td>
<td>$\phi_u^o = -0.0393N_{60}^2 + 2.1062N_{60} + 20.3$</td>
</tr>
<tr>
<td>Undrained Shear Strength, $s_u$</td>
<td>$s_u = 7.0511N_{60} + 4.7$</td>
</tr>
<tr>
<td></td>
<td>$s_u = 18.34\ln(q_t) + 40.4$</td>
</tr>
</tbody>
</table>

- In this research, potential correlations between the compression index and field investigation results have been investigated for the first time for Bangladeshi soils. It is clearly visible from the study that the new correlations ($R^2 > 60\%$) provide useful guidance for preliminary assessments of the compressibility of Bangladeshi soils. Proposed correlations are provided below:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Correlation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression Index ($C_c$)</td>
<td>$C_c=0.01(0.63N_{1,60}+14.6)$</td>
</tr>
<tr>
<td></td>
<td>$C_c=0.24e^{-0.19q_t}$</td>
</tr>
<tr>
<td></td>
<td>$C_c=0.2336-0.0333 q_t$</td>
</tr>
<tr>
<td></td>
<td>$C_c=0.2231e^{-0.066(q_t/p_a)^{N_{1,60}}}$</td>
</tr>
<tr>
<td></td>
<td>$C_c=0.2198-0.0145 (q_t/p_a)^{N_{1,60}}$</td>
</tr>
</tbody>
</table>
• Very little influence of plasticity properties with SPT has been observed but CPT seems to influence plastic limit which can be interpreted as PL = 4.0412q_t + 18.4.

A major objective of this research is geotechnical characterization based on in-situ data. The objective has been fulfilled through the following observations.

a. Bore log for soil can be prepared using Cone Penetration Test
b. Index properties of soil can be estimated using CPT-SPT correlation
c. A preliminary assessment of strength parameters, settlement characteristics and plasticity properties can be done with the help of in-situ test data with provided correlations

5.2 Recommendation for Future Study

This research is aimed to facilitate the current practices to use in-situ test data in the best way possible for geotechnical designs. Correlations for important parameters like shear strength and friction angles have been established based on very limited data set which leaves a scope for future study. In addition, physical properties like unit weight, relative density, state parameter, hydraulic conductivity etc. can be interpreted from in-situ test results. Important seismic parameters like shear wave velocity can also be interpreted from CPT. Recently, Bangladesh government has taken several major steps to prevent seismic vulnerability; so, CPT interpretation for seismic design parameters is very relevant and important in current circumstances. This leaves enormous scope for future investigation and research work in the field of in-situ test interpretation. So, the recommendations may be summarized as follows.

a. Study may be carried out to interpret strength and compressibility parameters with a large data set for reliable correlation.
b. Study may be carried out to establish reliable correlation for other physical properties like unit weight, hydraulic conductivity etc. for local soil
c. Study may be carried out for interpretation of seismic design parameters from CPT data
REFERENCES


FAO/ UNDP. (1986). *Classification of the soils of Bangladesh*.


Soil Resource Development Institute, SRDI. (1997). *General Soil Map Bangladesh*. GIS unit, SRDI.


APPENDIX A
FIELD INVESTIGATION ANALYSES
CPT Test Results: Anwara BH-1

- Cone Tip Resistance $q_c$ (Mpa)
- Sleeve Friction $f_s$
- Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Anwara BH-1

- Corrected Cone Tip Resistance ($q_t$)
- Friction Ratio ($R_f$)
- Normalized Friction Ratio ($R'_f$)
SPT Test Results: Anwara BH-2

Depth (m)

0.0 25.0 50.0

0 25 50

N60

0.0 1.5 3.0 4.5 6.0 7.5 9.0 10.5 12.0 13.5 15.0 16.5 18.0 19.5 21.0 22.5 24.0 25.5 27.0 28.5 30.0

Depth (m)

0.0 1.5 3.0 4.5 6.0 7.5 9.0 10.5 12.0 13.5 15.0 16.5 18.0 19.5 21.0 22.5 24.0 25.5 27.0 28.5 30.0

N1-60

0 25 50

6.8 2.9 1.2 5.1 1.8 0.8 1.6 0.7 1.4 1.9 1.2 1.8 5.1 6.6 8.5
CPT Results Analyses Part I: Anwara BH-2

Corrected Cone Tip Resistance ($q_t$)

Friction Ratio ($R_f$)

Normalized Friction Ratio ($R_f$)
CPT Results Analyses Part II: Anwara BH-2

SBT Chart

Cone Resistance, qc/\(\text{pa}\)
Friction Ratio, \(R_f\)

SBTn Chart

Cone Resistance, Qt
Friction Ratio, \(F_r\)
SPT Test Results: Puratanbad Barguna BH-1

Depth (m)
CPT Test Results: Natunbad Barguna BH-2

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Natunbad Barguna BH-2

Corrected Cone Tip Resistance ($q_t$)

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Results Analyses Part III: Natunbad Barguna BH-2

Depth vs. Ic SBT/STn

- Ic
- Gravelly sand to sand
- Clean sand to silty sand
- Silty sand to Sandy Silt
- Clayey Silt to Silty Clay
- Silty Clay to Clay
CPT Test Results: Natunbad Barguna BH-3

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
SPT Test Results: Puratanbad Barguna BH-4

Depth (m) vs. N60

Depth (m) vs. N1-60
CPT Test Results: Puratanbad Barguna BH-4

Cone Tip Resistance $q_c$ (MPa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (MPa)
CPT Results Analyses Part III: Puratanbad Barguna BH-4

Ic SBT

Ic SBTn

Depth

- Ic
- Gravelly sand to sand
- Clean sand to silty sand
- Silty sand to Sandy Silt
- Clayey Silt to Silty Clay
- Silty Clay to Clay
CPT Test Results: Charfasson Bhola BH-1

- Cone Tip Resistance $q_c$ (Mpa)
- Sleeve Friction $f_s$
- Pore Pressure $u_2$ (Mpa)

Depth (m): 0.0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15.0, 16.5, 18.0, 19.5, 21.0, 22.5, 24.0, 25.5, 27.0, 28.5, 30.0

Depth (m): 0.0, 1.5, 3.0, 4.5, 6.0, 7.5, 9.0, 10.5, 12.0, 13.5, 15.0, 16.5, 18.0, 19.5, 21.0, 22.5, 24.0, 25.5, 27.0, 28.5, 30.0

Depth (m): 0.0, 0.1, 0.2, 0.3, 0.4, 0.5, -0.05, 0.45, 0.95
CPT Results Analyses Part I: Charfasson Bhola BH-1

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Results Analyses Part I: Maheshkhali Cox's Bazar BH-2

Corrected Cone Tip Resistance, \( q_t \)

Friction Ratio, \( R_f \)

Normalized Friction Ratio, \( R_f \)
SPT Test Results : Charfasson Bhola BH-3

Depth (m) vs. N60 and N1.60 values.
CPT Test Results: Charfasson Bhola BH-3

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Charfasson Bhola BH-3

- Corrected Cone Tip Resistance ($q_t$)
- Friction Ratio ($R_f$)
- Normalized Friction Ratio ($R''_f$)
CPT Results Analyses Part II: Charfasson Bhola BH-3

SBT Chart

SBTn Chart
CPT Results Analyses Part III: Charfasson Bhola BH-3

Depth

Ic SBT

Ic SBTN

- Ic
- Gravelly sand to sand
- Clean sand to silty sand
- Silty sand to Sandy Silt
- Clayey Silt to Silty Clay
- Silty Clay to Clay
CPT Test Results: Haripur Ghat Gaibandha BH-1

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Haripur Ghat Gaibandha BH-1

Corrected Cone Tip Resistance, $q_t$

Depth (m)

Friction Ratio, $R_f$

Depth (m)

Normalized Friction Ratio, $R_f$

Depth (m)
CPT Results Analyses Part II: Haripur Ghat Gaibandha BH-1

SBT Chart

SBTn Chart
SPT Test Results: Bojari Ghat Gaibandha BH-2

Depth (m) vs. N<sub>60</sub> and N<sub>1-60</sub>

- Depth (m): 0.0 to 30.0
- N<sub>60</sub>: 0.0 to 50.0
- N<sub>1-60</sub>: 0.0 to 50.0

- Depth markers: 0.0, 1.5, 3.0, ..., 30.0
- N<sub>60</sub> values: 0.0, 25.0, 50.0
- N<sub>1-60</sub> values: 0.0, 25.0, 50.0

- Depth range from 0.0 to 30.0 meters
- N<sub>60</sub> values range from 0.0 to 50.0
- N<sub>1-60</sub> values range from 0.0 to 50.0

Note: The diagram shows the distribution of SPT test results at different depths.
SBT Chart

Cone Resistance, $q_c/p_a$

Friction Ratio, $R_f$

SBTn Chart

Cone Resistance, $q_t$

Friction Ratio, $F_r$
CPT Test Results: Chilmari Kurigram BH-1

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction, $f_s$

Pore Pressure, $u_2$ (Mpa)
CPT Results Analyses Part I: Chilmari Kurigram BH-1

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
SPT Test Results: Romna Khamar Kurigram BH-2
CPT Results Analyses Part II: Romna Khamar Kurigram BH-2

**SBT Chart**

- Cone Resistance, $q_c/\rho_a$
- Friction Ratio, $R_f$

**SBTn Chart**

- Cone Resistance, $Q_t$
- Friction Ratio, $F_r$
CPT Results Analyses Part III: Romna Khamar Kurigram BH-2

Ic SBT

Gravelly sand to sand
Clean sand to silty sand
Silty sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay

Ic SBTn

Gravelly sand to sand
Clean sand to silty sand
Silty sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay
CPT Test Results: Bengmara Ghat Kurigram BH-3

- Cone Tip Resistance, \( q_c \) (Mpa)
- Sleeve Friction, \( f_s \)
- Pore Pressure, \( u_2 \) (Mpa)
CPT Results Analyses Part I: Bengmara Ghat Kurigram BH-3

- Corrected Cone Tip Resistance $q_t$
- Friction Ratio, $R_f$
- Normalized Friction Ratio, $R_{nf}$
CPT Test Results: Maheshkhali Cox's Bazar BH-1

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Maheshkhali Cox's Bazar BH-1

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Results Analyses Part I: Maheshkhali Cox's Bazar BH-2

Corrected Cone Tip Resistance $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Results Analyses Part II: Maheshkhali Cox's Bazar BH-2

[SBT Chart]

[SBTn Chart]
Ic SBT

- Gravelly sand to sand
- Clean sand to silty sand
- Silty sand to Sandy Silt
- Clayey Silt to Silty Clay
- Silty Clay to Clay
SPT Test Results: Patharghata Barguna BH-1

Depth (m) vs. N60

Depth (m) vs. N1.60
CPT Test Results: Patharghata Barguna BH-1

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Patharghata Barguna BH-1

Corrected Cone Tip Resistance, \( q_t \)

Friction Ratio, \( R_f \)

Normalized Friction Ratio, \( R_f \)
CPT Test Results: Patharghata Barguna BH-2

- Cone Tip Resistance $q_c$ (Mpa)
- Sleeve Friction $f_s$
- Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Patharghata Barguna BH-2

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Test Results: Ramgoti Lakshmipur BH-1

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I : Ramgoti Lakshmipur BH-1

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Results Analyses Part II: Ramgoti Lakshmipur BH-1

SBT Chart

SBTn Chart
CPT Test Results: Ramgoti Lakshmipur BH-2

- Cone Tip Resistance $q_c$ (Mpa)
- Sleeve Friction $f_s$
- Pore Pressure $u_2$ (Mpa)

Graphs showing variations with depth for Cone Tip Resistance, Sleeve Friction, and Pore Pressure.
CPT Results Analyses Part I: Ramgoti Lakshmipur BH-2

Corrected Cone Tip Resistance, \(q_t\)

Friction Ratio, \(R_f\)

Normalized Friction Ratio, \(R_f\)
CPT Results Analyses Part I: Shaymnagar Satkhira BH-1

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Results Analyses Part III: Shaymnagar Satkhira BH-1

Graphs showing depth (depth) and electrical conductivity (Ic, Ic SBT, Ic SBTn) with color-coded layers indicating different soil types:
- Gravelly sand to sand
- Clean sand to silty sand
- Silty sand to Sandy Silt
- Clayey Silt to Silty Clay
- Silty Clay to Clay
CPT Results Analyses Part II: Shaymnagar Satkhira BH-2

SBT Chart

- Cone Resistance, $q_c/p_a$
- Friction Ratio, $R_f$

SBTn Chart

- Cone Resistance, $Q_t$
- Friction Ratio, $F_r$
SPT Test Results: Kaliganj Satkhira BH-3

Depth (m) 0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0

N<sub>60</sub> 25.0 29.0 33.0 37.0 41.0 45.0 49.0 53.0 57.0 61.0 65.0

N<sub>1.60</sub> 25.0 29.0 33.0 37.0 41.0 45.0 49.0 53.0 57.0 61.0 65.0
CPT Test Results: Kaliganj Satkhira BH-3

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Kaliganj Satkhira BH-3

Corrected Cone Tip Resistance $q_c$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R'_f$
CPT Results Analyses Part III: Kaliganj Satkhira BH-3

Ic SBT

Ic SBTn

Depth

Ic
Gravelly sand to sand
Clean sand to silty sand
Silty sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay

Depth

Ic
Gravelly sand to sand
Clean sand to silty sand
Silty sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay
SPT Test Results : Hard Point Sirajganj BH-1

Depth (m)
SPT Test Results: Hard Point Sirajganj BH-2

- **N<sub>60</sub>**
  - Depth (m): 0.0, 25.0, 50.0
  - Values: 8, 7, 7, 13, 27, 28, 25, 25, 26, 28, 32, 32, 34, 38, 40, 37, 37, 41, 44

- **N<sub>1.60</sub>**
  - Depth (m): 0.0, 25.0, 50.0
CPT Test Results: Hard Point Sirajganj BH-2

Cone Tip Resistance $q_c$ (Mpa)

Sleeve Friction $f_s$

Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part II: Hard Point Sirajganj BH-2

SBT Chart

SBTn Chart
CPT Results Analyses Part III: Hard Point Sirajganj BH-2

Ic SBT

Ic SBTn

---

Depth

1 2 3 4

1.00 2.00 3.00 4.00 5.00

---

Gravelly sand to sand
Clean sand to silty sand
Silty sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay

---

Gravelly sand to sand
Clean sand to silty sand
Silty sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay
SPT Test Results: Hard Point Sirajganj BH-3
CPT Results Analyses Part I: Hard Point Sirajganj BH-3

Corrected Cone Tip Resistance, $q_t$:

- Depth (m):
  - 0.0
  - 1.5
  - 3.0
  - 4.5
  - 6.0
  - 7.5
  - 9.0
  - 10.5
  - 12.0
  - 13.5
  - 15.0
  - 16.5
  - 18.0
  - 19.5
  - 21.0
  - 22.5
  - 24.0
  - 25.5
  - 27.0
  - 28.5
  - 30.0

Friction Ratio, $R_f$:

- Depth (m):
  - 0.0
  - 1.5
  - 3.0
  - 4.5
  - 6.0
  - 7.5
  - 9.0
  - 10.5
  - 12.0
  - 13.5
  - 15.0
  - 16.5
  - 18.0
  - 19.5
  - 21.0
  - 22.5
  - 24.0
  - 25.5
  - 27.0
  - 28.5
  - 30.0

Normalized Friction Ratio, $R_{nf}$:

- Depth (m):
  - 0.0
  - 1.5
  - 3.0
  - 4.5
  - 6.0
  - 7.5
  - 9.0
  - 10.5
  - 12.0
  - 13.5
  - 15.0
  - 16.5
  - 18.0
  - 19.5
  - 21.0
  - 22.5
  - 24.0
  - 25.5
  - 27.0
  - 28.5
  - 30.0
CPT Results Analyses Part II: Hard Point Sirajganj BH-3

**SBT Chart**

**SBTn Chart**
CPT Results Analyses Part III : Hard Point Sirajganj BH-3

![Graph showing CPT results with depth and soil types: gravelly sand to sand, clean sand to silty sand, silty sand to sandy silt, clayey silt to silty clay, silty clay to clay.](image_url)
SPT Test Results: Hard Point Sirajganj BH-4

Depth (m)

\(N_{60}\)

\(N_{1.60}\)

0.0 25.0 50.0

0.0 25.0 50.0
CPT Test Results: Hard Point Sirajganj BH-4

- Cone Tip Resistance $q_c$ (Mpa)
- Sleeve Friction $f_s$
- Pore Pressure $u_2$ (Mpa)

Graphs showing the variation of $q_c$, $f_s$, and $u_2$ with depth.
CPT Results Analyses Part II: Hard Point Sirajganj BH-4

SBT Chart

Friction Ratio, $R_f$

Friction Ratio, $R_t$

SBTn Chart

Friction Ratio, $F_r$

Cone Resistance, $q_e/p_a$

Cone Resistance, $q_t$
CPT Results Analyses Part III : Sitakunda BH-1

Depth

Ic SBT

Gravelly sand to sand
Clean sand to silty sand
Silty sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay
CPT Results Analyses Part I : Sitakunda BH-2

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_f$
CPT Results Analyses Part II: Sitakunda BH-2

SBT Chart

Friction Ratio, $R_f$

Cone Resistance, $q_c/p_a$

SBTn Chart

Friction Ratio, $F_r$

Cone Resistance, $q_t$
CPT Test Results: Tankir Bazar Noakhali BH-1

- Cone Tip Resistance ($q_c$) (MPa)
- Sleeve Friction ($f_s$)
- Pore Pressure ($u_2$) (MPa)

Graphs showing variations with depth (m) for cone tip resistance, sleeve friction, and pore pressure.
SPT Test Results: Tankir Bazar Noakhali BH-2
CPT Test Results: Tankir Bazar Noakhali BH-2

- Cone Tip Resistance $q_c$ (Mpa)
- Sleeve Friction $f_s$
- Pore Pressure $u_2$ (Mpa)
CPT Results Analyses Part I: Tankir Bazar Noakhali BH-2

Corrected Cone Tip Resistance, $q_t$

Friction Ratio, $R_f$

Normalized Friction Ratio, $R_n$
APPENDIX B

GRAIN SIZE DISTRIBUTION ANALYSIS RESULTS
Gaibandha BH-1

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D3 (4.5m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D4 (6.0m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D5 (7.5m)
Gaibandha BH-1

Grain Size Distribution Curve

D6 (9.0m)

Grain Size Distribution Curve

D7 (10.5m)

Grain Size Distribution Curve

D8 (12.0m)
Gaibandha BH-1

Grain Size Distribution Curve

D15 (22.5m)

Grain Size Distribution Curve

D20 (30.0m)
Gaibandha BH-2

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)
Gaibandha BH-2

Grain Size Distribution Curve

D15 (22.5m)

Particle Size (mm)

% Finer

Grain Size Distribution Curve

D20 (30.0m)

Particle Size (mm)

% Finer
Chilmari, Kurigram BH-1

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D1 (1.5m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D2 (3.0m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D3 (4.5m)
Chilmari, Kurigram BH-1

Grain Size Distribution Curve

D6 (9.0m)

Grain Size Distribution Curve

D10 (15.0m)

Grain Size Distribution Curve

D11 (16.5m)
Chilmari, Kurigram BH-1

Grain Size Distribution Curve

- D13 (19.5m)

Grain Size Distribution Curve

- D17 (25.5m)

Grain Size Distribution Curve

- D19 (28.5m)
Chilmari, Kurigram BH-2

Grain Size Distribution Curve

D1 (1.5m)

Grain Size Distribution Curve

D2 (3.0m)

Grain Size Distribution Curve

D4 (6.0m)
Chilmari, Kurigram BH-2

Grain Size Distribution Curve

Particle Size (mm)

% Finer

0.001 0.01 0.1 1 10

D5 (7.5m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

0.01 0.1 1 10

D6 (9.0m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

0.001 0.01 0.1 1 10

D8 (12.0m)
Grain Size Distribution Curve

Chilmarí, Kurigram BH-2
Ramna Khamar, Kurigram BH-3

Grain Size Distribution Curve

- D1 (1.5m)
- D5 (7.5m)
- D6 (9.0m)
Ramna Khamar, Kurigram BH-3

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D16 (24.0m)

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D20 (30.0m)
Hard Point, Sirajganj BH-1

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D1 (1.5m)
- D4 (6.0m)
- D5 (7.5m)
Hard Point, Sirajganj BH-1

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D8 (12.0m)

D9 (10.5m)

D10 (15m)

Particle Size (mm)
Hard Point, Sirajganj BH-1

Grain Size Distribution Curve

- D11 (16.5m)

Grain Size Distribution Curve

- D12 (18.0m)

Grain Size Distribution Curve

- D14 (19.5m)
Hard Point, Sirajganj BH-1

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D17 (25.5m)

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D12 (18.0m)
Hard Point, Sirajganj BH-2

Grain Size Distribution Curve

- D2 (3.0m)

Grain Size Distribution Curve

- D5 (7.5m)

Grain Size Distribution Curve

- D12 (18.0m)
Hard Point, Sirajganj BH-2

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D20(30.0m)
Hard Point, Sirajganj BH-3

Grain Size Distribution Curve

- D2 (3.0m)
- D4 (6.0m)
- D7 (10.5m)
Hard Point, Sirajganj BH-3

Grain Size Distribution Curve

- **D10(15.0m)**

Grain Size Distribution Curve

- **D4 (6.0m)**

Grain Size Distribution Curve

- **D20(24.0m)**
Hard Point, Sirajganj BH-4

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D2 (1.5m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D4 (6.0m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D5 (7.5m)
Hard Point, Sirajganj BH-4

Grain Size Distribution Curve

- D6 (9.0m)
- D8 (12.0m)
- D10 (15.0m)
Hard Point, Sirajganj BH-4

Grain Size Distribution Curve

- D11 (16.5mm)
- D15 (22.5mm)
- D20 (30.0mm)
Satkhira BH-1

Grain Size Distribution Curve

- D8 (12.0m)

Grain Size Distribution Curve

- D9 (12.0m)

Grain Size Distribution Curve

- D10 (15.0m)
Satkhira BH-1

Grain Size Distribution Curve

- **D11 (16.5m)**

Grain Size Distribution Curve

- **D16 (24.0m)**

Grain Size Distribution Curve

- **D20 (30.0m)**
Satkhira BH-2

Grain Size Distribution Curve

D1 (1.5m)

Grain Size Distribution Curve

D2 (3.0m)

Grain Size Distribution Curve

D3 (4.5m)
Satkhira BH-2

Grain Size Distribution Curve

- D4 (6.0m)

Grain Size Distribution Curve

- D10 (15.0m)

Grain Size Distribution Curve

- D11 (16.5m)
Satkhira BH-2

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D14 (21.0m)
- D17 (25.5m)
- D20 (30.0m)
Patharghata, Barguna BH-1

Grain Size Distribution Curve

- D1 (1.5m)

Grain Size Distribution Curve

- D5 (7.5m)

Grain Size Distribution Curve

- D6 (9.0m)
Patharghata, Barguna BH-1

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D9 (13.5m)

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D15 (22.5m)

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D18 (27.0m)
Patharghata, Barguna BH-2

Grain Size Distribution Curve

- D1 (1.5m)

Grain Size Distribution Curve

- D3 (4.5m)

Grain Size Distribution Curve

- D5 (7.5m)
Patharghata, Barguna BH-2

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

D6 (9.0m)

D8 (12.0m)

D10 (15.0m)
Patharghata, Barguna BH-2

Grain Size Distribution Curve

D12 (18.0m)

Grain Size Distribution Curve

D18 (27.0m)

Grain Size Distribution Curve

D20 (30.0m)
Gabbari, Puratonbad, Barguna BH-1

Grain Size Distribution Curve

% Finer

Particle Size (mm)
**Natunbad, Barguna BH-2**

Grain Size Distribution Curve

- **D2 (3.0m)**

Grain Size Distribution Curve

- **D4 (6.0m)**

Grain Size Distribution Curve

- **D6 (9.0m)**
Natunbad, Barguna BH-2

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D11 (16.5m)

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D14 (21.0m)
Gabbari, Natunbad, Barguna BH-3

Grain Size Distribution Curve

**D1 (1.5m)**

Grain Size Distribution Curve

**D3 (4.5m)**

Grain Size Distribution Curve

**D5 (7.5m)**
Gabbari, Natunbad, Barguna BH-3

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D10 (15.0m)

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D13 (19.5m)
Gabbari, Puratonbad, Barguna BH-4

Grain Size Distribution Curve

- D1 (1.5m)
- D4 (6.0m)
- D9 (13.5m)
Gabbari, Puratonbad, Barguna BH-4

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D13 (19.5m)
- D16 (24.0m)
- D20 (30.0m)
Grain Size Distribution Curve

 Particle Size (mm)

 % Finer

 0 10 20 30 40 50 60 70 80 90 100

 0.0001 0.001 0.01 0.1 1

 D2 (3.0m)

Grain Size Distribution Curve

 Particle Size (mm)

 % Finer

 0 10 20 30 40 50 60 70 80 90 100

 0.0001 0.001 0.01 0.1 1

 D3 (4.5m)

Grain Size Distribution Curve

 Particle Size (mm)

 % Finer

 0 10 20 30 40 50 60 70 80 90 100

 0.0001 0.001 0.01 0.1 1

 D4 (6.0m)

Ramgoti BH-1
Ramgoti BH-1

Grain Size Distribution Curve

- D5 (7.5m)

Grain Size Distribution Curve

- D6 (9.0m)

Grain Size Distribution Curve

- D7 (10.5m)
Ramgoti BH-1

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D8 (12.0mm)
- D9 (13.5mm)
- D11 (16.5mm)

Ramgoti BH-1
Ramgoti BH-1

Grain Size Distribution Curve

- D12 (18.0m)

Grain Size Distribution Curve

- D13 (19.5m)

Grain Size Distribution Curve

- D14 (21.0m)
Ramgoti BH-1

Grain Size Distribution Curve

- D16 (24.0m)

Grain Size Distribution Curve

- D17 (25.5m)

Grain Size Distribution Curve

- D18 (27.0m)
Ramgoti BH-1

Grain Size Distribution Curve

Particle Size (mm)
Charfasson, Bhola BH-1

Grain Size Distribution Curve

% Finer

Particle Size (mm)

0.01 0.1 1 10

D10 (15.0m)

Grain Size Distribution Curve

% Finer

Particle Size (mm)

0.01 0.1 1 10

D12 (18.0m)

Grain Size Distribution Curve

% Finer

Particle Size (mm)

0.01 0.1 1 10

D14 (21.0m)
Charfasson, Bhola BH-1

Grain Size Distribution Curve

- D17 (25.5m)

Grain Size Distribution Curve

- D20 (30.0m)
Charfasson, Bhola BH-2

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

D1 (1.5m)

D6 (8.0m)

D10 (15.0m)
Charfasson, Bhola BH-2

Grain Size Distribution Curve

- D13 (19.5m)

Grain Size Distribution Curve

- D16 (24.0m)

Grain Size Distribution Curve

- D19 (28.5m)
Charfasson, Bhola BH-3

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- **D1 (1.5m)**

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- **D6 (8.0m)**

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- **D8 (9.0m)**
Charfasson, Bhola BH-3

Grain Size Distribution Curve

- D11 (16.5m)
- D14 (21.0m)
- D17 (25.5m)
Charfasson, Bhola BH-4

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D1 (1.5m)
- D4 (6.0m)
- D8 (9.0m)
Charfasson, Bhola BH-4

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D18 (27.0m)

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D20 (30.0m)
Anwara, BH-2

Grain Size Distribution Curve

- **D5 (7.5m)**

Grain Size Distribution Curve

- **D6 (9.0m)**

Grain Size Distribution Curve

- **D7 (10.5)**
Anwara, BH-2

Grain Size Distribution Curve

% Finer vs. Particle Size (mm)

- D8 (12.0m)
- D9 (13.5m)
- D11 (16.5m)
Anwara, BH-2

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D14 (21.0m)
Sitakundu BH-1

Grain Size Distribution Curve

D4 (6.0m)

Grain Size Distribution Curve

D5 (7.5m)

Grain Size Distribution Curve

D6 (9.0m)
Sitakundu BH-2

Grain Size Distribution Curve

- **D1 (1.5m)**

Grain Size Distribution Curve

- **D2 (3.0m)**

Grain Size Distribution Curve

- **D3 (4.5m)**
Sitakundu BH-2

Grain Size Distribution Curve

- D4 (6.0m)

Grain Size Distribution Curve

- D5 (7.5m)

Grain Size Distribution Curve

- D6 (9.0m)
Sitakundu BH-2

Grain Size Distribution Curve

- **D13 (19.5m)**

Grain Size Distribution Curve

- **D16 (24.0m)**

Grain Size Distribution Curve

- **D19 (28.5m)**
Sitakundu BH-2

Grain Size Distribution Curve

% Filter vs. Particle Size (mm)

D20 (30m)
Grain Size Distribution Curve

% Finer

Particle Size (mm)

- D1 (1.5m)

- D2 (3.0m)

- D3 (4.5m)

Maheshkhali BH-1
Maheshkhali BH-2

Grain Size Distribution Curve

Particle Size (mm)

% Finer

D1 (1.5m)

D2 (3.0m)

D3 (4.5m)
Maheshkhali BH-2

Grain Size Distribution Curve

- D4 (6.0m)

Grain Size Distribution Curve

- D5 (7.5m)

Grain Size Distribution Curve

- D6 (9.0m)
Maheshkhali BH-2

Grain Size Distribution Curve

% Finer

Particle Size (mm)

D7 (10.5m)
APPENDIX C
ATTERBERG LIMIT TEST RESULTS
Liquid Limit Test Anwara

Anwara BH-1

Liquid Limit = 34

Anwara BH-2

Liquid Limit = 42
Barguna Puratonbad BH-1

Liquid Limit = 42

Barguna Puratonbad BH-4

Liquid Limit = 39
Liquid Limit Test Char Fasson

Char Fasson BH-1

Liquid Limit = 37

Char Fasson BH-2

Liquid Limit = 36
Liquid Limit Test Char Fasson

Char Fasson BH-3

Liquid Limit = 33

Char Fasson BH-4

Liquid Limit = 32
Liquid Limit Test Kurigram and Satkhira

Kurigram BH-2

Liquid Limit = 39

Shatkhira BH-1

Liquid Limit = 34

y = -0.2389x + 44.384

y = -0.2114x + 39.769

Moisture Content %

Number of Blows, N (log scale)
Liquid Limit Test Maheshkhali

**Maheshkhali BH-1**

Liquid Limit = 29

**Maheshkhali BH-2**

Liquid Limit = 32
Liquid Limit Test Patharghata

**Patharghata BH-1**

Liquid Limit = 32

**Patharghata BH-2**

Liquid Limit = 41
Liquid Limit Test Ramgoti

Ramgoti BH-1

Liquid Limit = 33

Ramgoti BH-2

Liquid Limit = 37
Liquid Limit Test Sirajganj and Tanki Bazar

**Sirajganj BH-4**

Liquid Limit = 31

**Tanki Bazar BH-2**

Liquid Limit = 43
APPENDIX D

CONSOLIDATED DRAINED DIRECT SHEAR TEST RESULTS
Consolidated Drained Direct Shear Test Anwara BH-1

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal load 124 kPa

Peak Shear Stress (kPa) vs. Normal Stess (kPa)
- y = 0.541x + 4.6
- 124, 70.000
- 62, 43.200
- 31, 18.000

Effective Cohesion, c' (kPa) = 4.6
Effective Angle of Internal Friction φ' (°) = 28.5
Consolidated Drained Direct Shear Test Anwara BH-2

**Graph 1:**
- **Shear Stress (kPa)** vs. **Shear Displacement (mm)**
- Data points for normal loads: 62 kPa, 31 kPa, 124 kPa
- Equation: $y = 0.5667x$

**Graph 2:**
- **Peak Shear Stress (kPa)** vs. **Normal Stress (kPa)**
- Data points: (31, 17.057), (62, 33.792), (124, 71.065)

- Effective Cohesion, $c'$ (kPa) = 0.0
- Effective Angle of internal friction $\phi'$ (°) = 29.5
Consolidated Drained Direct Shear Test Barguna BH-1

Effective Cohesion, $c'$ (kPa) = 2.5
Effective Angle of internal friction $\phi'$ (°) = 28.0
Consolidated Drained Direct Shear Test Barguna BH-2

Shear Stress (kPa) vs. Shear Displacement (mm)

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal Load 124 kPa

Effective Cohesion, $c'$ (kPa) = 0.0
Effective Angle of internal friction $\phi'$ (°) = 35.0

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

- Normal Load 62 kPa: 31, 25.678
- Normal Load 31 kPa: 62, 45.456
- Normal Load 124 kPa: 124, 85.264

$y = 0.703x$
Consolidated Drained Direct Shear Test Barguna BH-3

Effective Cohesion, $c'$ (kPa) = 9.1
Effective Angle of internal friction $\phi'$ (°) = 20.8
Consolidated Drained Direct Shear Test Barguna BH-4

Shear Stress (kPa) vs Shear Displacement (mm)

- Normal Load 62 kPa
- Normal load 124 kPa
- Normal Load 31 kPa

Normal Stess (kPa) vs Peak Shear Stress (kPa)

- y = 0.5276x + 3.9993
- 24, 69.54389
- 62, 36.32786
- 31, 20.60730

Effective Cohesion, $c'$(kPa) = 4.0
Effective Angle of internal friction $\phi'$(°) = 27.8
Consolidated Drained Direct Shear Test Charfasson BH-1

Shear Stress (kPa)

Normal load 62 kPa  Normal load 31 kPa  Normal load 124 kPa

Shear Stress (kPa)

Normal Stress (kPa)

Peak Shear Stress (kPa)

Effective Cohesion, $c'(kPa) = 6.8$
Effective Angle of internal friction $\phi'(\degree) = 31.2$

$y = 0.6082x + 6.7769$
Consolidated Drained Direct Shear Test Charfasson BH-2

Shear Stress (kPa) vs. Shear Displacement (mm)
- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal Load 124 kPa

Peak Shear Stress (kPa) vs. Normal Stress (kPa)
- 31, 18
- 62, 40
- 124, 73

Effective Cohesion, $c'$ (kPa) = 1.5
Effective Angle of internal friction $\phi'$ (°) = 30.2
Consolidated Drained Direct Shear Test Charfasson BH-3

Shear Stress (kPa) vs. Shear Displacement (mm)

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal Load 124 kPa

Effective Cohesion, \( c' \) (kPa) = 7.3
Effective Angle of internal friction \( \phi' \) (°) = 31.2

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

\[ y = 0.6057x + 7.3 \]
Consolidated Drained Direct Shear Test Charfasson BH-4

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal load 124 kPa

Effective Cohesion, $c'$ (kPa) = 8.05
Effective Angle of internal friction $\phi'$ (°) = 34.1
Consolidated Drained Direct Shear Test Gaibandha BH-1

Shear Stress (kPa) vs. Shear Displacement (mm)

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal load 124 kPa

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

- Effective Cohesion, $c' (kPa) = 9.3$
- Effective Angle of internal friction $\phi' (°) = 24.5$

$y = 0.4563x + 9.325$

Points:
- 31, 24.2
- 62, 36.6
- 124, 66.3
Consolidated Drained Direct Shear Test Gaibandha BH-2

Shear Stress (kPa) vs. Shear Displacement (mm)

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal load 124 kPa

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

- y = 0.6082x + 3.5495

Effective Cohesion, c’ (kPa) = 3.6
Effective Angle of internal friction \( \phi' \) (°) = 31.3
Consolidated Drained Direct Shear Test Kurigram BH-1

- Effective Cohesion, \( c' \) (kPa) = 5.21
- Effective Angle of internal friction \( \phi' \) (°) = 28.2
Consolidated Drained Direct Shear Test Kurigram BH-2

Shear Stress (kPa) vs. Shear Displacement (mm)

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal Load 124 kPa

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

- y = 0.5334x

Effective Cohesion, c' (kPa) = 0.0
Effective Angle of internal friction $\phi'$ (°) = 28.0
**Consolidated Drained Direct Shear Test Maheshkhali BH-1**

- **Shear Stress (kPa)** vs **Shear Displacement (mm)**
  - Normal Load 62 kPa
  - Normal Load 31 kPa
  - Normal load 124 kPa

- **Normal Stress (kPa)** vs **Peak Shear Stress (kPa)**
  - Effective Cohesion, $c'$ (kPa) = 0.64
  - Effective Angle of internal friction $\phi'$ (°) = 28.9

- The regression line is given by $y = 0.5562x + 0.6454$

- Data points:
  - (62, 38.725)
  - (124, 68.415)
  - (31, 15.490)
Consolidated Drained Direct Shear Test Maheshkhali BH-2

Shear Stress (kPa) vs. Shear Displacement (mm)

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal Load 124 kPa

Effective Cohesion, $c'$ (kPa) = 2.6
Effective Angle of internal friction $\phi'$ (°) = 29.7

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

$y = 0.5711x + 2.581$

- 62, 37.4
- 31, 20.7
- 124, 73.6
Consolidated Drained Direct Shear Test Patharghata BH-1

\[ y = 0.735x + 4.5 \]

Effective Cohesion, \( c' \text{ (kPa)} = 4.5 \)
Effective Angle of internal friction \( \phi' ^{\circ} = 36.0 \)
Consolidated Drained Direct Shear Test Ramgoti BH-1

Shear Stress (kPa) vs. Shear Displacement (mm)

Normal Load 62 kPa
Normal Load 31 kPa
Normal Load 124 kPa

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

$y = 0.6463x + 0.1658$

Effective Cohesion, $c'(kPa) = 0.16$
Effective Angle of internal friction $\phi'(\degree) = 32.9$
Consolidated Drained Direct Shear Test Ramgoti BH-2

Shear Stress (kPa) vs. Shear Displacement (mm)

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal load 124 kPa

Effective Cohesion, $c'$ (kPa) = 5.1
Effective Angle of internal friction $\phi'$ (°) = 27.6
Consolidated Drained Direct Shear Test Satkhira BH-1

Shear Stress (kPa) vs Shear Displacement (mm)

- Normal Load 30 kPa
- Normal Load 15 kPa
- Normal load 60 kPa

Peak Shear Stress (kPa) vs Normal Stress (kPa)

- y = 0.443x + 8.145
- Effective Cohesion, c' (kPa) = 8.1
- Effective Angle of internal friction $\phi'$ (°) = 23.9
Consolidated Drained Direct Shear Test Satkhira BH-2

Shear Stress (kPa) vs. Normal Stess (kPa) graph:
- Normal Load 30 kPa
- Normal Load 15 kPa
- Normal load 60 kPa

Peak Shear Stress (kPa) vs. Normal Stress (kPa) graph:
- $y = 0.4286x + 6$
- $60, 32.0$
- $30, 18.0$
- $15, 13.0$

Effective Cohesion, $c'(kPa) = 6.0$
Effective Angle of internal friction $\phi'(\degree) = 23.6$
Consolidated Drained Direct Shear Test Sirajganj BH-4

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal Load 124 kPa

Shear Stress (kPa) vs. Shear Displacement (mm)

- Effective Cohesion, $c'$ (kPa) = 0.0
- Effective Angle of internal friction $\phi'$ (°) = 38.1

$y = 0.7548x$
Effective Cohesion, $c'$ (kPa) = 2.4
Effective Angle of internal friction $\phi'$ (°) = 33.7
Consolidated Drained Direct Shear Test Tankibazar BH-2

- Normal Load 62 kPa
- Normal Load 31 kPa
- Normal Load 124 kPa

Shear Stress (kPa) vs. Shear Displacement (mm)

Peak Shear Stress (kPa) vs. Normal Stress (kPa)

Effective Cohesion, $c'$ (kPa) = 0.0
Effective Angle of internal friction $\phi'$ (°) = 29.6
APPENDIX E
ONE DIMENSIONAL CONSOLIDATION TEST RESULTS
Consolidation Test Result Anwara BH-2 UD-2

e-logp curve

Void Ratio, $e$

Pressure $p$ in kPa (log scale)

Loading
Unloading

$C_c = 0.259$

$C_s = 0.016$
Consolidation Test Result Barguna Notunbad BH-2 UD-2

Void Ratio, e

Pressure p in kPa (log scale)

Loading

Unloading

Cc = 0.182

Cs = 0.028
Consolidation Test Result Charfasson BH-3 UD-2

e-logp curve

Void Ratio, e

Pressure p in kPa (log scale)

$C_c = 0.206$

$C_s = 0.003$
Consolidation Test Result Char fasson BH-2 UD-2

e-logp curve

Void Ratio, e

Pressure p in kPa (log scale)

$C_c = 0.199$

$C_s = 0.044$
Consolidation Test Result Maheshkhali BH-1 UD-2

e-logp curve

Void Ratio, e

Pressure p in kPa (log scale)

Cc = 0.239

Cs = 0.013
Consolidation Test Result Maheshkali BH-2 UD-2

e-logp curve

Void Ratio, e

Pressure p in kPa (log scale)

Cc=0.188

Cs=0.144
Consolidation Test Result Patharghata BH-1 UD-2

e-logp curve

Void Ratio, $e$

Pressure p in kPa (log scale)

$C_e=0.193$

$C_s=0.025$
Consolidation Test Result Patharghata BH-2 UD-2

e-logp curve

Void Ratio, e

Pressure p in kPa (log scale)

$C_c = 0.202$

$C_c = 0.016$
Consolidation Test Result Ramgoti BH-2 UD-2

Void Ratio, e

Pressure p in kPa (log scale)

$C_e = 0.188$

$C_s = 0.032$
Consolidation Test Result Ramgoti BH-1 UD-2

$e$-log$p$ curve

\begin{align*}
Cc &= 0.186 \\
Cs &= 0.016
\end{align*}
Consolidation Test Result Satkhira BH-3 UD-2

e-logp curve

Void Ratio, e

Pressure p in kPa (log scale)

Consolidation Test Result Satkhira BH-3 UD-2

$C_c = 0.157$

$C_s = 0.029$
Consolidation Test Result Satkhira BH-2 UD-2

Void Ratio, $e$

Pressure $p$ in kPa (log scale)

e-logp curve

$C_c=0.200$

$C_s=0.009$
Consolidation Test Result Tanki Bazar BH-1 UD-2

Void Ratio, $e$

Pressure $p$ in kPa (log scale)

e-logp curve

$C_e = 0.170$

$C_e = 0.004$
APPENDIX F

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
Triaxial Test Anwara BH-1

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Deviator stress vs Axial strain 200 kPa

Mohr circle plot Anwara BH-1

- Effective stress 50kPa
- Effective stress 100kPa
- Effective stress 200kPa
- Failure Curve

Effective stress 50kPa
Effective stress 100kPa
Effective stress 200kPa
Failure Curve

Failure Curve

Undrained Cohesion, \( c_u \) (kN/m\(^2\)) = 70.0
Undrained Angle of Internal Friction, \( \phi_u \) (°) = 24.8
Triaxial Test Anwara BH-2

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Mohr circle plot Anwara BH-2

Effective stress 50kPa
Effective stress 100kPa
Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 10.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 28.3°
Triaxial Test Barguna BH-1

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Mohr circle plot Barguna BH-1

Effective stress 50 kPa
Effective stress 100 kPa
Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 36.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 23.3
Undrained Cohesion, $c_u$ (kN/m$^2$) = 28.5
Undrained Angle of Internal Friction, $\phi_u$ (°) = 27.0
Triaxial Test Barguna BH-3

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Mohr circle plot Barguna BH-3

- Effective stress 50 kPa
- Effective stress 100 kPa
- Failure Curve

Undrained Cohesion, \( c_u (kN/m^2) = 40 \)
Undrained Angle of Internal Friction, \( \phi_u (^\circ) = 25.0 \)
Triaxial Test Barguna BH-4

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Mohr circle plot Barguna BH-4

Effective stress 50 kPa
Effective stress 100 kPa
Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 26.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 24.5
Triaxial Test Charfasson BH-1

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Deviator stress vs Axial strain 200 kPa

Mohr circle plot Charfasson BH-1

- Effective stress 50kPa
- Effective stress 100kPa
- Effective stress 200kPa
- Failure Curve

Undrained Cohesion, $c_u$ (kN/m$^2$) = 50.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 30.0
Triaxial Test Chorfasson BH-2

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Deviator stress vs Axial strain 200 kPa

Mohr circle plot Chorfasson BH-2

- Effective stress 50kPa
- Effective stress 100kPa
- Effective stress 200kPa
- Failure Curve

Undrained Cohesion, \( c_u \) (kN/m\(^2\)) = 50.0
Undrained Angle of Internal Friction, \( \phi_u \) (°) = 32.0
Triaxial Test Charfasson BH-3

Mohr circle plot Charfasson BH-3

- Effective stress 30kPa
- Effective stress 60kPa
- Effective stress 120kPa
- Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 60.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 34.0
Triaxial Test Kurigram BH-2

Deviator stress vs Axial strain 60kPa

Deviator stress vs Axial strain 120 kPa

Mohr circle plot Kurigram BH-2

Peak Shear Stress, \( \tau \) (kPa)

Normal Stress, \( \sigma \) (kPa)

Undrained Cohesion, \( c_u \) (kN/m\(^2\)) = 50.0
Undrained Angle of Internal Friction, \( \phi_u \) (°) = 35.0
Triaxial Test Maheshkhali BH-1

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Mohr circle plot Maheshkhali BH-1

Effective stress 50 kPa
Effective stress 100 kPa
Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 40
Undrained Angle of Internal Friction, $\phi_u$ (°) = 23.0
Triaxial Test No Moheshkhali BH-2

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100 kPa

Mohr circle plot Moheshkhali BH-2

- Effective stress 50 kPa
- Effective stress 100 kPa
- Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 78.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 32.0
**Triaxial Test Patharghata BH-1**

### Deviator stress vs Axial strain 50kPa

- **Deviator stress, \( \sigma_d \) (kPa)**: 0, 20, 40, 60, 80, 100, 120, 140, 160, 180
- **Axial strain, \( \varepsilon \) (%):** 0, 1, 2, 3, 4, 5, 6

### Deviator stress vs Axial strain 100 kPa

- **Deviator stress, \( \sigma_d \) (kPa)**: 0, 50, 100, 150, 200, 250, 300, 350, 400, 450
- **Axial strain, \( \varepsilon \) (%):** 0, 5, 10, 15, 20

### Mohr circle plot Patharghata BH-1

#### Effective stress 50 kPa
- Undrained Cohesion, \( c_u \) (kN/m\(^2\)) = 30.0
- Undrained Angle of Internal Friction, \( \varphi_u \) (°) = 23.0

#### Effective stress 100 kPa
- **Failure Curve**: \( 23.0° \)

### Peak Shear Stress, \( \tau \) (kPa)

- **Normal Stress, \( \sigma \) (kPa)**: 0, 100, 200, 300, 400, 500

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**Legend**

- Orange: Effective stress 50 kPa
- Blue: Effective stress 100 kPa
- Black: Failure Curve
**Triaxial Test Pathorghata BH-2**

### Deviator stress vs Axial strain 50kPa

- **Deviator stress, \(\sigma_d\) (kPa):** 0, 50, 100, 150, 200, 250
- **Axial strain, \(\varepsilon\) (%):** 0, 1, 2, 3, 4, 5, 6, 7

### Deviator stress vs Axial strain 100 kPa

- **Deviator stress, \(\sigma_d\) (kPa):** 0, 100, 200, 300, 400, 500
- **Axial strain, \(\varepsilon\) (%):** 0, 5, 10, 15, 20

### Mohr circle plot Pathorghata BH-2

- **Peak Shear Stress, \(\tau\) (kPa):** 34.0
- **Normal Stress, \(\sigma\) (kPa):** 0, 100, 200, 300, 400, 500
- **Effective stress 50 kPa**
- **Effective stress 100 kPa**
- **Failure Curve**

**Mohr's circle parameters:**
- **Undrained Cohesion, \(c_d\) (kN/m²):** 46.0
- **Undrained Angle of Internal Friction, \(\phi_d\) (°):** 34.0
Triaxial Test Ramgoti BH-1

Deviator stress vs Axial strain 50kPa

Deviator stress vs Axial strain 100kPa

Mohr circle plot Ramgoti BH-1

Effective stress 50 kPa
Effective stress 100 kPa
Failure Curve

Undrained Cohesion, \(c_d\) (kN/m²) = 25.0
Undrained Angle of Internal Friction, \(\phi_d\) (°) = 36.0
Triaxial Test Satkhira BH-1

Deviator stress vs Axial strain 60kPa

Deviator stress vs Axial strain 120kPa

Mohr circle plot Satkhira BH-1

Effective stress 50 kPa
Effective stress 100 kPa
Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 10.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 24.0

24.0°
Triaxial Test Satkhira BH-2

Deviator stress vs Axial strain 60kPa

Deviator stress vs Axial strain 120 kPa

Mohr circle plot Satkhira BH-2

Effective stress 50 kPa
Effective stress 100 kPa
Failure Curve

Undrained Cohesion, $c_u$ (kN/m²) = 32.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 25.0
Triaxial Test Satkhira BH-3

Deviator stress vs Axial strain 60kPa

Deviator stress vs Axial strain 120 kPa

Mohr circle plot Satkhira BH-3

Effective stress 50 kPa
Effective stress 100 kPa
Failure Curve

Peak Shear Stress, $\tau$ (kPa)
Normal Stress, $\sigma$ (kPa)

Undrained Cohesion, $c_u$ (kN/m²) = 12.0
Undrained Angle of Internal Friction, $\phi_u$ (°) = 23.0