CORRELATION BETWEEN SPT-N VALUES, UNIT COHESION AND ANGLE OF INTERNAL FRICTION OF SOIL FOR GUWAHATI CITY

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CANDIDATE DECLARATION

I do hereby declare that the work presented in the dissertation report entitled "CORRELATION BETWEEN SPT-N VALUES, UNIT COHESION AND ANGLE OF INTERNAL FRICTION OF SOIL FOR GUWAHATI CITY" is an authentic record of my own work carried out in partial fulfillment of the requirement for the award of the degree of Master of Technology in Civil Engineering with specialization in Geotechnical Engineering under the supervision and guidance of Dr. Diganta Goswami, Associate Professor, Department of Civil Engineering, Assam Engineering College, Jalukbari, Guwahati-13, Assam. The matter embodied in this dissertation has not been submitted by me for the award of any other degree.

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ABSTRACT

The properties of soil play an important role in many practices for geotechnical engineering. Laboratory and in-situ test are performed to know about such properties of the soil which ultimately helps in determining the ultimate bearing capacity of the soil while designing of foundations. To determine the real values of these properties special techniques should be followed such as undisturbed samples and initial overburden pressures should be taken into consideration. However, many a times due to budget constraints, time limitations, poor laboratory conditions and laboratory tests might not be possible. In this situation, for preliminary investigation of the soil empirical correlations between the properties can be used.

Standard penetration test (SPT) provides a good opportunity to obtain these parameters without using of more laboratory tests. Standard penetration tests (SPT), rough measure the strength of soil. The great merit of this test and the main reason for its widespread use is that it is simple and inexpensive.

In this study, an attempt has been made to correlate the SPT-N value with that of the cohesion of soil and angle of internal friction which has been obtained from laboratory test, by analysing the data obtained from boreholes of different locations in Guwahati City, this empirical correlation determines how strongly SPT-N values, unit cohesion and angle of internal friction of soil are related. It will help in determining the cohesion and internal friction angle of the soil using SPT-N values in the field.

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INTRODUCTION

1.1 GENERAL

Estimating geotechnical parameters using in-situ tests such as the standard penetration test (SPT) is widely performed in civil engineering projects due to the lack of required types of equipment, time constraints, financial issues, and disturbed samples. Besides, geotechnical properties of various soils are widely studied to analyse their parameters and obtain the possible relations among them.

In general, various soil properties have been calculated using field and laboratory experiments, such as elastic and strength characteristics. There is an ability to discard conducting certain experiments in the absence of an appropriate budget, time constraints and a challenging field scenario. Instead, using data from adjoining sites or some statistical correlations are used to assess such soil properties. In the past, empirical correlations have been comprehensively used to estimate the soil properties for published data from various sources including the discrepancy of the test methods, test materials, and data explanation. The empirical soil correlations were established using field Standard Penetration Test values (SPT) N-value. The N-value for its simplicity is commonly used as a simple strength assessment index value. In addition, for calculating soil bearing capacity and shear wave velocity, the N-value is used. The most traditional procedure for general soil characterization is the Standard Penetration Test (SPT) field test. Obtaining a consistent soil correlation can be used in assisting the field engineers when the laboratory and in situ test results are unavailable where it can be beneficial in predicting the mechanical properties of the soil.

The parameters like unit cohesion, angle of internal friction of the soil etc. which is generally determined in the laboratory using various tests, can be determined from the N-values obtained from the Standard Penetration Test. The correlations between corrected N-values, unit cohesion of soil and angle of internal friction once derived, can be used for determining the unknown parameters in the field itself. These parameters once obtained can then be used for analysis of the soil type, foundation size etc. These correlations are tried to be established through this study.

1.2 GEOTECHNICAL CHARACTERISATION OF SOIL

Characterization of subsurface conditions is one of the most challenging yet important activities required for successful planning, design, construction, and operation of transportation infrastructure. The broad purpose of site characterization is to inform geotechnical specialists, planners, designers, constructors, and other professionals about ground conditions so that these decision makers can effectively identify and address risks attributed to ground conditions.

- **Role and value of site characterization**

The term "risk" is used to represent a potential for loss, generally expressed in terms of financial costs. The term is used qualitatively, but risk can also be expressed quantitatively as the product of some potential cost and the likelihood of that cost being incurred. In this context, the costs involved broadly include costs incurred to resolve construction or performance problems that may arise from failure to effectively characterize ground conditions. Costs may also often include indirect costs associated with reduced mobility or public safety. The likelihood of these potential costs depends on the reliability of measures taken to characterize ground conditions, which in turn depends on the reliability of available information regarding ground conditions.

Improved site characterization will generally reduce risks associated with design, construction, and operation of transportation infrastructure. Improved site characterization directly reduces the likelihood of encountering unforeseen ground conditions during construction, which often lead to claims, change orders, and cost overruns during construction, and may lead to unacceptable performance following construction.

Despite these considerable benefits, it is important to recognize that improved site characterization generally comes at some cost. Thus, there are consequences associated with conducting more extensive investigations. The value of investigations performed for site characterization is derived from the fact that costs for the investigations are often substantially less than costs associated with constructing features to accommodate uncertain and ambiguous ground conditions and potential costs that may be incurred if construction or performance problems are encountered.

- **Challenges for effective site characterization**

There are numerous challenges to effective site characterization, especially when considered in light of the fundamental value proposition. Several of these challenges include:

1. The volume of material to be characterized is generally very large in comparison to the volume of material that can be sampled and/or tested, even for the most extensive investigations.

2. The materials to be characterized are inherently heterogeneous with characteristics that can vary substantially in both space and time.

3. The mechanical behaviour of soil and rock is relatively complex and affected by many factors that can be difficult to understand and accurately replicate in laboratory and/or field tests.

4. A large number of different measurements can be made to evaluate different ground characteristics, each with different advantages and disadvantages, but the relative value of different types of measurements for specific conditions is generally not quantitatively established and often not widely recognized.

5. Site characterization is commonly performed in the early stages of projects, sometimes before locations and important details about specific structures are established, before anticipated loading conditions are completely defined, and/or before all potentially important limit states are identified.

6. The mechanical behaviour of soil and rock is influenced by groundwater conditions, which inevitably vary over time and are often difficult to predict.

7. Practical issues like site access constraints, regulatory requirements, and complex budgeting issues may restrict the type, quantity, and/or location of measurements that can be made.

These challenges, and others, lead to the condition that some level of uncertainty about ground conditions is inevitable, regardless of the scope of investigations performed. Perfect site characterization is simply not possible; thus, the objective for site characterization should be to characterize the site to some acceptable level commensurate with comparison of costs for investigation and costs and risks associated with design, construction, and operation of the features being considered. As risk is necessarily dependent on uncertainty, site characterization should also include characterization of the reliability or uncertainty of the information obtained in addition to establishing appropriate characteristics and parameters required for design and construction.

Similarly, hindsight may sometimes reveal that the scope of a specific investigation was insufficient or excessive, despite judicious decisions considering the expected value of the investigation. Unfortunately, sometimes doing the “right thing” does not lead to the desired outcome for specific projects. However, consistent consideration of the value proposition will lead to reduced risks and improved use of funds when considered over a large collection of individual projects that make up an agency’s portfolio of projects.

- **Objectives, uses, and products of site characterization investigations**

In the context of site characterization, the word “investigation” is used to represent a systematic study conducted to identify the ground conditions present at a site and to accurately characterize the behaviour of the soil and/or rock.

Investigations performed for site characterization may include subsurface investigations that involve boring, probing, excavation, or other testing below the ground surface; geophysical investigations that may or may not require access to the soil and/or rock below the ground surface; laboratory investigations that generally involve testing of soil and/or rock specimens acquired from the site; and examination of maps, imagery, and other information that often do not require access to the specific site.

Prior to planning and executing specific investigations for site characterization, it is important to fully understand the objectives and anticipated products of the investigations.

Planning and execution of investigations for site characterization will also be improved with knowledge of the anticipated or potential uses for such products.

- **General objectives for site characterization**

Specific investigations for site characterization are conducted for a broad range of reasons. The products of these investigations are also used for different purposes by different personnel. Such broad motivation and use contribute to the value and importance of site characterization since it directly contributes to and affects projects during planning, design, construction, and operation.

While products from the investigations are used for many purposes, specific investigations are generally performed with specific objectives in mind. The specific objectives being addressed can affect the quantity, scope, and type of investigations that should be performed, and the resulting value of the products developed from the investigations.

Investigations performed for site characterization are generally intended to address one or more of the following objectives:

1. **Stratigraphy** – To identify and qualitatively characterize the types of soil and/or rock present at a site, including definition of discrete strata and development of cross-sections (profiles) that describe how stratigraphy varies across a site.

2. **Groundwater Conditions** – To characterize groundwater conditions at a site and identify potential impacts that such conditions may have for design, construction, and operation.

3. **Design Parameters** –To develop reliable estimates for relevant design parameters needed for design.

4. **Constructability** – To characterize ground conditions that may affect construction methods and schedule, and identify risks that may impact project delivery.

5. **Hazard Identification** –To identify geotechnical hazards that are present and characterize those hazards that may impose risks to design, construction, operation, and/or performance.

6. **Suitability** –To characterize the suitability of soil and/or rock encountered for specific uses (e.g., use as engineered fill or aggregate source).

7. **Condition Assessment and Performance Monitoring** – To assess current ground conditions to inform condition assessment for transportation features.

8. **Location and Alignment** –To qualitatively characterize ground conditions to inform location and alignment decisions for infrastructure projects (e.g., identifying conditions that may necessitate substantial remediation or motivate re-alignment of a corridor).

The suitability of different types of investigations, and of specific site characterization methods, is intimately tied to the objectives of the investigation. Some field and laboratory tests that may be suitable for hazard identification or location and alignment studies may be poorly suited for characterizing geotechnical design parameters.

Conversely, field and laboratory tests that are well-suited for characterizing geotechnical design parameters may be poor choices for profiling or constructability evaluations. In many cases, specific field and laboratory techniques can be used to address several or even all of the objectives listed above. The suitability of specific techniques for addressing the disparate objectives listed above is addressed in subsequent chapters.

Unfortunately, it is often difficult to identify individual practices that will produce results that simultaneously address the objectives for all potential end-users. Nevertheless, it is helpful to identify relevant objectives for all involved parties, so that the value of geotechnical investigations can be enhanced.

- **Classes of site characterization investigations**

Because objectives for specific site characterization investigations can vary and sometimes be competing, different classes or phases of investigations are generally conducted to provide a systematic approach to site characterization. While the terminology used by different organizations to refer to different classes or phases of investigations varies, specific activities for different classes of investigations often share common characteristics and objectives as described in the following sections.

- **Desk Studies:**

A critical first step for practically all site characterization programs is a study to gather and evaluate available information about conditions that may be present at a site. These activities are often referred to as "desk studies" because they are often completed prior to conducting field or laboratory activities. Common sources of information for desk studies include:

- Historical records from prior site investigations at or near a project site;
- Performance records from nearby structures or facilities, potentially even including review of articles from the popular press;
- Geologic reports and publications;
- Various maps that commonly include geologic maps, soil survey maps, topographic maps, utility maps, insurance maps, etc.
- Aerial photographs, satellite imagery, and other remote-sensing products;
- Consultation with professional colleagues with experience at or near a project site.
- Review of pertinent laws, policies, and regulations that may govern a particular site.

These sources of information contribute to characterization of a site and development of practical, yet effective geotechnical site characterization programs.

Desk studies inform planning and scoping of investigations for site characterization and help ensure that field crews are suitably equipped and prepared for the conditions that are likely to be encountered.

- **Preliminary Investigations:**

Preliminary investigations are usually performed in the early stages of project development to support project planning, to provide somewhat simple and often qualitative information for preliminary design, and to provide information to support planning for more rigorous investigations.

Preliminary investigations also often contribute to early identification and characterization of potential risks that may be imposed by geotechnical hazards.

For roadway projects, preliminary investigations often include relatively sparse subsurface investigations that may include shallow borings, test pits, and/or in situ test soundings located at relatively large spacing along the anticipated project alignment.

Preliminary investigations often include collection of bulk or disturbed samples that are used for simple "index property" tests that support initial characterization of the soil/rock present. Preliminary investigations seldom include collection of higher quality samples that are appropriate for "performance" tests to measure strength and stress-strain properties for soil and rock.

- **Design Investigations:**

Design investigations are typically performed after the project alignment and grade have been set and after locations for retaining walls, bridge piers, bridge abutments, and cut/fill slopes have been established.

Compared to preliminary investigations, design investigations are more rigorous and "targeted" investigations that usually involve more advanced, and more costly field and laboratory techniques.

The principal objective for design investigations is to: (1) confirm or refine preliminary interpretations of site stratigraphy, and (2) establish reliable values for relevant design parameters.

For large projects, several phases of design investigations may be performed, either to address different design and construction issues, or to sequentially refine interpretations as more information is acquired and as specific details of a project are established.

Conversely, for small projects, preliminary investigations and design investigations may be combined into a single investigation, although this practice introduces some risk into the site characterization process.

In addition to routine design investigations, "special" investigations are sometimes performed to investigate specific geotechnical challenges for a project. Such investigations often include field and/or laboratory tests that are not commonly performed for routine design investigations and often include greater quantities of tests than would be performed for routine projects.

- **Borrow Site Investigations:**

For projects that require substantial quantities of off-site “borrow” materials for construction, borrow site investigations may be performed to evaluate the suitability of potential borrow sources.

Borrow site investigations generally focus on characterizing stratigraphy and general soil and/or rock type, as opposed to characterizing in situ soil properties, since the soil properties will change as a result of excavation, transport, and placement of the borrow soil or rock.

Borrow site investigations may also include test pits and other destructive means to characterize stratigraphy and to obtain relatively large quantities of disturbed samples for further laboratory characterization. As borrow sources are refined, and the suitability of a particular source is confirmed, borrow site investigations may include more extensive laboratory testing on compacted specimens of the borrow material to further characterize relevant engineering properties for design.

1.2.1 Geotechnical reporting document

A number of different documents are generated from investigations for site characterization. The most common of these include field investigation logs, geotechnical data reports, and geotechnical design reports. The following sections provide general descriptions of different geotechnical reporting documents for the purpose of understanding common products that result from site characterization.

- **Field Investigation Logs:**

The most common products from investigations for site characterization are field investigation logs that may include boring logs, test pit logs, in situ testing logs, groundwater monitoring logs, and geophysical survey reports.

While the specific form and content of field investigation logs vary substantially for different types of investigations, and from agency to agency, the logs generally include some representation of stratigraphy determined from the investigation, engineering descriptions of the soil and rock materials encountered, documentation of groundwater level observations, as well as test measurements from laboratory or field tests.

Field investigation logs should also preferably include important field observations that may impact decision making such as difficult drilling, loss of drilling fluid, “rod drops” that may indicate karst, and borehole stability problems that may indicate flowing sands.

- **Geotechnical Data Reports:**

Products from geotechnical investigations commonly include some form of "geotechnical data report", which generally includes a description of the investigations performed, field investigation logs, and results of laboratory and field test measurements.

As is true for field investigation logs, the content of geotechnical data reports varies substantially with the type of investigations that are performed with the requirements for the specific project.

Geotechnical data reports produced from preliminary investigations are commonly brief and include only measurements from relatively simple index property tests that facilitate identification and classification of soil and/or rock and potential geotechnical hazards.

Conversely, geotechnical data reports from design investigations or special investigations may be rather lengthy and include results from large numbers of lab and/or field measurements of both index properties and "performance" properties.

Geotechnical data reports may also be produced for geophysical investigations (e.g., seismic velocity or electrical resistivity) or other highly specialized investigations. In some cases, geotechnical data reports may also include relevant existing data collected from desk studies.

An important characteristic of all geotechnical data reports is that they include only factual data; geotechnical data reports should not include interpretations derived from reported measurements or recommendations for design and construction. Because only factual information is provided, geotechnical data reports are sometimes included as part of project plans and bid documents, and are an essential part of design-build contracts.

As such, geotechnical data reports may become legally binding and may often influence potential claims and change orders, and overall project costs.

- **Geotechnical Design Reports:**

In contrast with geotechnical data reports, "geotechnical design reports" generally include much more than just factual data. Geotechnical design reports usually include relatively detailed descriptions of a characterized site along with additional content such as descriptions of analysis and design methods, results from design analyses, interpretation of analysis results, and recommendations for design and construction.

Geotechnical design reports often include descriptions of the soil and/or rock encountered, interpretations of stratigraphy, descriptions of observed and anticipated groundwater conditions, descriptions of geotechnical hazards and potential risks that may be introduced by those hazards, and interpretations of relevant geotechnical design parameters.

Geotechnical design reports also often include much of the factual information that is included in geotechnical data reports to support the interpretations provided. Alternatively, geotechnical design reports may reference one or more geotechnical data reports. Because geotechnical design reports often include rather complete and comprehensive interpretations of ground conditions, and because they often include much or most of the factual information collected, geotechnical design reports can be considered as the “complete” characterization of a site and the ultimate end product of site characterization activities.

However, because geotechnical design reports often include subjective interpretations about ground conditions based on available information, the reports are rarely, if ever, included as part of project plans and bid documents.

Nevertheless, geotechnical design reports document design assumptions, parameters and procedures, and design and construction considerations, and may have different legal ramifications depending upon the project location and prevailing law.

1.2.2 Benefits of site characterization

The fundamental value of site characterization is derived from benefits that arise during planning, design, construction, and operation of transportation infrastructure. These benefits include direct financial benefits as well as improved public safety and mobility that are more difficult to express in financial terms. If the financial benefits produced from site characterization exceed the cumulative costs for those investigations, the investigations contribute value to the project and the funding agency.

Conversely, effective site characterization increases confidence about ground conditions, which in turn reduces the likelihood of claims, change orders, and cost overruns, and reduces risks associated with ground conditions, both of which can produce substantial cost savings for agencies and projects.

1.2.3 Planning and scoping for site characterization activities

Planning and scoping for site characterization activities is challenging due to the numerous requirements and constraints for characterization and the wide variety of available techniques and approaches.

Planning and scoping are further complicated because the reliability of the acquired information, and more specifically the reliability of estimates for design parameters, is dependent on both the quantity and quality of measurements that are made. Thus, many possible alternative scopes can be developed, with a wide range of resulting reliabilities for the acquired information.

The first and third requirements are generally similar for most projects, although specific measurements may vary from one site to the next based on site characteristics. For example, greater numbers of borings or in situ test soundings are appropriate for defining stratigraphy at sites with highly variable stratigraphy than at sites with more consistent stratigraphy.

Similarly, the types of borings or measurements made to identify hazards may vary substantially depending on the geologic setting and the potential for encountering specific hazards.

Use of design parameters derived from in situ tests is also quite common as a surrogate for more fundamental soil and rock properties. The specific parameters required for a project depend on a number of factors that include:

- Soil or rock type
- Type of geotechnical feature being designed
- Specific limit states or design conditions being evaluated
- Agency performance requirements, design practices, and policies (i.e., design methods)
- Agency or site investigation contractor equipment and capabilities

It is also common that specific features may be designed using one of several alternative methods that require design parameters derived from different types of measurements. For example, driven piles may be designed using so-called “rational” methods that require fundamental measures of soil or rock strength as inputs.

Alternatively, driven piles may be designed using methods that require parameters obtained from in situ test measurements such as Standard Penetration Tests (SPT) or Cone Penetration Tests (CPT).

Specific requirements for site characterization therefore depend on the design methods that will be used.

As such, it is important to maintain awareness of agency practices and communicate with those that use the products of site characterization to ensure that appropriate types of measurements are obtained.

Due to the wide range of alternative methods that may be used for site characterization, and because of the additional challenges described, appropriate scopes for site investigations should be developed considering the following activities:

1. Communicating with project managers, designers, and/or owners to develop a thorough understanding of the broader project, including special constraints, anticipated schedule, anticipated method of procurement, etc.
2. Assessing soil and/or rock types that are anticipated to exist at the site.
3. Identifying appropriate design conditions and limit states, along with associated design and analysis methods that will be used for design.
4. Identifying design parameters that are required for the identified design and analysis methods.
5. Identifying constructability issues that may exist and establishing measurements needed to inform constructability decisions.
6. Identifying appropriate site characterization methods for establishing the identified design parameters, with consideration for agency and site investigation contractor capabilities.
7. Evaluating the relative merits of alternative types of measurements for the respective design parameters.
8. Estimating the number of measurements required to establish values for the identified design parameters with an appropriate level of reliability.
9. Developing a scope and plan for investigations that will produce the appropriate number and type(s) of measurements, with appropriate consideration for cost and schedule.
10. Communicating the scope and plan for site characterization to those that will execute the investigations (which may range from relatively straightforward internal communications to more complex legal and procurement documents for contracting site characterization investigations).

Most of these activities require deliberate coordination and communication among designers and users of geotechnical reports. The activities are often best completed by geotechnical design engineers in consultation with geologists, structural design engineers, construction engineers, project managers, and field crews.

1.2.4 Collection and interpretation of existing information

Collection and interpretation of existing information is one of the most effective ways to improve planning and scoping for site characterization, which in turn improves the effectiveness of site characterization programs.

Existing information informs those that plan and execute site characterization programs about ground conditions that can be anticipated so that appropriate means and methods for investigation can be selected. Existing information also often serves as the initial basis for identifying potential hazards, which in turn may dictate the type and scope of investigations that should be performed, and influence planning and decision making for the broader transportation project.

Capabilities and tools for collecting and evaluating existing information have improved dramatically over the past decade.

1.2.5 Identification and classification of soil and rock

Measurements from relatively simple laboratory and field "index" tests are commonly collected for all projects because they provide an inexpensive way to identify and formally classify soil and rock encountered at a site.

Common index tests for soils include water content, unit weight, Atterberg limits, particle-size distribution, and specific gravity. Hardness, unit weight, abrasivity, and durability are often measured for intact rock as are several measurements performed on rock core.

- **Objectives for identification and classification of soil and rock**

Formal soil and rock classifications derived from laboratory and field index property measurements are commonly used for several purposes. Index properties are often effectively used as an indication of anticipated engineering behaviour and to assess general characteristics of soil and rock. For example, high plasticity clay is often identified based on measurements of Atterberg limits and, as a first approximation, may indicate significant potential for low strength, low hydraulic conductivity, and high swell potential. If such behaviour may substantially impact a project, the behaviour should be confirmed using more rigorous "performance" tests described in subsequent chapters of this manual.

However, identification of potential issues based on relatively simple tests provides significant benefit for geotechnical design.

Soil and rock classification is also used to help select samples for engineering property testing and to assess general variability and consistency among samples collected from a given site. In this context, the presumption is that soil and rock samples with similar classification are expected to behave similarly, again as a first approximation.

Thus, index property measurements and classifications can be used to establish whether samples collected from one boring are likely to be similar to samples collected from another and whether the samples can be considered to be from a single stratigraphic unit. In essence, index properties and classifications can be considered as "screening" tests that will often motivate additional investigations or measurements and facilitate grouping of different samples for design.

- **Boring and sampling requirements for index testing:**

Most index property measurements are insensitive to sample quality, so special boring and sampling procedures or equipment are seldom required. Samples for specific tests may require some care, but generally these requirements are easily satisfied. For example, it is important to prevent wetting or drying of samples acquired for measuring water content.

Similarly, samples acquired for measuring unit weight should be relatively undisturbed. Measurements for rock core also require care during coring and handling to prevent artificially breaking the rock core, which can bias index property measurements for rock. However, most index property tests can be performed on "bulk" or disturbed samples.

1.3 FUNDAMENTAL CONCEPTS FOR IDENTIFICATION AND CLASSIFICATION OF SOIL

The objective of identification and classification is to group soil or rock types that are expected to behave similarly. In this regard, it is important to distinguish between "coarse-grained" soils and "fine-grained" soils because their engineering behaviour is different and controlled by different factors.

Table 1.1 summarizes the most important compositional and "state" variables that influence the mechanical behaviour of coarse-grained and fine-grained soils.

Table 1.1: Soil characteristics affecting behaviour of coarse and fine-grained soils.

(Source- GEC5 – Geotechnical Site Characterization, Federal Highway Administration (FHWA))

Coarse-Grained Soils		**Fine-Grained Soils**	
Parameter Type	**Parameter**	**Parameter Type**	**Parameter**
Composition	Mean Grain Size Grain-Size Distribution Grain Mineralogy Grain Shape/Angularity Other Constituents (Carbonates, etc.)	Composition	Clay Size Fraction Clay Mineralogy Specific Surface Area Cation Exchange Other Constituents (Carbonates, etc.)
State	Void Ratio/Relative Density Confining Stress Stress History	State	Water Content Void Ratio Confining Stress Stress History

Coarse-grained soils, such as sands and gravels, are assemblages of individual particles with collective behaviour that depends on confinement, stress conditions, cementation, and particle packing. Since coarse-grained soils behave as particulate materials, characteristics of the individual particles and the collection of particles strongly influence mechanical behaviour.

The most important of these characteristics include mean grain size, grain-size distribution, grain shape, and grain hardness. Stress history also influences the behaviour of course-grained soils, although not to the degree observed for fine-grained soils.

In contrast, fine-grained soils are predominantly composed of small particles with large surface area. Silt particles are similar to sand particles, but much smaller. Silt particles are electrochemically neutral and are sometimes referred to as "surface dead" as they have no inherent attraction for other soil particles.

1.3.1 Grain-size distribution

Grain-size distribution refers to the proportion (by dry mass) of soil particles of different sizes within a soil sample. Grain-size distribution is commonly measured using mechanical sieves and/or hydrometer tests and is used to distinguish between fine- and coarse-grained soils, as well as to further classify coarse-grained soils.

Table 1.2 summarizes common criteria for characterizing different particle sizes along with descriptive terms used to refer to particles of different sizes.

Table 1.2: Descriptive terms for soil particle size ranges (Source- GEC5 – Geotechnical Site Characterization, Federal Highway Administration (FHWA))

Soil	Particle Size Ranges	Descriptive Term
Coarse-Grained	>12 in. (305 mm)	Boulders
	3 in. – 12 in. (75 mm – 305 mm)	Cobbles
	¾ in. – 3 in. (19 mm – 75 mm)	Coarse Gravel
	No. 4 Sieve – ¾ in. (4.75 mm – 19 mm)	Fine Gravel
	No. 10 – No 4 Sieve (2.00 mm – 4.75 mm)	Coarse Sand
	No. 40 – No. 10 Sieve (0.0425 mm – 2.00 mm)	Medium Sand
	No. 200 – No. 40 Sieve (0.075 mm – 0.0425 mm)	Fine Sand
Fine-Grained	0.075 mm – 0.002 mm	Silt
	< 0.002 mm	Clay

The distinction between coarse-grained soils and fine-grained soils is generally based on the proportion of soil retained on a No. 200 sieve, which has an opening size of 0.074 mm.

For the Unified Soil Classification System (USCS), soils having greater than 50 percent of particles (by mass) retained on the No. 200 sieve are considered to be coarse-grained. Soils where greater than 50 percent of particles by mass are finer than, or pass through, the No. 200 sieve are considered to be fine-grained. For the AASHTO Soil Classification System, coarse-grained soils are those with less than 35 percent of the soil particles passing the No.200 sieve.

- **Coarse-Grained Soils**

The traditional technique for determining the grain-size composition of coarse-grained soils is by mechanical sieve analysis (AASHTO T88, ASTM D422, and ASTM D6913). Results from mechanical sieve analyses are expressed collectively using a grain-size distribution curve similar to that shown in Figure-1.

The curves shown represent the cumulative percentage of soil particles (by dry mass) of different sizes. Curves plotting to the right of others in the diagram represent soils with more coarse grains while curves plotting to the left indicate soils with greater percentages of fine-grained particles.

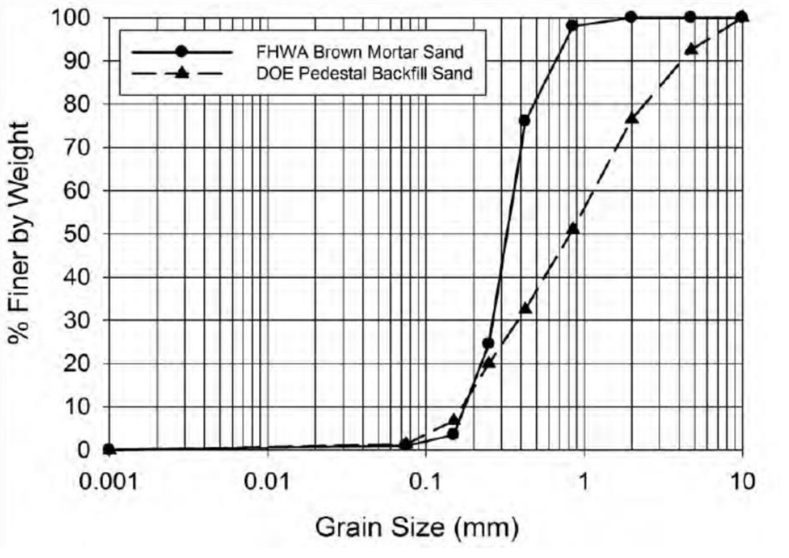


Figure 1.1: Grain-size distribution curves for two coarse-grained soils.

(Source- GEC5 – Geotechnical Site Characterization, Federal Highway Administration (FHWA))

Grain-size distribution curves can be used to establish several important quantitative measures that describe the shape and position of grain-size distribution curves and provide useful means for comparing different coarse-grained soils. The mean particle size, D_{50}, defined as the particle size (diameter) for which 50 percent of the soil particles are finer, serves as a measure of the relative position of different grain-size distribution curves. The "coefficient of uniformity", C_u, and "coefficient of curvature", C_c, are quantitative measures of the dispersion of particle sizes. The coefficient of uniformity is defined as

$$\mathbf{C = D_{60}/D_{10}}$$

Where, $\mathbf{D_{60}}$ is the grain size for which 60 percent of the soil particles are finer and $\mathbf{D_{10}}$ is the grain size for which 10 percent of the soil particles are finer. The coefficient of curvature, C_C, is defined as

$$\mathbf{C_C = (D_{30})^2/ (D_{10}.D_{60})}$$

Where, $\mathbf{D_{30}}$ is the grain size for which 30 percent of the soil particles are finer. Finally, the "percent fines" is often measured and reported as the percentage of the soil particles that pass through the No. 200 sieve, which includes the silt and clay fractions.

The coefficient of uniformity is used to help classify coarse-grained soils. The term "well-graded" is used to describe soils composed of a wide range of particle sizes while the term "poorly graded" is used to indicate that most of the particles fall within a narrow range. Table

1.3 gives criteria for identifying well-graded sands and gravels from results of sieve analyses for the USCS.

Coarse-grained soils not meeting both of the criteria shown are considered poorly graded. Table-4 summarizes values for the mean particle size, coefficient of uniformity, coefficient of curvature, percent fines, and descriptive gradation for the two particle size distributions in Figure 1.1

Table 1.3: USCS criteria for well-graded coarse-grained soils.

Soil	Grading	C_U	C_C
Sand	Well-Graded	$C_U \geq 6$	$1 \leq C_C \leq 3$
Gravel	Well-Graded	$C_U \geq 4$	$1 \leq C_C \leq 3$

Table 1.4: Grain-size characteristics for the two sands in Figure-1.1 (Source- GEC5 – Geotechnical Site Characterization, Federal Highway Administration (FHWA))

Soil	D_{50} (mm)	C_U	C_C	Fines (%)	Gradation
FHWA Sand	0.31	2.06	1.14	2.5	Poorly graded
DOE Sand	0.85	6.47	0.86	2.7	Poorly graded

Grain-size distribution can have a profound influence on the mechanical behaviour of coarse-grained soils. Well-graded soils are generally easier to compact and have higher strength and lower compressibility compared to poorly graded soils.

The wide range of particle sizes in a well-graded soil allows for tighter packing as the smaller grains fit into the void space between larger grains

1. Fine-Grained Soils

For most fine-grained soils, sieve analyses do not provide sufficient data to describe composition since the soils consist of smaller particles that cannot be separated by sieves. Grain-size distributions for fine-grained soils are therefore generally determined using the hydrometer test (AASHTO T88; ASTM D422).

The hydrometer test determines the proportion of silt- and clay-size particles using a sedimentation procedure and can be used to separate the fine-grained particle fraction into various sizes. It is sometimes useful to define not only the total silt content (percent between

0.075 mm and 0.002 mm) and clay content (percent < 0.002 mm) but also the coarse silt content (percent between 0.075 mm and 0.020 mm), fine silt content (percent between 0.020 mm and 0.002 mm), and fine clay content (percent < 0.001 mm, sometimes referred to as "colloids").

2. GRAIN SHAPE – COARSE-GRAINED SOILS

The shape of soil particles can exert a strong influence on the mechanical behaviour of coarse-grained soils. While the shapes of individual particles can be highly variable, it is useful to characterize particle shape as being rounded, angular, or an intermediate shape.

Figure 1.2 shows a comparison of different particle shape descriptions with illustrations showing the intended use. Generally, collections of angular soil particles produce more interlocking of particles that tends to create higher shear strength compared to rounded particles with the same degree of packing. Grain shape is not relevant for classification of fine-grained soils since individual particles cannot be distinguished with the naked eye.

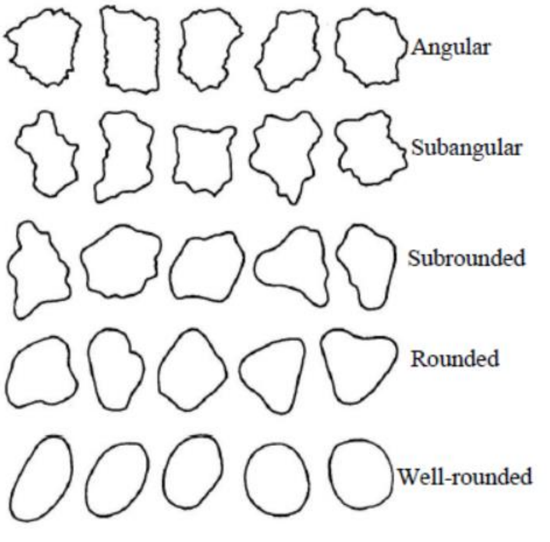


Figure 1.2: Particle shapes for coarse-grained soils (from Mitchell and Soga, 2005).

- Water content

The water content of soils is an important index property that is used to help interpret soil unit weight, relative consistency, and stress history, as well as to interpret groundwater levels. Water content, w, is generally expressed as a percentage and defined as-

$W = M_W/M_S$ = (Mass of water/ Mass of soil solids)*100%

Natural water contents, W_N, for sands typically range from 0 to 20 percent whereas natural water contents for inorganic and insensitive silts and clays generally range from 10 to 40 percent.

However, it is possible to have more water than solids so that water contents can exceed 100 percent depending upon mineralogy, formation environment, and structure. Soft and highly compressible clays, as well as sensitive, quick, or organic clays, can exhibit water contents of 40 to 300 percent, or more.

- **Unit weight and specific gravity**

The moist (total) mass density, ρt, of a soil or rock sample is given by

$$\rho_t = \frac{m_t}{V_t}$$

Where, mt is the total mass of the sample and Vt is the total volume of the sample. Dry mass density, ρd, is similarly given by-

$$\rho_d = \frac{m_s}{V_t}$$

Where, ms is the dry mass of the sample. The moist (total) unit weight, γt, and dry unit weight, γd, are similarly given by

$$\gamma_t = \frac{W_t}{V_t}$$

$$\gamma_d = \frac{W_s}{V_t}$$

where Wt and Ws are respectively the total and dry weight of the sample. The total and dry mass density and the total and dry unit weight are respectively related by the natural water content, wn, as

$$\rho_d = \frac{\rho_t}{1+w_n}$$

$$\gamma_d = \frac{\gamma_t}{1+w_n}$$

The terms density and unit weight are often used incorrectly and interchangeably. The correct usage is that density implies mass measurements while unit weight implies weight measurements. When the usage is independent of the specific definition, these terms will be referred to as “density (unit weight)”.

The specific gravity, Gs, of soil or rock solids is a measure of the density of the solid mineral particles referenced to the density of water. Specific gravity is computed as

$$G_s = \frac{m_s}{V_s \cdot \gamma_w}$$

Where, ms is the mass of soil or rock solids, Vs is the volume of soil or rock solids, and γw is the unit weight of water.Typical values of specific gravity for most soils lie within the narrow range of Gs=2.7±0.1. Exceptions to typical values occur for soil with appreciable organics (e.g., peat), ores and mine tailings, and soil or rock with high calcium carbonate content.

- **Atterberg limits**

At a very high water-content, a disturbed mixture of soil and water behaves as a viscous liquid. As the water content is reduced, the mixture takes on characteristics of a semi-solid and, finally, at a sufficiently low water content, the mixture behaves as a solid. The water contents where these changes in behaviour occur are called Atterberg limits, after the Swedish soil scientist A. Atterberg.

The "liquid limit", LL or wL, is the water content where the disturbed soil transitions from liquid to plastic behaviour. The "plastic limit", PL or wP, is the water content at the transition between the plastic and semisolid states of a soil. Finally, the "shrinkage limit", SL, is the water content corresponding to the transition between the semisolid and solid states of the soil.

Figure 1.3 illustrates the change in volume associated with changes in water content at the various Atterberg limits. Conceptually, the volume decreases linearly from the liquid limit through the plastic limit to the shrinkage limit as the water content decreases. At the shrinkage limit, the volume of the soil becomes constant and further drying produces no further reduction in volume.

Atterberg limits provide a relative indication of the ability for a silt or clay to retain water without changing state from a semi-solid to a viscous liquid. Atterberg limits can also provide an indication of the relative stiffness of soil by comparing the natural water content to the liquid and plastic limits; soils with water contents near the liquid limit can be expected to be soft while soils with water contents near the plastic limit can be expected to be much stiffer.

Finally, Atterberg limits serve as the primary basis for classification of fine-grained soils, as described in Section 4.16.

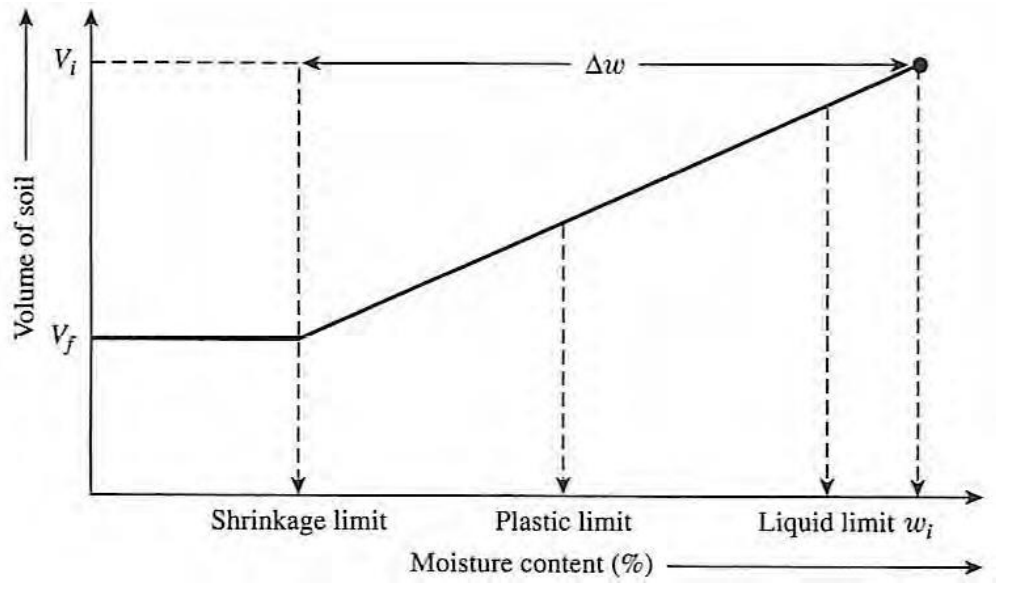


Figure 1.3: Idealized relation between volume and water content of soil including Atterberg limits. (Source- civilseek.com/atterberg-limits/)

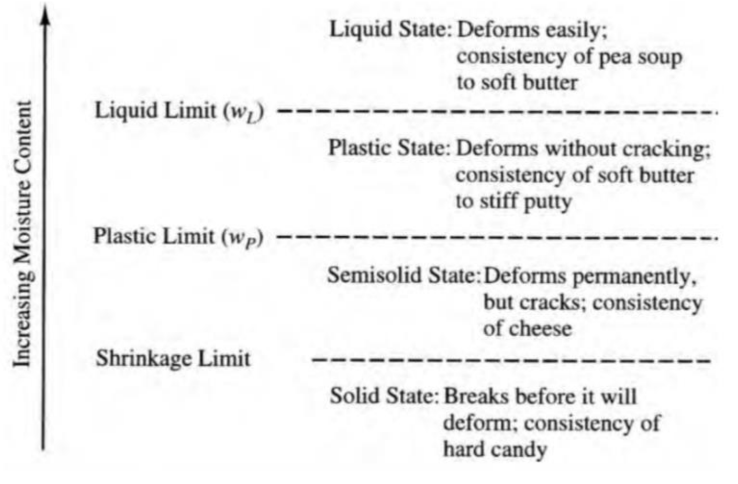


Figure 1.4: Conceptual model of Atterberg limits (from Coduto, 2001).

1.3.2 Shear properties of soil

In order to determine the shear properties of soil depending upon the soil drainage and loading conditions, following tests are performed in the laboratory-

i) Unconfined compression test, (UC)

ii) Consolidated Drained triaxial test, (CD) and,

iii) Direct shear test (DS).

1.4 STANDARD PENETRATION TEST

The standard penetration test (SPT) was developed 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 cohesion-less 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.

- **The Percussion Drilling Rig**

The machine used for making boreholes commonly is called a drilling rig. This machine is power driven by gasoline or diesel or compressed air or electric. 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.

- **Split-spoon Sampler**

The standard sampling tube for obtaining samples from the soil during a standard penetration test is the split spoon sampler. The assembly of the split spoon sampler consists of a short tube with a cutting edge (cutting shoe) on one end and threads on the other (Fig.1.8)

A split tube threads the shoe to a head assembly, which is attached to the drill rod. When unscrewed from the shoe and head assembly, the split spoon sampler can actually be opened into two equal segments for visual inspection of the sample or for removing part of the sample for preservation or future analysis.

Split spoon samples 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. Such samples are usually regarded as disturbed samples. They are disturbed in the sense that the grain structure of particle arrangement of the soil is altered.

- **Hammer**

Drivage is accomplished by a trip hammer weighing 64 kg, falling from a distance of 760 mm onto the drive head, which is fitted at the top of the rods. 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.

- **Drill Rods**

A rod enclosed in a tube or sleeve 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.

1.4.1 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.

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 word to release the hammer. Also, different types of anvils and rod lengths are prevalent.

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 the data become 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 a U2 hammer to its full length of 45mm or otherwise penetration refusal. The tube is then pulled to the surface, removed from the sampling hammer, and labelled 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 (Shelby-tube samples or split-spoon samples) are likewise properly labelled 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.

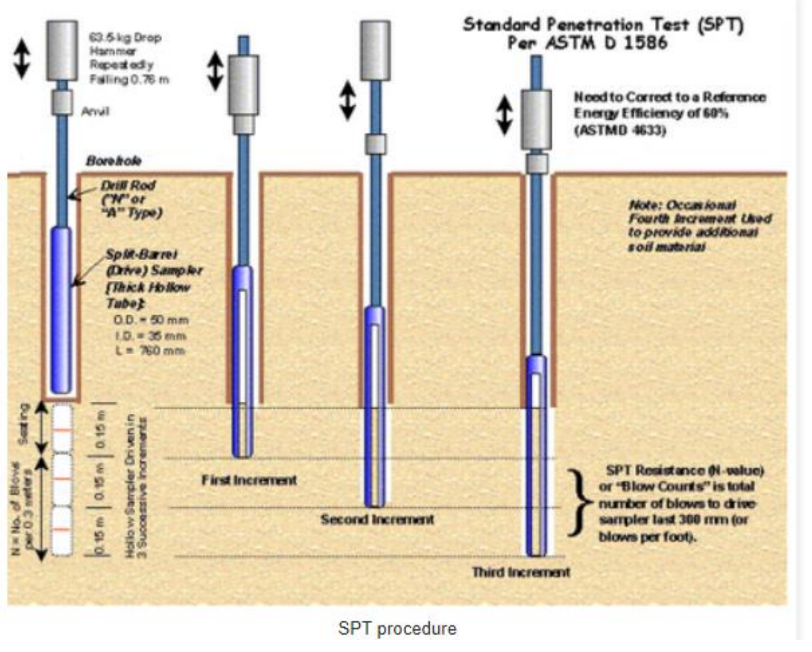


Figure 1.6: SPT procedure (Source- http://foundationeng.blogspot.com/2015/07/243-standard-penetration-test-astm-d1586.html)

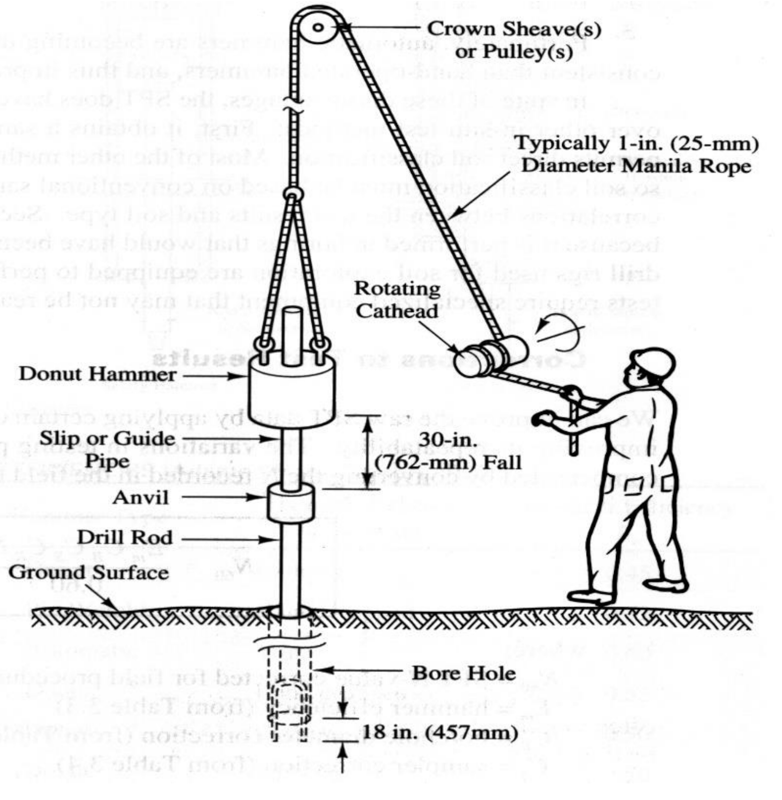


Figure 1.7:SPT procedure (Source- http://foundationeng.blogspot.com/2015/07/243-standard-penetration-test-astm-d1586.html)

1.4.2 Typical internal designs of safety hammers

As Per ASTM D 1586-11, "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils," sampler dimensions and test parameters for the SPT must be as follows:

- **Sampler inside tube diameter** =1.5 in. (3.81 cm)
- **Sampler outside tube diameter** =2.0 in. (5.08 cm)

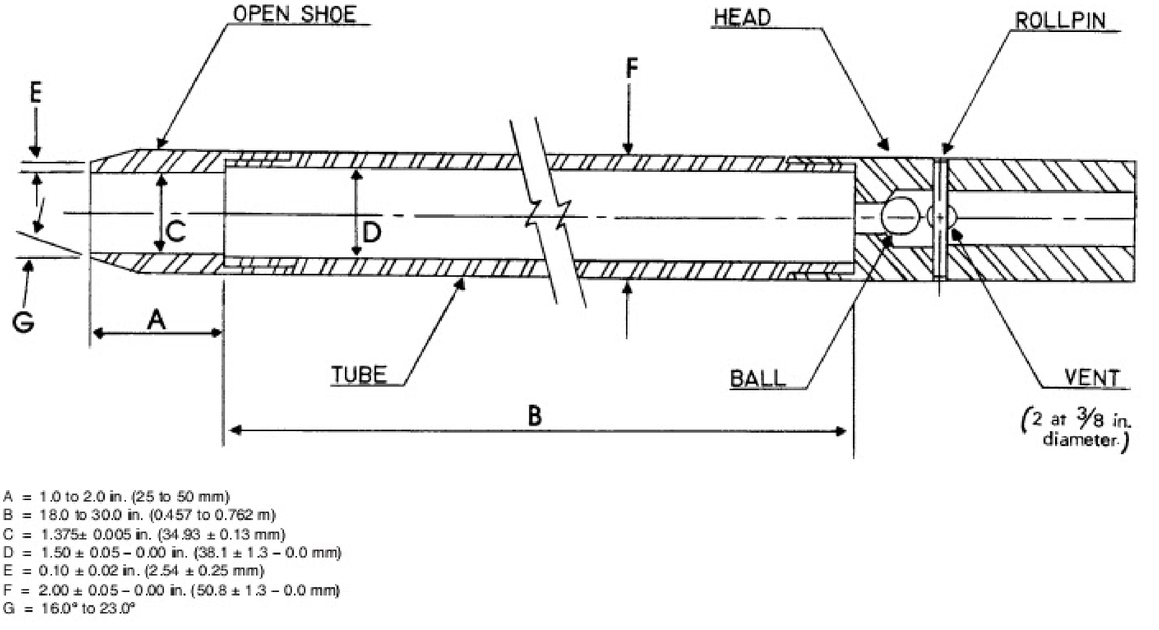


FIG. 2 Split-Barrel Sampler

Fig 1.8: Split Barrel Sampler (Sources- ASTM D 6066-96 (2004))

The corrections of the observed SPT-N values are done as per IS 1893 2016 in chapter-3.

1.5 FORMATION OF GEOTECHNICAL UNITS

From the field as well as laboratory test data on the samples collected from the different boreholes and based on the soil parameters hey possess, the subsoil is generalized into the following geotechnical units-

Geotechnical Units	Observed SPT-N value	Cohesion (g/cm^2)	Angle of internal friction (ϕ)	Liquid Limit	Plastic Limit
I	>20	0	25-40	-	NP
II	12-20	0.0-0.4	5-25	30-40	20-22
III	<12	>0.4	0-5	>40	>22

Table-1.5: Geotechnical units

1.6 ORGANISATION OF THESIS

Chapter one of the thesis gives a brief introduction of the study and depicts the importance of the study of correlations between Standard Penetration Test N-values and various other parameters like unit cohesion, internal frictional angle etc. The need for soil investigation and analysis for formation of various geotechnical structures is also studied in the current chapter.

Review of literature is done in chapter two and it consists of the various works done by the previous researchers to establish correlations between SPT N-values, angle of internal friction and unit cohesion. The equations and the regression analysis given by them and the methods undertaken are briefly discussed in the upcoming chapter.

In Chapter three, the objective of the study is described along with the detailed methodology undertaken to derive correlations among the soil parameters. The analysis is done and the results are presented in the next chapter.

Results from the analysis are presented in chapter four. The various correlations obtained in this study are mentioned in this chapter along with the significance of each correlation and its application in field is described.

Chapter five deals with the summary of the whole study along with the findings of the study. The scope of the study is also mentioned along with the limitation of the study which is of utmost importance.

The last chapter of the report is followed by the reference section.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

In order to interpret the results of the Standard Penetration Test, a number of research workers has developed correlation charts and correlation tables to determine the mechanical properties of soils and to design foundations.

From the correlation chart and correlation table, the allowable bearing capacity of the soil can be estimated. The number of blows can also be related to the allowable bearing pressure - the coarser or harder the material, the higher the number of blows needed to be able to penetrate the soil in question.

2.2 CORRELATIONS BETWEEN SPT N VALUES AND DIFFERENT PARAMETERS OF SOIL

The SPT has been used to correlate different soil parameters i.e., unit weight (γ), relative density (Dr), angle of internal friction (φ) and undrained compressive strength (qu). It has also been used to estimate the bearing capacity of foundations and for estimating the stress-strain modulus (Es).

Terzaghi and Peck gave the following correlation between SPT value and other soil parameters.

Table 2.1: Penetration Resistance and Soil Properties on the Basis of SPT (Cohesionless Soil: Fairly reliable) (Peck et. al. 1974; Bowles, 1977)

SPT N-value		0 to 4	4 to 10	10 to 30	30 to 50	>50
Compactness		very loose	loose	medium	dense	very dense
Relative Density, D_r (%)		0 to 15	15 to 35	35 to 65	65 to 85	85 to 100
Angle of Internal Friction, φ(°)		<28	28 to 30	30 to 36	36 to 41	>41
Unit Weight (moist)	pcf	<100	95 to 125	110 to 130	110 to 140	>130
	kN/m^3	<15.7	14.9 to 19.6	17.3 to 20.4	17.3 to 22.0	>20.4
Submerged unit weight	pcf	<60	55 to 65	60 to 70	65 to 85	>75
	kN/m^3	<9.4	8.6-10.2	9.4 to 11.0	10.5 to 13.4	>11.8

Table 2.2: Penetration Resistance and Soil Properties on the Basis of SPT (Cohesive Soil: rather unreliable) (Peck et. al. 1974; Bowles, 1977)

SPT N-value		0 to 2	2 to 4	4 to 8	8 to 16	16 to 32	>32
Consistency		very soft	soft	medium	stiff	very stiff	hard
Unconfined Comp. Test	lb/ft^2	0 to 250	250 to 500	500 to 1000	1000 to 2000	2000 to 4000	>4000
	kPa	0 to 25	25 to 50	50 to 100	100 to 200	200 to 400	>400
Unit Weight (Saturated)	pcf	<100	100 to120	110 to 125	115 to130	120 to 140	>130
	kN/m^3	<15.7	15.7 to 18.8	17.3 to 19.6	18.1 to 20.4	18.8 to 22.0	>20.4

A number of research workers gave various correlation of SPT-N value and angle of internal friction of cohesionless soil.Some of them are listed in table 2.3

Table 2.3: Correlations between N-value and angle of internal friction for cohesion-less soils

Soil Type	ϕ°	References
Sandy soil	0.3N+27	Peck et. al. (1953)
Angular and well-graded soil particles Round and well-graded or angular and uniform-graded soil particles Round and uniform-graded soil particles	$(12N)^{0.5}+25$ $(12N)^{0.5}+20$ $(12N)^{0.5}+15$	Dunham (1954)
Sandy soil	$(20N)^{0.5}+15$	Osaki et al. (1959)
Granular soil	$27.1+0.3N_{60}+0.00054(N_{60})^2$	Peck et al. (1974) Wolff (1989)
Sandy soil	$(15N)^{0.5}+15\leq45$ (N>5)	Japan Road Association (1990)
Cohesionless soil	$\phi = \tan^{-1}\left[\frac{N_{60}}{12.2+20.3\left(\frac{\sigma_0'}{p_a}\right)}\right]^{0.34}$ Where Pa= atmospheric pressure in the same unit as σ_0'	Schmertmann (1975) Kuhawy and Mayne (1990)

Meyerhoff (1956) proposed that SPT N-value in exploratory borings gives a qualitative guide to the in-situ engineering properties and provides an indication of the relative density and friction angle of the soil. He provided the relationships between SPT-N value with relative density and frictional angle.

Table 2.4: SPT N-value versus friction angle and relative density (Meyerhoff, 1956)

SPT N [Blows/0.3 m]	Soil packing	Relative Density [%]	Friction angle [°]
< 4	Very loose	< 20	< 30
4 -10	Loose	20 - 40	30 - 35
10 - 30	Compact	40 - 60	35 - 40
30 - 50	Dense	60 - 80	40 - 45
> 50	Very Dense	> 80	> 45

As per IS-6403:1981, a relationship has been established between SPT-N value and angle of internal friction which can be shown by the chart below.

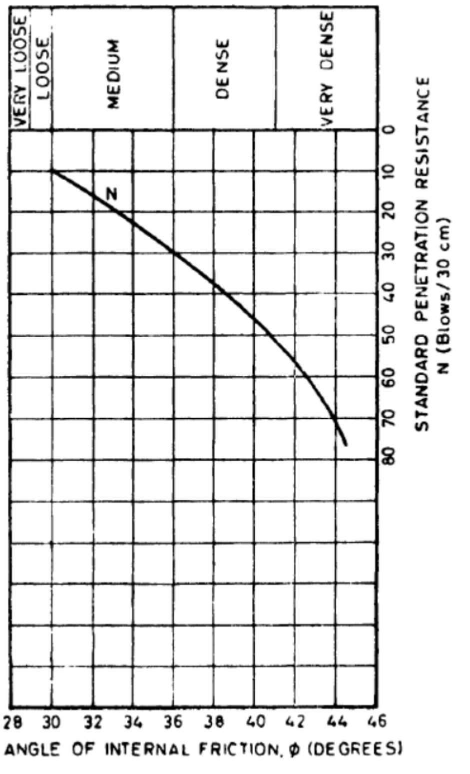


Figure 2.1:Relationship between SPT-N and angle of internal friction (Source- IS 6403:1981)

Many studies on empirical relationships have been done in the past on different soil types. Empirical relations were developed between cohesion and SPT N value, and between angle of friction and SPT N value (Brown and Hettiarachchi 2008; Hettiarachchi and Brown 2009).

Empirical correlations were developed between angle of friction and SPT N value by Suzuki et al. (1993) and Hatanaka and Uchida (1996). Correlations between undrained shear strength and SPT N value were developed by Hara et al. (1974); Sivrikaya and Togrol (2006) and Kalantary et al. (2009)

2.3 ESTIMATION OF ENGINEERING PROPERTIES OF SOILS FROM FIELD SPT USING RANDOM NUMBER

Kumar et al.2016 estimated properties of soil from field SPT using Random Number Generation procedure. Data are generated through random number generation technique. LHS technique (Mckay et al. 1979) was adopted as this is an inexpensive way as compared to laboratory testing. Upper and lower limits of these random variables were known and it was assumed that mean and standard deviation of these random variables were not available, hence uniform distribution is adopted.

2.3.1 Development of correlation between cohesion and SPT-N value

Table 2.5: Ranges of SPT N value with cohesion for cohesive soils by Karol (1960)

SPT *N* value	>30	15–30	8–15	4–8	2–4	<2
Cohesion, kPa	192	96–192	48–96	24–48	12–24	12
Soil conditions	Hard	Very stiff	Stiff	Firm	Soft	Very soft

Table 2.6: Ranges of SPT N value with Cohesion for intermediate soils by Karol (1960)

SPT *N* value	>30	10–30	<10
Cohesion, kPa	48	5–48	5
Soil conditions	Dense	Medium	Loose

Correlation between cohesion of soil and SPT N value has been given by Karol (1960) along with soil conditions representing various ranges of cohesion as given in Table-2.5.

Ranges of angle of friction of soil with SPT N value has been given by Terzaghi and Peck (1967) along with soil conditions representing various ranges of cohesion as shown in Table 2.6.

It was observed from Tables 2.5 and 2.6 that, four and one ranges of values were available for both the parameters respectively. Here, fifty and three hundred random numbers were generated for each range in Tables 2.5 and 2.6 respectively and the data were arranged in ascending order in each range.

For cohesive and intermediate soils, best fit curve was obtained by using Curve Expert 1.37 (Daniel 2001). The best fit curve for cohesion of soil vs. SPT N value for cohesive soils with R^2 as 0.998 is represented by following equation.

$$\mathbf{c = -2.2049+6.484N \ (R^2 = 0.998)} \qquad \mathbf{(2.1)}$$

where, c is cohesion, kPa; N SPT N value (range 2–30)

The best fit curve for intermediate soils with R^2 as 0.998 is represented by following equation.

$$\mathbf{c = -16.5+2.15N \ (R^2 = 0.998)} \qquad \mathbf{(2.2)}$$

where, c is cohesion, kPa; N SPT N value (range 10–30)

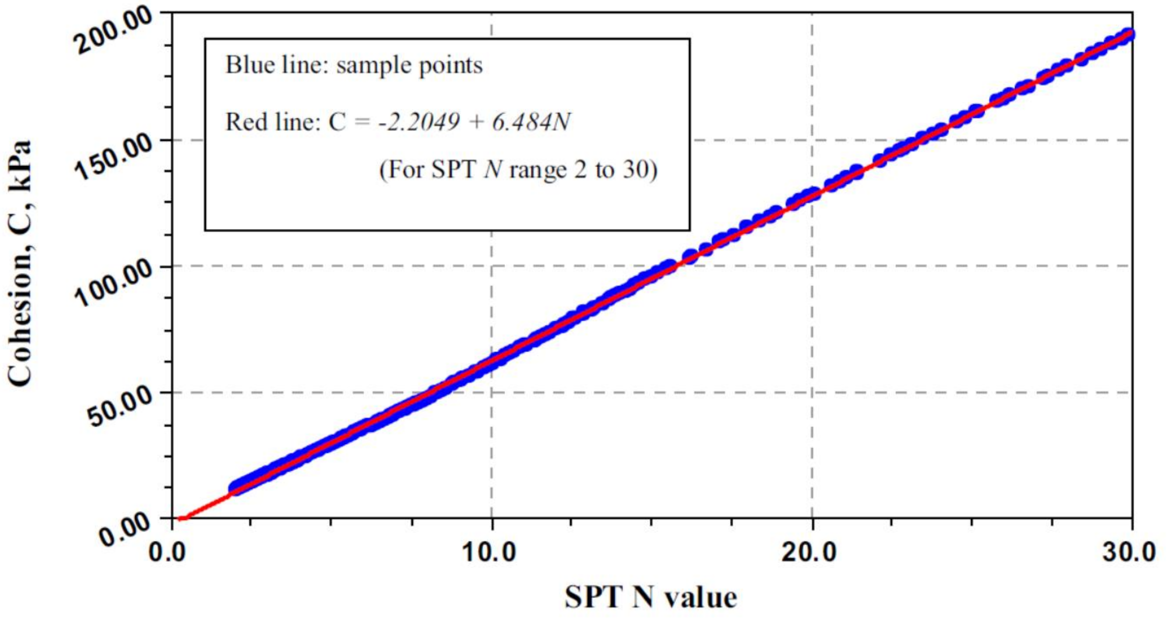


Figure 2.2: Plot of 200 pair of data points of SPT N and cohesion for cohesive soils. (Source-Kumar et.al 2016)

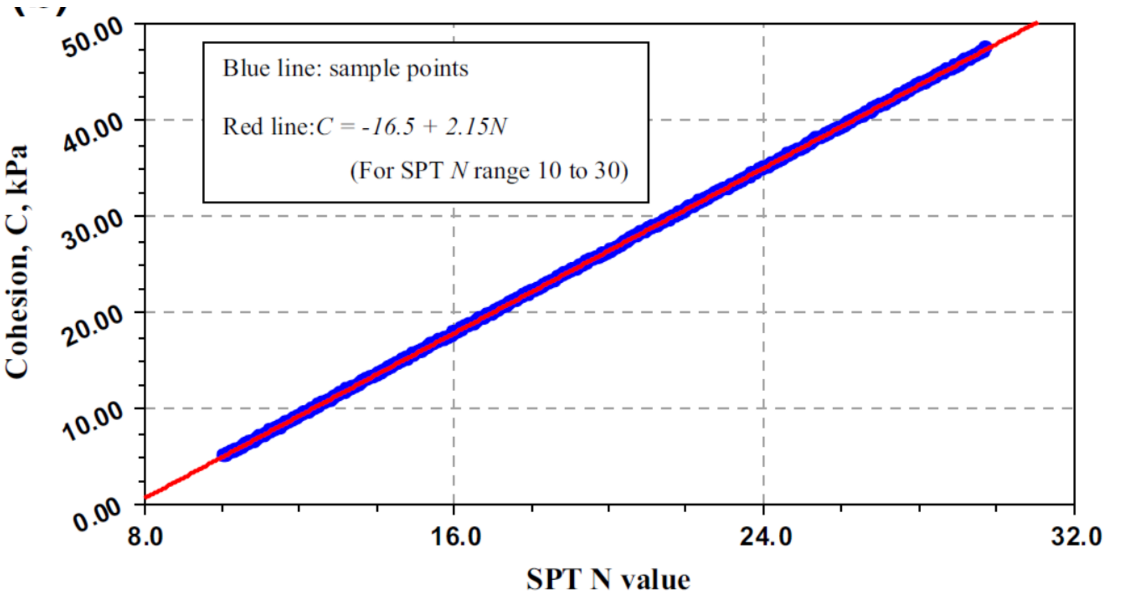


Figure 2.3: Plot of 300 pair of data points of SPT N and cohesion for intermediate soils (Source-Kumar et.al 2016)

2.3.2 Development of correlation between angle of friction and SPT N value

Table 2.7: Ranges of SPT N value with angle of friction, data from Terzaghi and Peck (1967)

SPT N value	>50	30–50	10–30	4–10	0–4
Angle of friction, degree	>41	36–41	30–36	28–30	<28
Soil conditions	Very good	Good	Fair	Poor	Very poor

Ranges of angle of friction of soil with SPT N value has been given by Terzaghi and Peck (1967) along with soil conditions representing various ranges of cohesion as shown in Table 2.7 Initial four ranges were selected from Table 2.7 for development of correlation. It was observed from Table 2.7 that there was continuation of ranges. In the four ranges, fifty random numbers were generated for each range. The generated random numbers were arranged in ascending order in each range. In the first range, minimum value of angle of friction was taken as zero degree. The two hundred increasingly ordered random numbers representing data points were plotted.

The best fit curve was obtained by using Curve Expert 1.37 (Daniel 2001). The best fit curve with R^2 as 0.998 is represented by following equation.

$$\phi = 7N \ (R^2 = 0.998); \text{ for } N \leq 4 \qquad (2.3)$$

$$\phi = 27.14 + 0.2857N \ (R^2 = 0.998); \text{ for } N = 4 \text{ to } 50 \qquad (2.4)$$

Where ϕ = angle of internal friction and N= SPT-N value

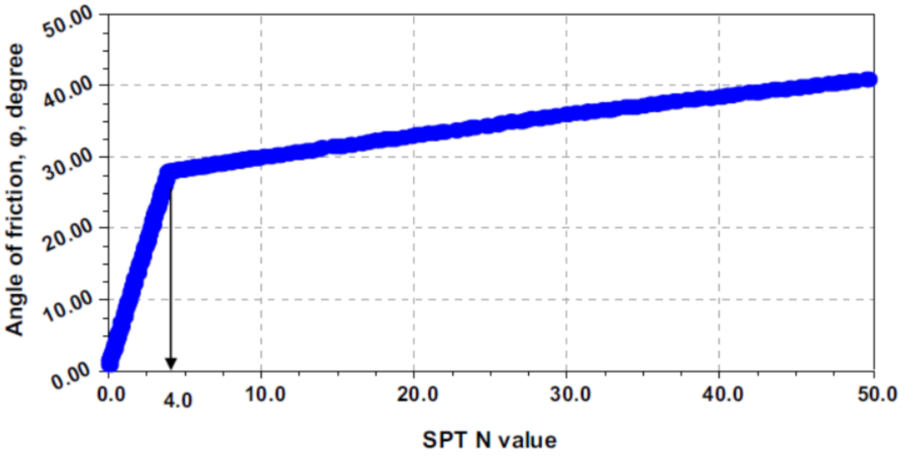


Figure 2.4: Plot of 200 pair of data points of SPT N value and angle of friction (Kumar et.al 2016)

In case of cohesion, typical values were available for two types of soils namely cohesive and intermediate soils. The ranges of values for both types of soils are totally different. Hence, two different relationships for cohesion for broadly two types of soils are proposed.

In case of angle of friction, even if typical values were available for soil, sudden change in the nature of plot of randomly generated data was observed. Due to this sudden change in plot, two different relationships for angle of friction were proposed for different ranges of SPT N value.

The results of regression analysis show maximum correlation coefficient and minimum standard error. The proposed relationships have been validated with the help of experimental data available in literature. The usefulness of random number generation technique is established for development of correlations.

2.4 PREDICTION OF ANGLE OF INTERNAL FRICTION BASED ON SPT N VALUES

Based on the SPT data, an empirical correlation has been established by Subhashree Dalai and Chittaranjan Patra (National Institute of Technology, Rourkela) between standard penetration number N and internal friction angle to predict the friction angle of soils. All the field standard penetration test data were collected from six different places of East India. Regression analyses were performed using the SPT data collected from 40 different boreholes containing 330 data points. The SPT-N values obtained from different sites are observed to vary between 4 and 70.

The in-situ bulk density of undisturbed samples recovered through pitcher sampler was in the range of 17.90 kN/m3-18.90 kN/m3. In situ water content and fines content observed plays an important role in case of c-ϕ soil, hence plasticity index (PI) and fines content (p) were also included in model equation in case of c-ϕ soil.

The predicted results obtained from developed model equation appeared to be in good agreement with existing equations in various literature. By using regression analysis, the empirical equations were developed.

2.4.1 Linear regression analysis

The linear regression analysis is used as predictive model to an observed data set for predicting, forecasting and error reduction. In the study, linear regression model was used for correlation between angle of internal friction and SPT (N) value by involving the parameters plasticity index and percentage finer. To correlate the unconfined compressive strength, SPT (N) value and plasticity index, linear equation was derived. Linear regression model gives simple equation with greater accuracy. It can be used in field condition because the results are validated with experimental value of the response variable.

2.4.2 Non-Linear regression analysis (NLREG)

For cohesionless soils, NLREG is used as power function and polynomial function to estimate the equation between angle of friction and SPT (N) for different type of soils i.e. sandy and silty sand soils. In this analysis, coefficient of variation is above 0.80. It is acceptable if co-efficient of variation is greater than 0.8. Polynomial function is also known as multiple linear function. For c-ϕ soil, NLREG is used as power variation, multiple linear function and polynomial function.

2.4.3 Results and Analysis

- **Cohesionless soil**

The number of bore-hole used for collecting SPT (N) value was sixty and depth of each borehole was 30m to 100m. The effective grain size of sand ranged from 0.075mm to 0.42mm. Ground water table was at depth of 6.5m to 8.6m of borehole. SPT (N) value ranged from 4 to 100. NLREG analysis is carried out to find simple relation between angle of friction and SPT (N) value with a co-efficient of variation 0.802. The predicted values were compared with experimental values to estimate the variation which comes within ±10%. Then the predicted equation was validated with the equation given by Peck et al. It shows good similarity with the equation given by Wolff (1989).

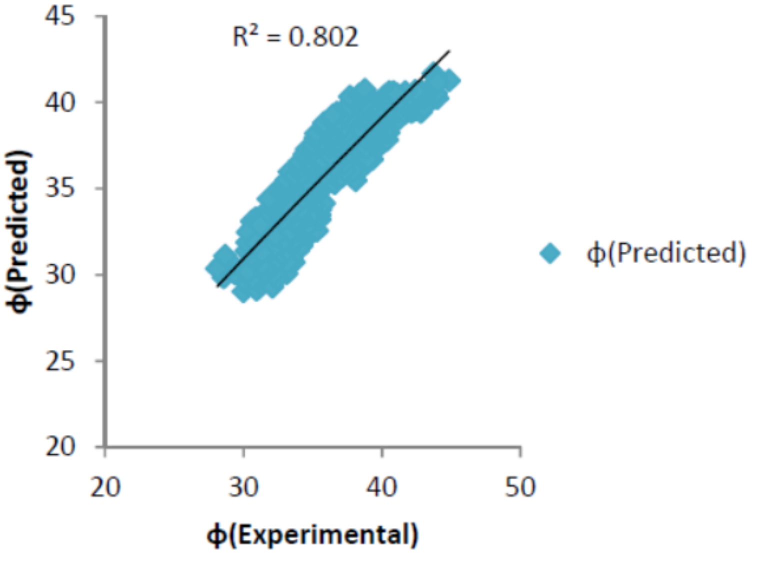


Figure 2.5: Variation between Predicted and experimental angle of friction (Source- Prediction of Angle of Internal Friction Based on SPT N Values by Subhashree Dalai and Chittaranjan Patra)

From the NLREG analysis, the predicted equation is-

$$\boldsymbol{\emptyset = 8.103 \times N^{0.458}} \qquad \textbf{(2.5)}$$

In the study, the corrections are not made for field SPT N value. It is directly used for prediction the equations. So, it gives difference. It is for cohesionless soils.

- **c-ϕ soil**

The geotechnical investigation was carried out on ash pond of NTPC at Kahalgaon. Bihar. Laboratory tests are carried out at ash silo, ESP unit area and chimney area. SPT was conducted with split spoon sampler to determine the properties of soil.

Tests were done at sites are unconfined compressive strength, direct shear test, triaxial shear test for all conditions, consolidation test, standard proctor compaction test and chemical test. Total 22 number of boreholes are sunk in different zones by using shell and auger boring.

Undisturbed soil samples were collected from the borehole and disturbed soil samples were collected from the split spoon sampler. The soil varied from medium stiff to very stiff silty clay with traces of kanker in ash silo zone. Dense to very dense yellowish grey silty sand soil was observed in ESP unit area in which SPT N ranges from 51to 56.

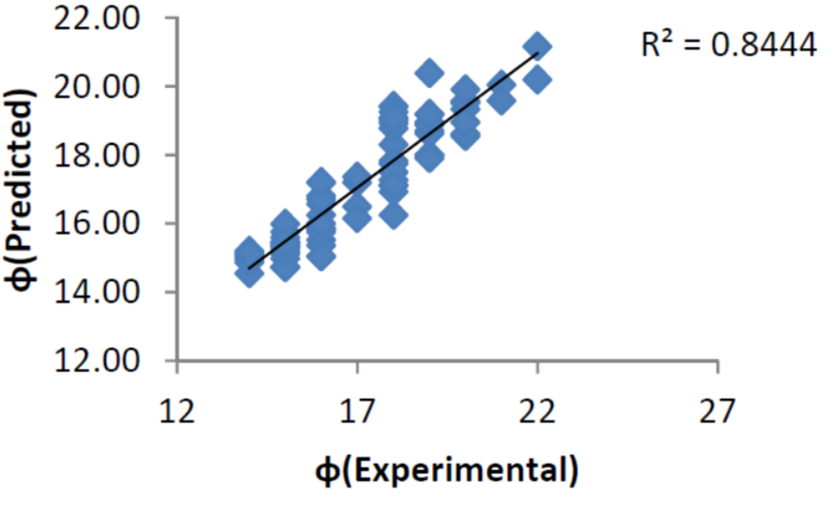


Figure 2.6: Variation between Predicted and experimental angle of friction of silty-clay for consolidated undrained case (Source- Prediction of Angle of Internal Friction Based on SPT N Values by Subhashree Dalai and Chittaranjan Patra)

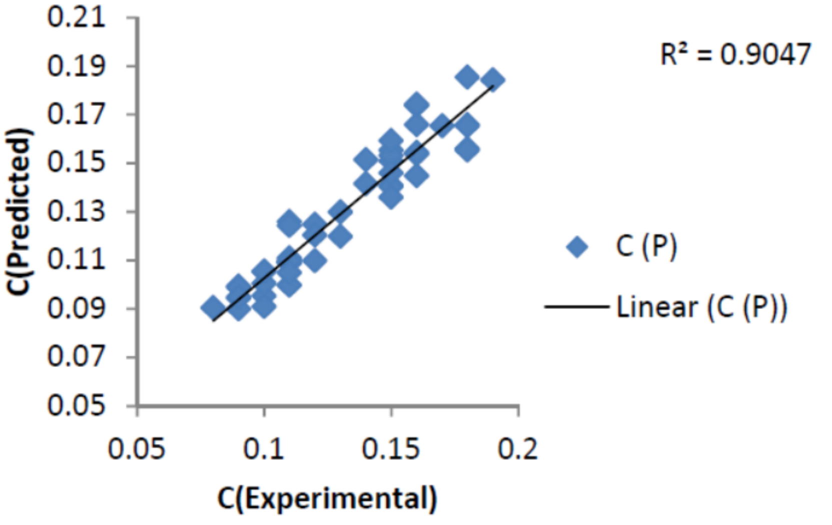


Figure 2.7: Variation between Predicted and experimental cohesion for over consolidated undrained case (Source- Prediction of Angle of Internal Friction Based on SPT N Values by Subhashree Dalai and Chittaranjan Patra)

The predicted equation for shear strength parameters were as follows-

$$\mathbf{c = 0.00054*Ip + 0.005*N + 0.09} \quad \textbf{(2.6)}$$

$$\mathbf{\emptyset = 0.24*N + 0.0061*Ip - 0.313*p + 43.42} \quad \textbf{(2.7)}$$

Where, Ip = plasticity index
p = fines content
N = SPT –N value
∅ = Angle of internal friction
c = unit cohesion in kn/m^2

2.5 EVALUATION OF SOIL CHARACTERISTICS FROM FIELD SPT VALUES USING RANDOM NUMBER GENERATION TECHNIQUE

The study was conducted by Mustafa Najdat Kasim and Aram Mohammed Raheem, a set of field data for Standard Penetration Test (SPT) values was collected from more than twenty different places in Kirkuk city. In addition, using the random number generation method, several empirical relationships of various soil properties were advanced in terms of the spectrum of the collected SPT values.

Latin Hypercube Sampling method (LHS) was implemented as a reasonable approach compared to the laboratory soil testing. Both Upper and lower bounds of these random data were known, and a uniform distribution was used in the absence of the mean and standard deviation.

2.5.1 Correlation between cohesion and SPT-N values

Karol (1960) proposed a correlation between the soil cohesion and SPT-N values based on the soil conditions that vary from very soft to hard conditions corresponding to SPT-N and cohesion values.

It was noticed that SPT-N values range from 2 to 30 and the cohesion varied from 5 kPa to 192 kPa with different soil conditions. Based on the used random number generation technique, around 300 data point was created to simulate the relation between the cohesion versus the SPT-N values.

A correlation was built between the random generated values of soil cohesion with their corresponding SPT-N values as follows:

$$\mathbf{c = 6.5808 * N - 9.079 \ (R^2 = 0.9942)} \qquad \mathbf{(2.8)}$$

Where: c is the cohesion (kPa), and N is SPT-values.

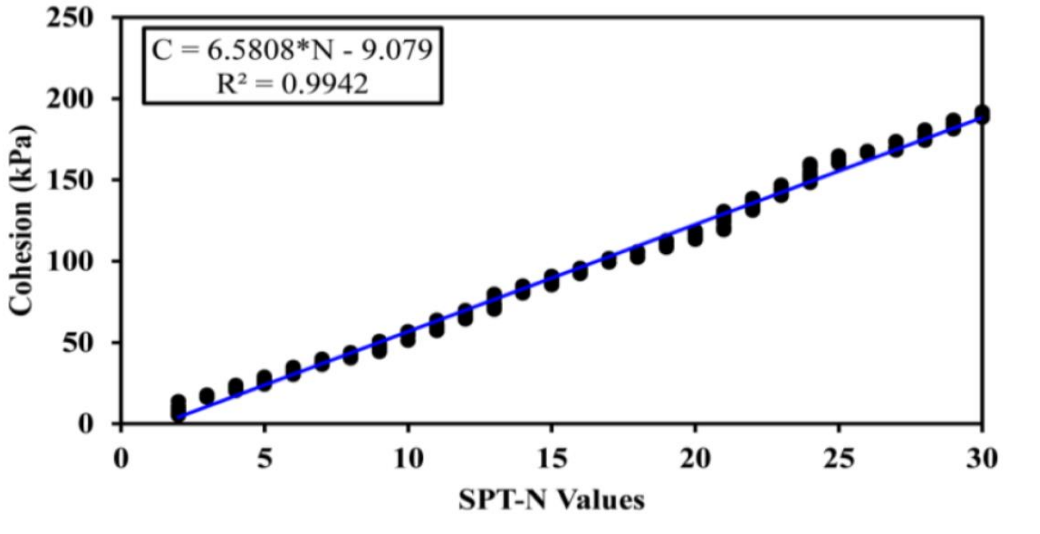


Figure 2.8: The random distribution of soil cohesion with SPT-N values.(Source- Evaluation of soil characteristics from field SPT values using random number generation technique by Mustafa Najdat Kasim and Aram Mohammed Raheem.)

2.5.2 Correlation between angle of internal friction and SPT-N values

Terzaghi and Peck (1967) have given a wide range of soil angle of internal friction with SPT-N values for various soil conditions.

The SPT-N values range from 0 to 50 and the angle of internal friction varies from 0 ˚ to 41˚ with different soil conditions. Based on the used random number generation technique, around 300 data point were created to simulate the relation between the angles of internal friction versus the SPT-N values.

A correlation has been developed between the random generated values of soil angle of internal friction with their corresponding SPT-N values as follows:

$$\boldsymbol{\emptyset = 0.8531 * N + 0.1581 \ (R^2 = 0.9963)} \qquad \textbf{(2.9)}$$

Where: Ø in degree is the angle of internal friction, and N is SPT-values.

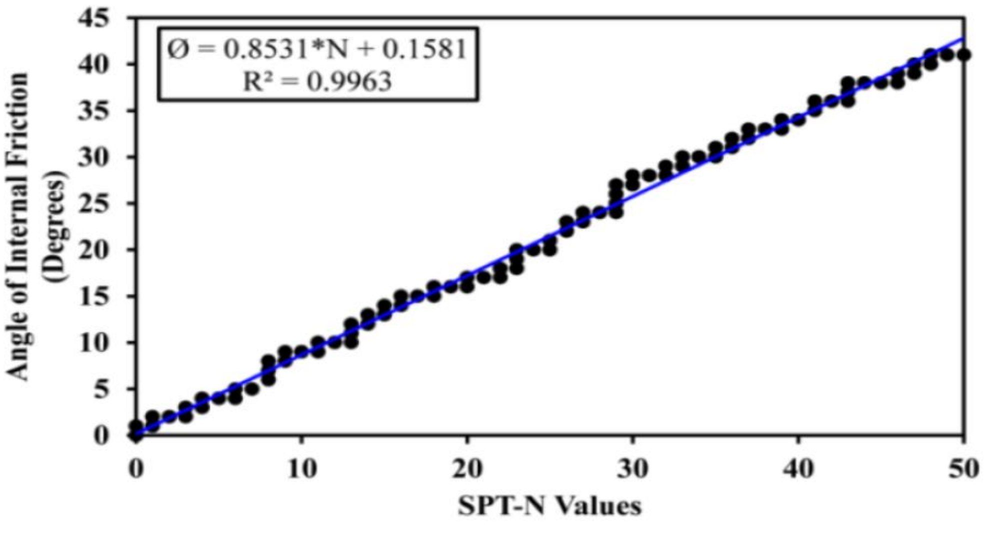


Figure 2.9: The random distribution of soil angles of internal friction with SPT-N values. (Source-Evaluation of soil characteristics from field SPT values using random number generation technique by Mustafa Najdat Kasim and Aram Mohammed Raheem)

The soil cohesion values were predicted depending on the SPT-N data for Kirkuk city. For the SPT-N values ranged from 4 to 44, the predicted soil cohesion increased from 17 (kPa) to 281 (kPa). It was clearly indicated that the places with low cohesion are mainly granular soil whereas the places with high cohesion are cohesive soil zones.

In a similar manner, the field SPT values were used to predict the expected angle of internal friction for Kirkuk soil. For the SPT-N values ranged from 6 to 44, the predicted angle of internal of friction increased from 4 ˚ to 38˚. It is clearly indicated that the places with low angle of internal friction were mainly cohesive soil whereas the places with high angle of internal of friction were granular soil zones.

2.6 STANDARD PENETRATION TEST IN PREDICTING PROPERTIES OF SILTY CLAY WITH SAND SOIL

Mostafa Abdou Abdel Naiem Mahmoud (2013) studied the reliability of using standard penetration test (SPT) in predicting some properties, (such as Atterberg limits LL, PL, PI, and shear strength parameters of silty clay with sand soil.

The site of this work was Tabarjal - Al-Jouf, KSA. The field work consisted of drilling and sampling of more than 100 boreholes to depths between 10 m to 15 m below ground surface.

A standard penetration test (SPT) was carried out according to ASTMD – 1586 – 84. A suit of in situ testing and sampling within the boreholes was planned including standard penetration test SPT.

The correlation coefficient (R^2), the best fitting between the results was plotted. The purpose of use of this statistical method was to give a statistic known as the correlation coefficient which was a summary value of a large set of data representing the degree of linear association between two measured variables. R^2 is a statistic that gives some information about the goodness of fit of a relationship. In regression, the R^2 coefficient of determination is a statistical measure of how well the regression line approximates the real data points (Taylor, 1990).

According to the values of R^2, the relationship between any two parameters can be classified as **(R^2 <0.30)** are considered to have **no correlation**, **(R^2 of 0.30 to 0.499**) are considered to be a **mild relationship**, (**R^2 of 0.50 to 0.699**) are considered to be a **moderate relationship** and, **(R^2 of 0.70 to 1.0)** are considered to be a **strong relationship**.

- **SPT versus shear strength parameters**

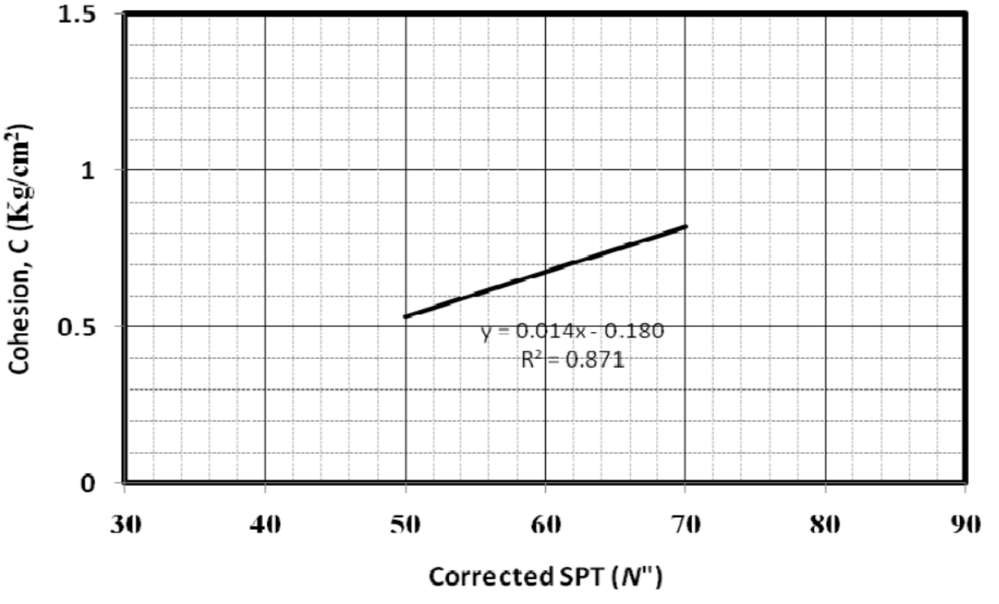


Figure 2.10: Corrected SPT number (N") versus cohesion (c)(Mahmoud 2013)

A correlation coefficient (R^2 = 0.871) can be an indication of a good correlation between corrected SPT number (N") and cohesion (c).

From the results and relationships and regression analysis, empirical equation to estimate the shear strength parameters (c) for silty clay with sand soil with the help of corrected SPT number (N") as follow:

$$c = 0.014\, N'' - 0.18 \qquad (2.10)$$

Where, (N") is the corrected SPT number, c is the cohesion in (Kg/cm2).

For identification of shear strength parameters of silty clay with sand soil, using SPT is adequate rather than using laboratory tests because SPT carries out in the field on undisturbed soil.

Hence a number of prominent research workers have been found to establish correlation between the SPT N value with cohesion and angle of internal friction. There contributions have highly helped in predicting the properties of soil in the field quite accurately.

CHAPTER 3

OBJECTIVE AND METHODOLOGY

Due to rapid industrialization and urbanization, geotechnical investigation reports play an important role in construction of all infrastructure projects in the developing countries. To determine the mechanical properties of soil strata, Standard Penetration Test (SPT) is most frequently used since it is a quick and inexpensive method.

From the SPT test, the sub soil characteristics, angle of shearing resistance can be determined in cohesion less soils and undrained shear strength in cohesive soils.

3.1 OBJECTIVE

To establish correlation between:

1. SPT-N corrected i.e $(N_1)_{60cs}$ and the unit cohesion (c) of soil for Guwahati City.
2. SPT-N corrected i.e $(N_1)_{60cs}$ and angle of internal friction of soil for Guwahati City.
3. SPT-N corrected i.e $(N_1)_{60cs}$, unit cohesion (c) and the angle of internal friction (ϕ) of soil for Guwahati City.

3.2 METHODOLOGY

The methodology of establishing the correlations between SPT-$(N_1)_{60cs}$, unit cohesion (c) and angle of internal friction (ϕ), can be divided into the following steps-

Let us take a Geotechnical Test report for a for the proposed Apartment Building (G+4) at Shreenagar, near State Zoo, Guwahati-5.

3.2.1 STEP 1: Obtaining SPT-N observed values

From the SPT test which was conducted at Borehole-1 and from the bore-log data, N-observed values for different depths of exploration is noted down. The SPT-N value is obtained at every 1.5 m depth from the ground surface. In this case, it is seen that the explored depth is about 25m from the ground level and the ground water is encountered at a depth of 0.5 m from the ground level.

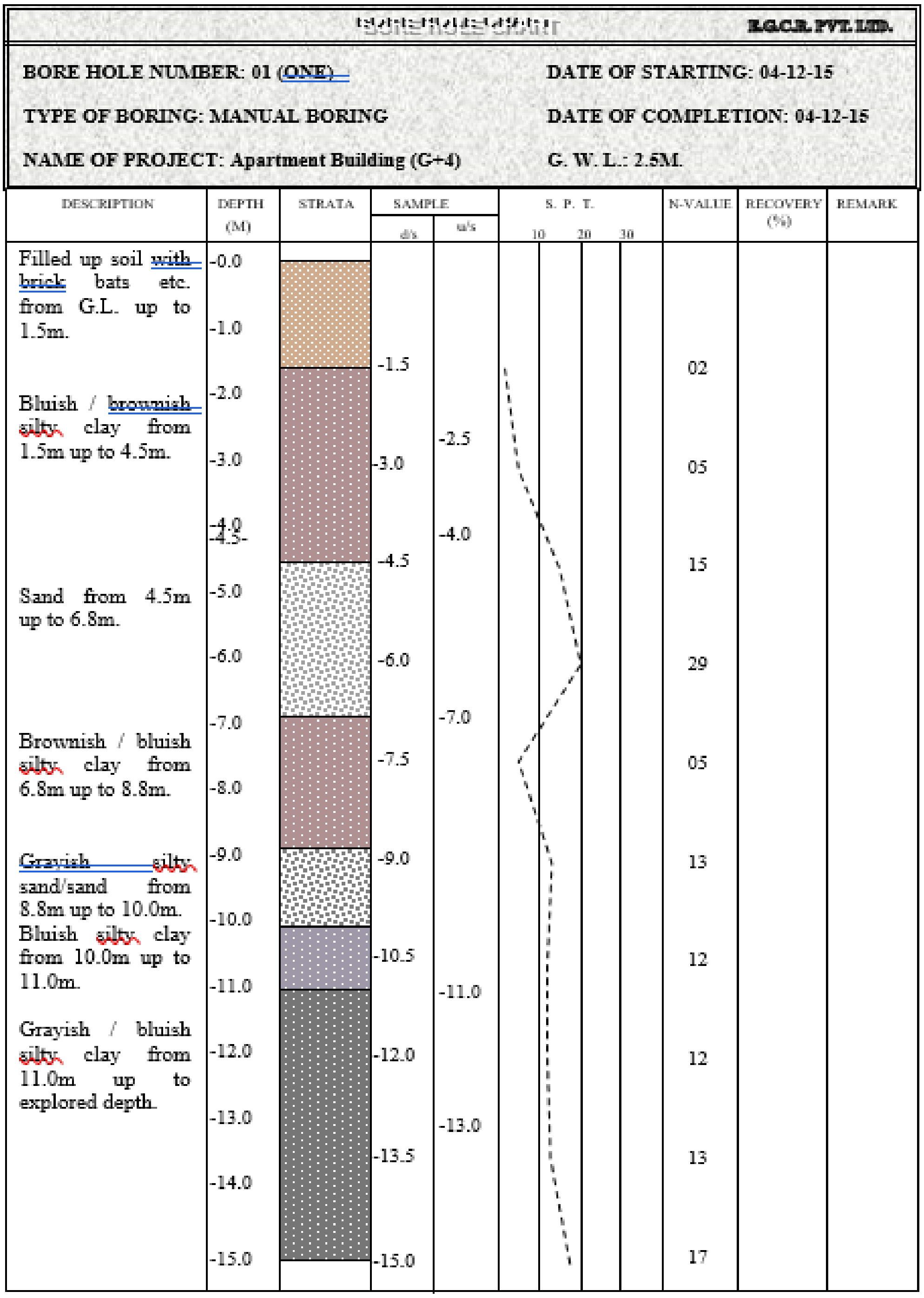
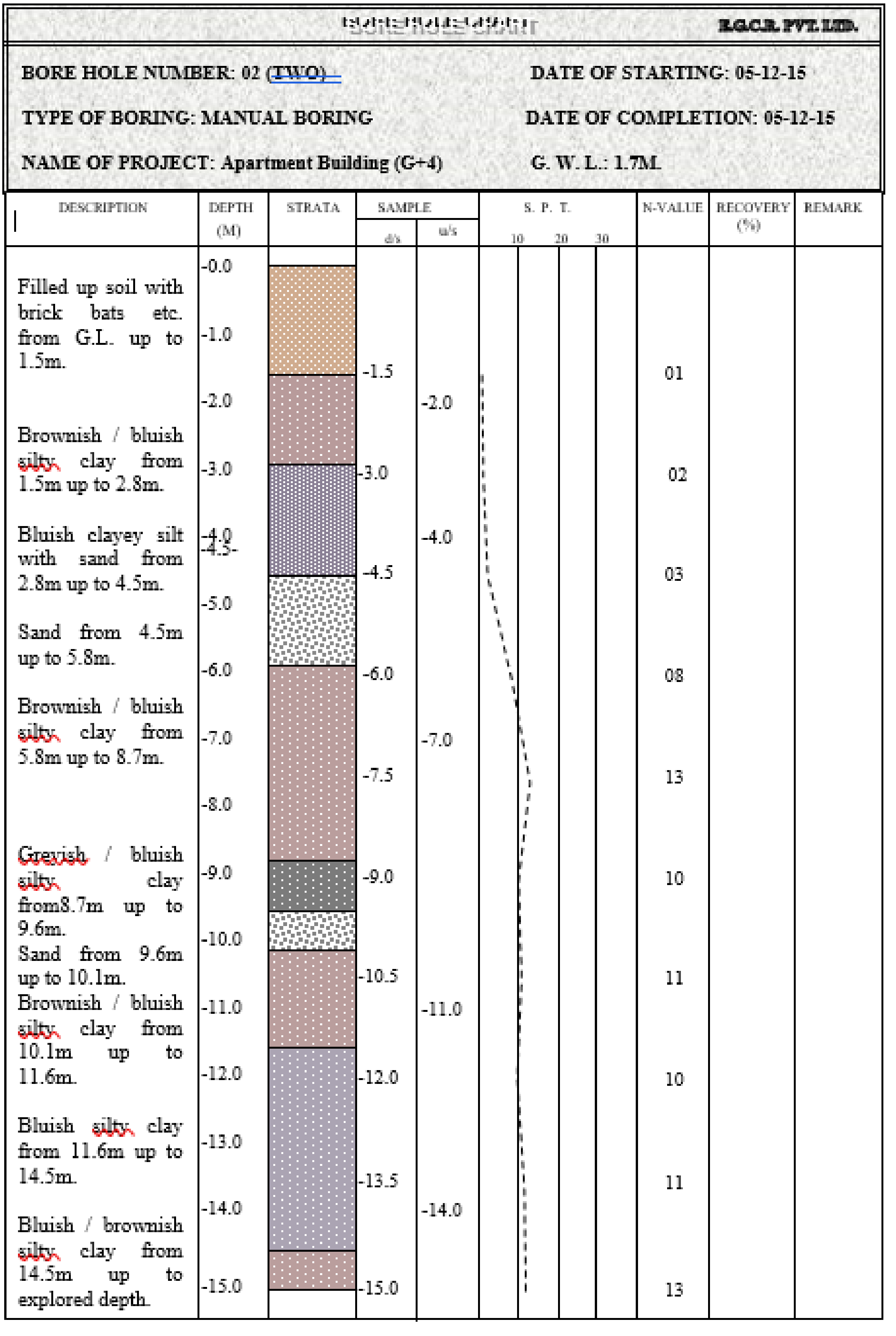


Figure 3.1: Bore-log of borehole-1 obtained from Geotechnical Test report for the proposed Apartment Building (G+4) at Shreenagar, near State Zoo, Guwahati-5.



BORE HOLE CHART — R.G.C.R. PVT. LTD.

BORE HOLE NUMBER: 02 (TWO)
DATE OF STARTING: 05-12-15
TYPE OF BORING: MANUAL BORING
DATE OF COMPLETION: 05-12-15
NAME OF PROJECT: Apartment Building (G+4)
G. W. L.: 1.7M.

DESCRIPTION	DEPTH (M)	STRATA	SAMPLE d/s	SAMPLE u/s	S. P. T. (10 20 30)	N-VALUE	RECOVERY (%)	REMARK
Filled up soil with brick bats etc. from G.L. up to 1.5m.	-0.0, -1.0		-1.5			01		
Brownish / bluish silty clay from 1.5m up to 2.8m.	-2.0, -3.0		-3.0	-2.0		02		
Bluish clayey silt with sand from 2.8m up to 4.5m.	-4.0, -4.5-		-4.5	-4.0		03		
Sand from 4.5m up to 5.8m.	-5.0, -6.0		-6.0			08		
Brownish / bluish silty clay from 5.8m up to 8.7m.	-7.0, -8.0		-7.5	-7.0		13		
Greyish / bluish silty clay from8.7m up to 9.6m.	-9.0		-9.0			10		
Sand from 9.6m up to 10.1m.	-10.0							
Brownish / bluish silty clay from 10.1m up to 11.6m.	-11.0		-10.5	-11.0		11		
Bluish silty clay from 11.6m up to 14.5m.	-12.0, -13.0, -14.0		-12.0, -13.5	-14.0		10, 11		
Bluish / brownish silty clay from 14.5m up to explored depth.	-15.0		-15.0			13		

Figure 3.2: Bore-log of borehole-2 obtained from Geotechnical Test report for the proposed Apartment Building (G+4) at Shreenagar, near State Zoo, Guwahati-5.

3.2.2 STEP 2: Correction of SPT-N values

The Observed N values are corrected for hammer efficiency of 60% and for effect of fines content using **IS 1893 (Part 1): 2016**

For evaluation SPT blow count N_{60}, for hammer efficiency of 60% of non- standard type equipment, N_{60} shall be obtained by the relation-

$$\mathbf{N_{60}= N*C_{60}} \tag{3.1}$$

where N= observed N value (uncorrected)

and $$\mathbf{C_{60}=C_{HT}*C_{WT}*C_{SS}*C_{RL}*C_{BD}} \tag{3.2}$$

Factors C_{HT}, C_{WT}, C_{SS}, C_{RL} and C_{BD} as recommended by various investigators for some common non-standard SPT configurations are provided in the table 3.1

Table 3.1: Correction Factors for Non-standard SPT Procedures and Equipment (Clause-F-1, step 6(a))

Sl No. (1)	Correction for (2)	Correction Factor (3)
i)	Non-standard hammer weight or height of fall	C_{HT} = 0.75 (for Donut hammer with rope and pulley) 1.33 (for Donut hammer with trip/auto) and Energy ratio = 80 percent
ii)	Non-standard hammer weight or height of fall	$C_{HW} = \frac{HW}{48387}$ where H = height of fall (mm), and W = hammer weight (kg)
iii)	Non-standard sampler setup (standard samples with room for liners, but used without liners)	C_{SS} = 1.1 (for loose sand) 1.2 (for dense sand)
iv)	Non-standard sampler setup (standard samples with room for liners, but liners are used)	C_{SS} = 0.9 (for loose sand) 0.8 (for dense sand)
v)	Short rod length	C_{RL} = 0.75 (for rod length 0-3 m) = 0.80 (for rod length 3-4 m) = 0.85 (for rod length 4-6 m) = 0.95 (for rod length 6-10 m) = 1.0 (for rod length 10-30 m)
vi)	Nonstandard borehole diameter	C_{BD} = 1.00 (for bore hole diameter of 65-115 mm) = 1.05 (for bore hole diameter of 150 mm) = 1.15 (for bore hole diameter of 200 mm)

NOTES

1 N = Uncorrected SPT blow count.

2 $C_{60} = C_{HT}C_{HW}C_{SS}C_{RL}C_{BD}$

3 $N_{60} = NC_{60}$

4 C_N = Correction factor for overburden pressure $(N_1)_{60} = C_N C_{60} N$.

The computed N_{60} is then normalised to an effective overburden pressure of approximately 100 Kpa using overburden correction factor C_N

$$(N_1)_{60}=C_N*N_{60} \tag{3.3}$$

where $C_N = (P_a / \sigma')^{0.5} \leq 1.7$ **(3.4)**

and **σ'= Effective stress**

The effect of fines content can be rationally accounted by correcting $(N_1)_{60}$ and finding $(N_1)_{60cs}$ as follows-

$$(N_1)_{60cs} = \alpha+\beta*(N_1)_{60} \tag{3.5}$$

where

Table3.2: Values of α and β according to the varying Fines content (Source-IS 1893 Part1:2016)

$\alpha = 0$	$\beta = 1$	For FC ≤ 5 *percent*
$\alpha = e^{\left[1.76-\left(\frac{190}{FC^2}\right)\right]}$	$\beta = 0.99 + \frac{FC^{1.5}}{1000}$	$For\ 5\ percent < FC < 35\ percent$
$\alpha = 5$	$\beta = 1.2$	For FC ≥ 35 *percent*

Table 3.3: Table showing SPT-N value correction process in borehole-1 of proposed Apartment Building (G+4) at Shreenagar, near State Zoo, Guwahati.

Site-Apartment Building (G+4) at Shreenagar, near state Zoo,

BORE HOLE 1	Depth	Observed N-value	Bulk density (g/cc)	Bulk density(kN/m3)	Total stress (kN/m2)	Effective stress(kN/m2)	Cn	Cn	Crl	C60	(N1)60	FC	(N1)60cs	C(kg/cm2)	C (kN/m2)	Angle of internal friction (degree)
G.W.L=2.5 m																
	1.5	2	1.94	19.031	22.955	23.103	2.0805	1.7000	0.75	0.4534	1.5415	100	7	0.25	24.53	2
	3.0	5	1.94	19.031	47.775	42.870	1.5273	1.5273	0.75	0.4534	3.4622	100	9	0.38	37.28	2
	4.5	15	1.96	19.228	76.616	56.996	1.3246	1.3246	0.85	0.5138	10.2090	2	17	0.00	0.00	31
	6.0	29	1.96	19.228	105.458	71.123	1.1858	1.1858	0.85	0.5138	17.6689	2	26	0.00	0.00	34
	7.5	5	1.96	19.228	134.299	85.249	1.0831	1.0831	0.95	0.5743	3.1099	100	9	0.39	38.26	2
	9.0	13	1.96	19.228	163.140	99.375	1.0031	1.0031	0.95	0.5743	7.4890	26	13	0.57	55.92	2
	10.5	12	1.96	19.228	191.982	113.502	0.9386	0.9386	1.00	0.6045	6.8089	100	13	0.57	55.92	2
	12.0	12	1.99	19.522	221.265	128.070	0.8836	0.8836	1.00	0.6045	6.4099	100	13	0.57	55.92	2
	13.5	13	2.00	19.620	250.695	142.785	0.8369	0.8369	1.00	0.6045	6.5766	100	13	0.57	55.92	2
	15.0	17	2.00	19.620	280.125	157.500	0.7968	0.7968	1.00	0.6045	8.1885	100	15	0.65	63.77	2

Table 3.4: Table showing SPT-N value correction process in borehole-2 of proposed Apartment Building (G+4) at Shreenagar, near State Zoo, Guwahati.

Site-Apartment Building (G+4) at Shreenagar, near state Zoo,

BORE HOLE 2	Depth	Observed N-value	Bulk density (g/cc)	Bulk density(kN/m3)	Total stress (kN/m2)	Effective stress(kN/m2)	Cn	Cn	Crl	C60	(N1)60	FC	(N1)60cs	C(kg/cm2)	C (kN/m2)	Angle of internal friction (degree)
G.W.L=1.7m																
	1.5	1	1.93	18.933	22.808	22.8080	2.0939	1.70	0.7500	0.4534	0.77074	100	6	0.25	24.525	2.0
	3.0	2	1.93	18.933	50.463	37.7096	1.6284	1.63	0.7500	0.4534	1.47659	58	7	0.25	24.525	2.0
	4.5	3	1.91	18.737	78.568	51.1003	1.3989	1.40	0.8500	0.5138	2.15638	7	2	0.12	11.772	6.0
	6.0	8	1.91	18.737	106.674	64.4909	1.2452	1.25	0.8500	0.5138	5.11866	100	11	0.51	50.031	2.0
	7.5	13	1.98	19.424	135.810	78.9116	1.1257	1.13	0.9500	0.5743	8.40413	100	15	0.51	50.031	2.0
	9.0	10	1.98	19.424	164.945	93.3323	1.0351	1.04	0.9500	0.5743	5.94434	100	12	0.51	50.031	2.0
	10.5	11	1.98	19.424	194.081	107.7530	0.9634	0.96	1.0000	0.6045	6.40581	99	13	0.56	54.936	2.0
	12.0	10	1.99	19.522	223.364	122.3209	0.9042	0.90	1.0000	0.6045	5.4657	100	12	0.56	54.936	2.0
	13.5	11	1.99	19.522	252.647	136.8887	0.8547	0.85	1.0000	0.6045	5.68336	100	12	0.56	54.936	2.0
	15.0	13	2.00	19.620	282.077	151.6037	0.8122	0.81	1.0000	0.6045	6.38241	100	13	0.64	62.784	2.0

Table 3.5: Table showing SPT-N value correction process in borehole-3 of proposed Apartment Building (G+4) at Shreenagar, near State Zoo, Guwahati.

		Site-Apartment Building (G+4) at Shreenagar, near state Zoo,														
BORE HOLE 3	Depth	Observed N-value	Bulk density (g/cc)	Bulk density(kN/m3)	Total stress (kN/m2)	Effective stress(kN/m2)	Cn	Cn	Crl	C60	(N1)60	FC	(N1)60cs	C(kg/cm2)	C (kN/m2)	Angle of internal friction (degree)
G.W.L=2.2 m																
	1.5	4	1.95	19.130	23.103	23.1030	2.0805	1.70	0.7500	0.4534	3.08295	80	9	0.40	39.24	2.0
	3.0	3	1.95	19.130	49.187	41.3393	1.5553	1.56	0.7500	0.4534	2.11542	100	8	0.40	39.24	2.0
	4.5	9	1.95	19.130	77.882	55.3186	1.3445	1.34	0.8500	0.5138	6.21759	4	6	0.00	0	26.0
	6.0	4	1.95	19.130	106.576	69.2978	1.2013	1.20	0.8500	0.5138	2.46897	100	8	0.40	39.24	2.0
	7.5	13	2.00	19.620	136.006	84.0128	1.0910	1.09	0.9500	0.5743	8.14499	100	15	0.65	63.765	2.0
	9.0	14	2.00	19.620	165.436	98.7278	1.0064	1.01	0.9500	0.5743	8.09148	87	15	0.65	63.765	5.0
	10.5	8	2.00	19.620	194.866	113.4428	0.9389	0.94	1.0000	0.6045	4.54044	100	10	0.49	48.069	5.0
	12.0	8	1.98	19.424	224.002	127.8635	0.8844	0.88	1.0000	0.6045	4.27674	100	10	0.49	48.069	2.0
	13.5	11	1.99	19.522	253.284	142.4314	0.8379	0.84	1.0000	0.6045	5.57168	100	12	0.56	54.936	2.0
	15.0	14	1.99	19.522	282.567	156.9992	0.7981	0.80	1.0000	0.6045	6.75423	100	13	0.56	54.936	2.0

3.2.3 Calculations

For Borehole-1,

Let us take a depth of 3m from the ground surface,

N-value observed=5

Bulk Density= 19.031 kN/m^3

Ground Water Level=2.5m

Total stress at the depth of 3m:

= 2.5*(dry density of soil till 0.5m from ground level) + (3-2.5) *(Bulk density at 3m depth)

= 0.5*1.56*9.81 + (3-2.5) *19.031

=47.775 kn/m^2

Effective Stress at the depth of 3m:

=Total stress-9.81*(3-2.5)

= 47.775-4.905

=42.870 kn/m^2

From equation 3.4, $C_N = (P_a / \sigma')^{0.5} \leq 1.7$

$= (100 / 42.870)^{0.5}$

$C_N = 1.5273$

Hence, $C_N = 1.5273$

From Table 3.1,

$C_{RL} = 0.75$ (Rod length 0-3m)

$C_{HT} = 0.75$ (Donut Hammer with rope and pulley)

$C_{HW} = HW/48387 = (65*750)/48387 = 1.0075$

where, H= Height of fall in mm=750mm,

W= Hammer weight in kg=65kg

$C_{SS}=0.8$ (Dense sand with liners)

$C_{RL}=0.75$

$C_{BD}=1.00$ (Considering 100mm diameter bore hole)

From equation 3.2,

$$C_{60}=C_{HT}*C_{WT}*C_{SS}*C_{RL}*C_{BD}$$

Hence, $C_{60} = 0.75*1.0075*0.8*0.75*1.00$

$\mathbf{C_{60} = 0.4534}$

Hence from equation 3.3, $(N_1)_{60}=C_N*N_{60}$ (where $N_{60}= N*C_{60}$)

$= C_N* N*C_{60}$

$= 1.5273*5*0.4534$

$\mathbf{(N_1)_{60} = 3}$

Fines Content =100%

Hence from table 14, $\alpha= 5$, $\beta = 1.2$

Hence, from Equation 3.5, $(N_1)_{60cs} = \alpha+\beta*(N_1)_{60}$

$= 0.5+1.2*1.5415$

$\mathbf{(N_1)_{60cs} = 9}$

Hence, the **Corrected N value at the depth of 3m, $(N_1)_{60cs}$ = 9**

cohesion at the depth of 3m (as collected from the report) = 0.38 kg/cm^2

Unit cohesion =37.28 kN/m^2

Angle of internal friction, $\boldsymbol{\phi = 2^\circ}$

In the same manner, all the SPT-N values for the subsequent depths are corrected and it is presented in a tabular form (Table 3.3)

The corresponding unit cohesion and angle of internal friction is also noted down in the table.

3.2.4 Step-3: $(N_1)_{60cs}$ determination for all boreholes

Approximately 100 numbers of boreholes from different sites in Guwahati City are analysed and the subsequent $(N_1)_{60cs}$ are found out from the N-observed values.

The $(N_1)_{60cs}$, unit cohesion and angle of internal friction is noted in the tabular form in table 3.6

Table 3.6: Corrected SPT-N values, cohesion and internal angle of friction values for boreholes of different sites in Guwahati City.

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
8	23.54	2
8	23.54	2
8	23.54	2
12	43.16	2
12	50.03	2
13	50.03	2
11	50.03	2
12	55.92	2
11	50.03	2
10	50.03	2
8	27.47	2
9	37.28	2
11	50.03	3

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
13	50.03	2
13	50.03	2
13	50.03	2
14	56.90	2
13	56.90	2
14	56.90	2
13	56.90	2
9	29.43	2
11	49.05	2
13	49.05	2
13	49.05	2
14	55.92	2
14	55.92	2
12	51.01	2
12	51.01	2
11	51.01	2
12	51.01	2
9	23.54	2
14	32.37	2
11	32.37	2
10	32.37	2
13	51.01	2
12	55.92	2
12	55.92	2
10	50.03	2
10	50.03	4
11	50.03	4
10	35.32	2
9	35.32	2
10	35.32	2
11	36.30	2
11	48.07	2
12	53.96	2
11	53.96	2
14	55.92	2
11	55.92	2
10	55.92	2
8	27.47	2
9	36.30	2
9	36.30	2
10	44.15	2
11	44.15	2
14	60.82	2
15	60.82	3
15	60.82	3
12	49.05	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
14	49.05	2
8	36.30	2
9	36.30	2
9	36.30	2
10	36.30	2
11	49.05	2
14	55.92	2
15	55.92	2
15	55.92	2
12	51.01	2
9	36.30	3
14	43.16	2
11	43.16	2
10	43.16	2
13	54.94	2
12	54.94	2
12	54.94	2
10	50.03	2
10	50.03	2
11	50.03	2
8	29.43	2
11	43.16	2
12	49.05	2
14	54.94	2
13	54.94	2
11	50.03	2
13	55.92	2
9	37.28	2
11	44.15	2
10	44.15	2
13	50.03	2
13	50.03	2
13	54.94	3
10	54.94	3
12	55.92	3
14	55.92	3
6	23.54	2
10	33.35	2
9	33.35	2
10	39.24	2
11	39.24	2
10	39.24	2
11	45.13	2
12	50.03	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
6	24.53	2
6	24.53	2
9	31.39	2
10	37.28	2
8	37.28	2
12	39.24	2
13	49.05	2
14	49.05	2
14	49.05	2
7	22.07	2
9	37.28	2
9	38.26	2
13	55.92	2
13	55.92	2
13	55.92	2
13	55.92	2
15	63.77	2
6	24.53	2
7	24.53	2
15	50.03	2
12	50.03	2
13	54.94	2
12	54.94	2
12	54.94	2
13	62.78	2
9	39.24	2
8	39.24	2
8	39.24	2
15	63.77	2
15	63.77	5
10	48.07	5
10	48.07	2
12	54.94	2
13	54.94	2
7	21.58	4
6	21.58	4
7	21.58	4
6	21.58	2
6	21.58	2
8	27.47	2
9	36.30	2
12	48.07	2
10	48.07	2
16	55.92	2
8	29.43	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
7	29.43	2
8	29.43	2
6	29.43	2
7	29.43	3
8	29.43	3
8	37.28	3
9	37.28	3
10	41.20	5
9	22.56	3
8	22.56	3
7	22.56	2
8	27.47	2
11	43.16	2
10	43.16	2
10	45.13	2
11	45.13	2
11	54.94	2
11	54.94	2
8	21.58	3
8	21.58	2
7	21.58	2
7	21.58	2
11	37.28	2
9	37.28	2
9	37.28	2
10	37.28	2
9	27.47	2
6	27.47	2
7	27.47	2
6	27.47	2
8	36.30	2
9	36.30	2
9	37.28	2
9	37.28	2
10	37.28	2
8	22.56	2
8	27.47	2
9	27.47	2
9	38.26	2
10	38.26	2
11	49.05	2
6	21.58	2
8	28.45	2
8	28.45	2
9	28.45	2
8	28.45	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
13	42.18	2
9	29.43	2
9	29.43	2
11	35.32	3
6	21.58	2
7	21.58	2
7	21.58	2
8	27.47	2
11	40.22	2
11	40.22	5
8	29.43	2
9	29.43	2
8	29.43	2
7	27.47	2
7	27.47	2
9	37.28	2
8	37.28	2
10	37.28	2
10	37.28	2
9	37.28	2
9	37.28	2
9	37.28	2
9	27.47	2
7	27.47	2
8	27.47	2
8	27.47	2
7	27.47	2
12	43.16	2
9	27.47	2
10	37.28	2
8	37.28	2
8	37.28	2
14	53.96	3
6	27.47	2
5	27.47	2
12	46.11	2
11	46.11	2
12	46.11	2
13	51.01	2
14	54.94	2
12	54.94	2
13	54.94	2
14	54.94	2
6	23.54	2
6	23.54	2
6	23.54	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
6	23.54	2
6	23.54	2
7	23.54	2
11	44.15	2
11	44.15	2
6	20.60	2
12	46.11	2
15	58.86	2
14	54.94	2
14	54.94	5
14	54.94	3
15	58.86	3
6	21.58	2
8	21.58	2
7	21.58	2
6	21.58	2
7	21.58	2
6	22.56	2
7	22.56	2
10	39.24	2
6	22.56	2
8	22.56	2
7	22.56	2
6	22.56	2
7	24.53	2
8	24.53	2
7	24.53	2
7	29.43	3
7	29.43	3
11	44.15	2
11	44.15	2
12	49.05	2
12	49.05	2
14	55.92	4
15	55.92	4
14	53.96	3
7	34.34	3
17	55.92	3
17	55.92	4
17	55.92	4
6	22.56	3
7	22.56	3
7	22.56	3
6	22.56	3
7	22.56	4
6	21.58	4

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
14	59.84	4
7	22.56	2
7	22.56	3
7	25.51	3
7	25.51	3
9	29.43	3
13	55.92	3
12	55.92	3
8	31.39	3
9	31.39	5
12	54.94	5
6	22.56	3
8	22.56	3
7	22.56	3
6	22.56	3
7	26.49	3
8	26.49	3
11	48.07	2
9	29.43	2
8	29.43	2
7	29.43	3
10	41.20	3
11	41.20	2
10	41.20	2
12	46.11	2
18	56.90	2
8	31.39	2
10	39.24	2
17	55.92	2
7	29.43	2
9	36.30	2
10	36.30	2
9	36.30	2
10	39.24	2
8	39.24	2
8	37.28	2
10	37.28	2
14	49.05	3
11	43.16	3
9	37.28	2
6	24.53	2
6	24.53	2
8	34.34	2
9	34.34	2
9	35.32	2
10	35.32	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
12	43.16	2
9	30.41	2
8	30.41	2
8	30.41	2
10	39.24	2
7	30.41	2
10	37.28	3
8	31.39	3
8	31.39	2
13	55.92	2
11	49.05	2
11	49.05	2
12	49.05	2
15	56.90	2
10	38.26	4
8	31.39	4
8	31.39	5
13	45.13	5
11	45.13	2
11	51.01	2
12	51.01	2
15	59.84	5
15	59.84	5
16	59.84	5
8	29.43	2
9	38.26	2
9	38.26	2
9	39.24	2
8	32.37	2
5	21.58	3
7	37.28	3
9	37.28	2
9	37.28	2
10	39.24	2
15	54.94	2
12	54.94	2
13	54.94	2
12	54.94	2
16	62.78	2
8	37.28	2
8	37.28	2
10	39.24	2
9	39.24	2
10	45.13	2
12	45.13	2
12	51.01	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
15	51.01	2
12	51.01	2
12	51.01	2
7	29.43	3
12	41.20	5
10	41.20	5
8	33.35	5
8	33.35	3
8	33.35	3
8	33.35	2
7	28.45	2
7	27.47	4
8	34.34	4
8	34.34	3
6	22.56	4
9	37.28	3
8	33.35	3
8	33.35	3
9	37.28	2
7	34.34	2
8	34.34	2
8	35.32	2
8	35.32	2
8	35.32	2
8	31.39	5
14	53.96	3
6	22.56	2
11	41.20	2
11	41.20	2
12	44.15	2
12	47.09	2
10	40.22	2
11	40.22	2
10	40.22	2
6	22.56	2
8	34.34	2
9	35.32	2
10	39.24	2
12	44.15	2
12	44.15	2
12	45.13	2
12	45.13	2
11	45.13	2
6	21.58	2
10	41.20	2
9	37.28	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
11	48.07	2
12	48.07	2
10	41.20	2
11	41.20	2
12	53.96	2
8	31.39	2
9	37.28	2
9	37.28	2
19	58.86	2
8	31.39	2
11	44.15	3
8	34.34	2
10	41.20	2
9	39.24	2
20	63.77	2
11	44.15	2
12	48.07	3
6	21.58	2
7	22.56	2
6	22.56	2
12	45.13	2
18	55.92	2
18	55.92	2
19	63.77	2
18	63.77	2
16	55.92	2
16	55.92	2
15	53.96	2
15	53.96	2
16	55.92	2
16	55.92	2
14	51.99	2
13	51.99	2
5	22.56	2
6	22.56	2
9	37.28	2
10	38.26	2
9	38.26	2
15	51.01	2
15	51.01	5
17	55.92	2
16	55.92	3
15	51.01	2
15	51.01	2
15	51.01	2
13	51.01	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
15	51.01	2
15	51.01	2
14	51.01	3
6	21.58	2
6	21.58	2
9	21.58	2
10	21.58	2
8	21.58	2
12	45.00	2
13	45.00	5
14	45.00	2
15	55.00	3
14	45.00	2
15	55.00	2
13	45.00	2
14	55.00	2
15	55.00	2
15	55.00	2
14	55.00	3
7	30.41	3
5	21.58	3
10	38.26	3
11	39.24	2
11	39.24	2
10	38.26	2
13	42.18	2
12	48.07	2
12	48.07	2
6	21.58	2
7	21.58	2
8	34.34	2
10	39.24	2
8	34.34	**2**
14	47.09	2
15	60.82	2
15	60.82	2
13	48.07	2
9	38.26	2
6	21.58	3
6	21.58	3
7	21.58	3
7	21.58	3
17	60.82	3
15	54.94	2
11	44.15	2
18	58.86	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
8	33.35	2
7	21.58	2
10	39.24	2
12	44.15	2
18	58.86	3
12	49.05	2
14	52.97	2
7	21.58	2
6	21.58	2
7	31.39	2
6	21.58	2
6	21.58	2
9	37.28	2
15	53.96	2
17	56.90	2
13	51.01	3
11	51.01	3
12	49.05	3
12	49.05	2
7	33.35	2
6	21.58	3
6	21.58	3
5	21.58	3
7	33.35	3
6	21.58	3
7	21.58	2
14	49.05	2
11	44.15	2
9	37.28	2
9	37.28	2
7	33.35	2
7	33.35	2
12	49.05	4
13	49.05	4
7	21.58	3
7	21.58	3
10	39.24	2
11	44.15	2
12	49.05	2
12	49.05	2
13	53.96	4
15	53.96	4
13	53.96	3
9	37.28	2
11	41.20	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
13	45.13	2
15	53.96	2
16	56.90	2
16	56.90	2
15	53.96	2
15	53.96	2
17	57.88	2
18	58.86	2
9	39.24	3
11	47.09	3
12	47.09	3
14	53.96	2
16	56.90	2
10	39.24	2
15	53.96	2
15	53.96	2
17	56.90	2
18	58.86	2
7	31.39	2
10	38.26	2
9	38.26	2
10	35.32	2
11	43.16	2
14	49.05	2
13	49.05	2
15	53.96	5
14	53.96	5
11	44.15	5
6	21.58	2
9	37.28	2
9	37.28	2
10	39.24	2
11	41.20	2
13	49.05	2
14	56.90	2
14	56.90	2
14	56.90	2
12	44.15	2
7	29.43	2
9	37.28	2
15	53.96	2
16	53.96	2
15	53.96	2
18	60.82	2
17	56.90	2
17	56.90	2

$(N_1)_{60cs}$	Cohesion(kN/m2)	Internal angle of friction (degree)
7	31.39	2
11	44.15	2
14	53.96	2
15	53.96	2
17	58.86	2
18	58.86	2
15	54.94	2
16	56.90	2
7	31.39	2
8	34.34	2
12	47.09	2
12	47.09	2
17	58.86	2
18	58.86	2
18	58.86	2
13	49.05	2
8	37.28	3
9	37.28	3
7	31.39	3
6	31.39	2
7	31.39	2
7	31.39	3
8	37.28	3
11	43.16	2
11	43.16	2
8	31.39	2
7	31.39	2
9	37.28	2
8	37.28	3

3.2.5 Step-4: Correlation between $(N_1)_{60cs}$ and unit cohesion(c) of soil

We try to derive a correlation between the $(N_1)_{60cs}$ and unit cohesion(c) considering the angle of internal friction.

For this we plot a graph between $(N_1)_{60cs}$ and unit cohesion of a certain range of soil having angle of internal friction, $\phi \leq 5^\circ$ in Origin pro 8.5 software.

Non-linear regression is performed on the data set and the best fit curve has been obtained as below

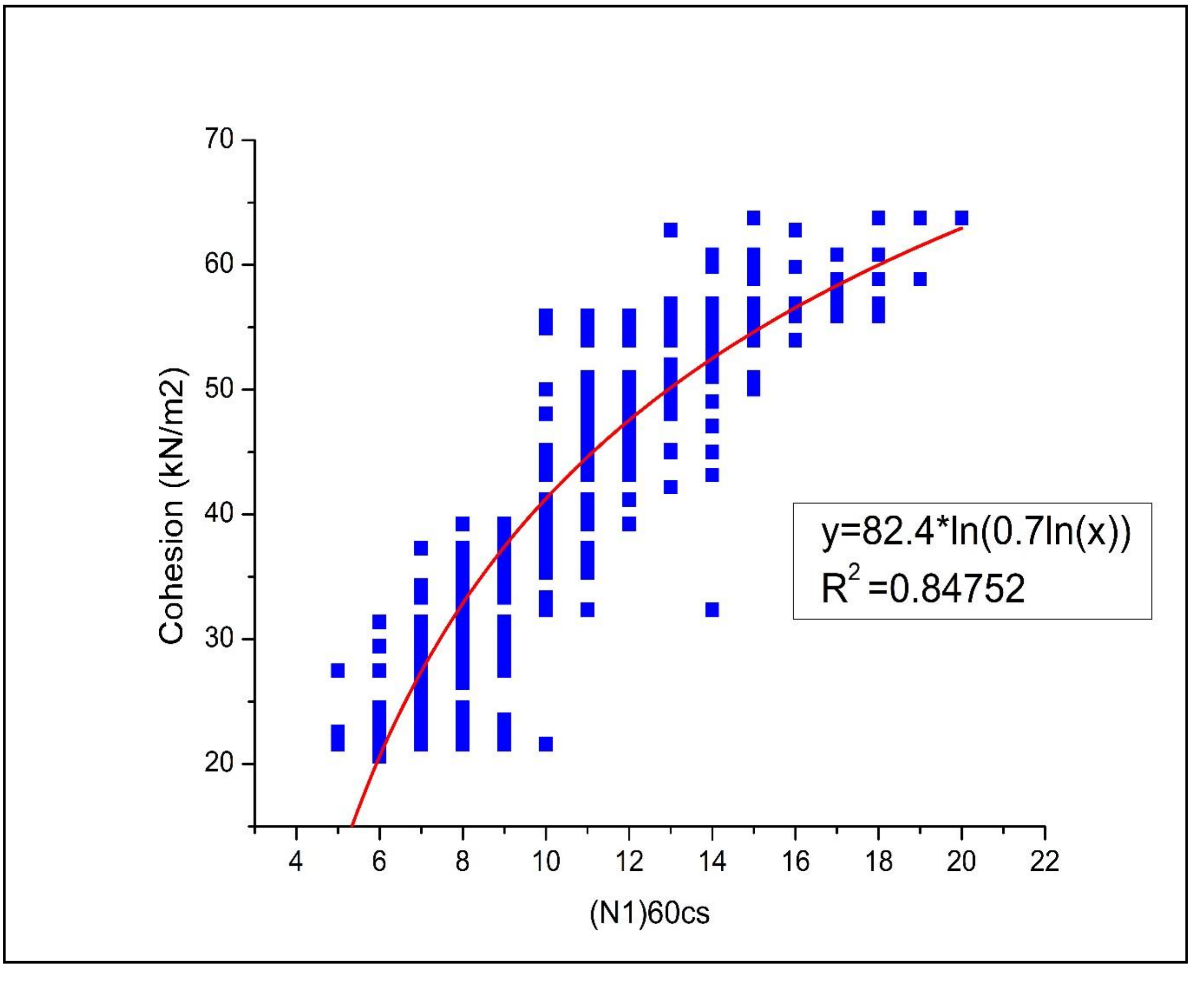


Figure 3.3: Graph between $(N_1)_{60cs}$ and unit cohesion (kN/m2) having angle of internal friction, $\phi \leq 5^\circ$

	A	B	C	D
1	Model	Bradley		
2	Equation	y = a*ln(-b*ln(x))		
3	Reduced Chi-Sqr	21.95933		
4	Adj. R-Square	0.84752		
5			Value	Standard Error
6	Cohesion	a	82.39481	1.38563
7		b	-0.71672	0.00618

Figure 3.4: Result of Non-linear curve fit analysis in origin pro 8.5

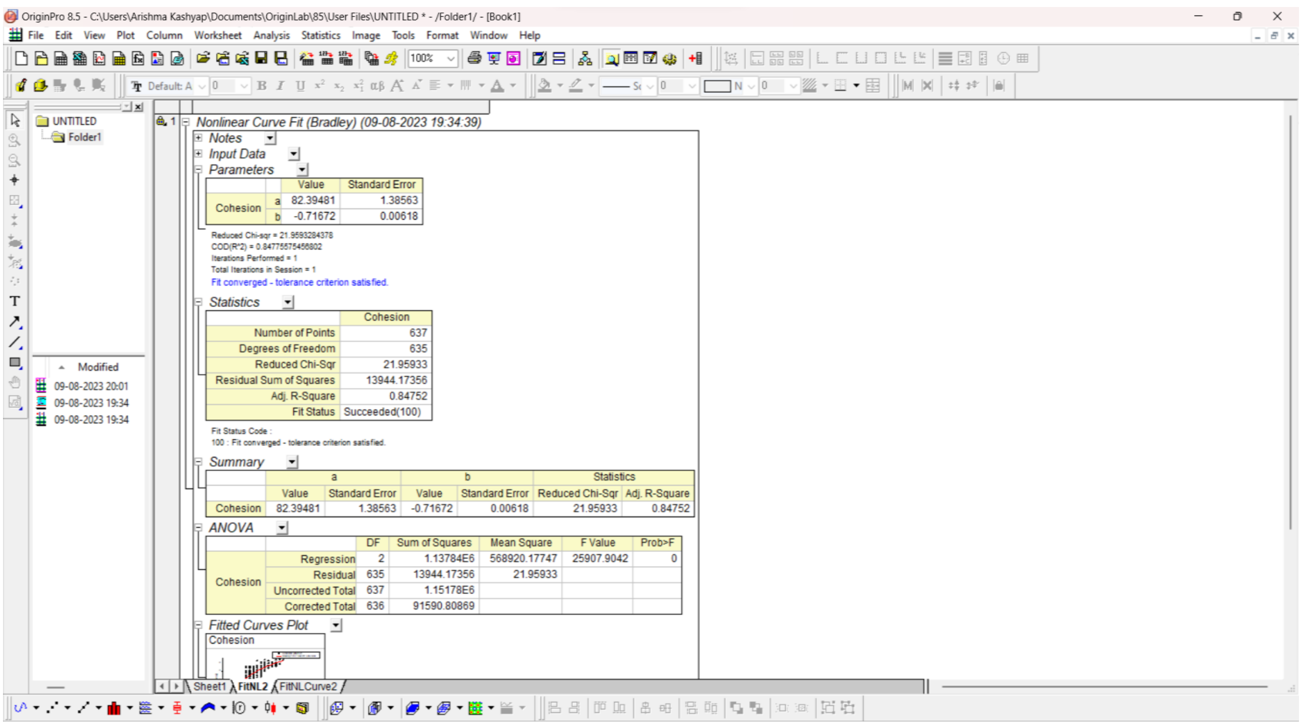


Figure 3.5: Result of Non-linear curve fit analysis in origin pro 8.5

3.2.6 Correlation between $(N_1)_{60cs}$ and Internal angle of friction of soil

From the data obtained from the bore-holes, a correlation between the $(N_1)_{60cs}$ and angle of internal friction for soil having negligible cohesion value i.e. 0 kN/m^2 is studied.

The data-set obtained from various bore-hole sites after necessary correction is as follows-

Table 3.7: Corrected SPT-N values and internal angle of friction for boreholes of different sites in Guwahati City.

$(N_1)_{60cs}$	Internal angle of friction (degree)
17	33.0
18	33.0
22	37.5
19	33.0
18	33.0
12	28.0
16	33.5
14	31.0
18	33.5
24	37.0
17	32.5
18	33.5
21	35.5
20	35.0
23	36.0
17	32.5
19	34.0
23	36.0
25	37.5
20	35.0
23	37.5
14	32.0
18	34.0
18	35.0
22	34.0
21	36.0
16	32.5
19	34.0
22	36.0
23	36.0
19	35.0
20	35.5
19	31.0

$(N_1)_{60cs}$	Internal angle of friction (degree)
22	35.0
19	35.0
22	35.0
12	30.0
13	30.0
11	28.0
16	30.0
16	31.0
16	33.0
13	32.0
15	32.5
16	33.0
18	34.0
20	35.0
18	34.0
16	30.0
16	30.5
17	33.5
16	33.0
17	33.5
18	34.0
17	33.0
14	31.0
15	32.5
16	33.0
14	31.5
16	33.5
16	33.5
16	33.5
17	34.0
11	28.0
13	29.0
10	28.0
11	28.0
11	28.0
17	33.0
14	32.0
15	32.5
16	33.0
19	34.0
19	34.0

Table 3.7: Corrected SPT-N values and internal angle of friction for boreholes of different sites in Guwahati City.

A graph between $(N_1)_{60cs}$ and angle of internal friction, ϕ, for the soil having negligible cohesion, in Origin pro 8.5 software, is plotted. Non-linear regression is performed on the data set and the best fit curve has been obtained as below.

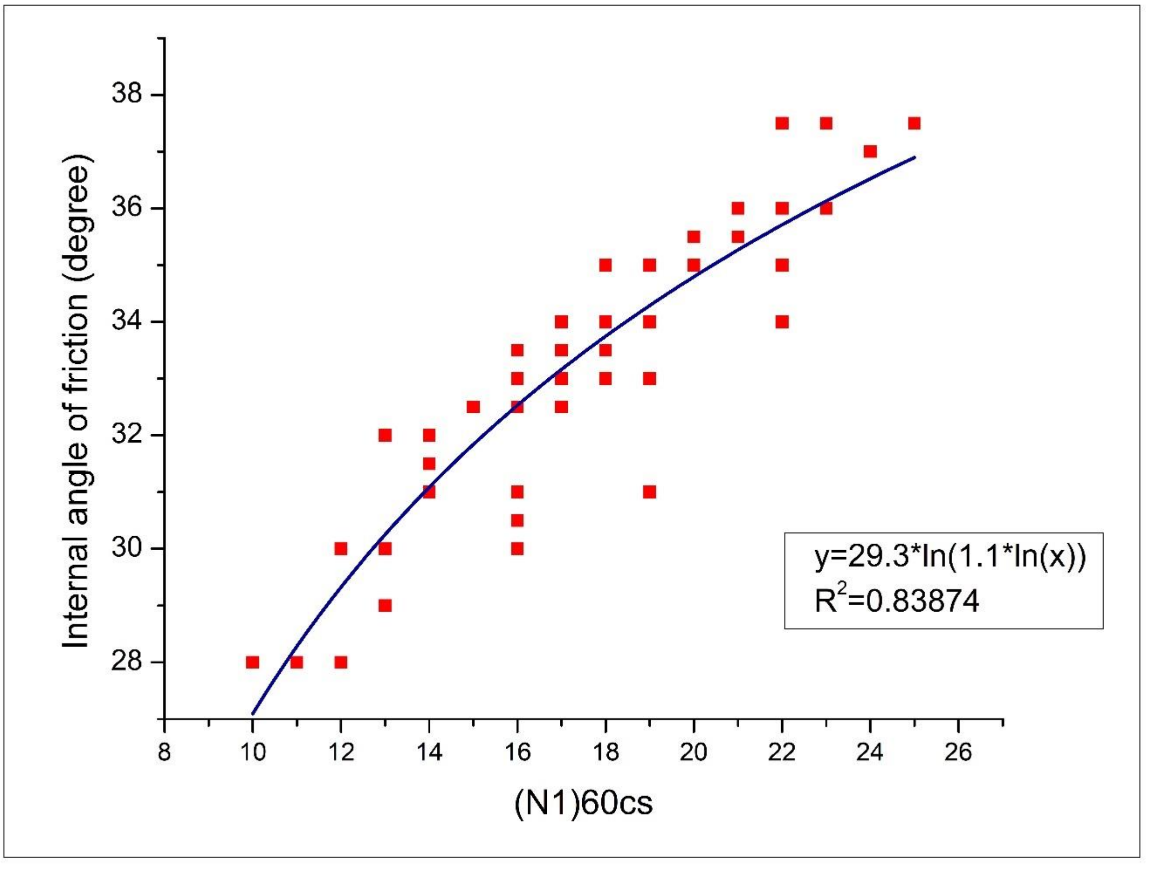


Figure 3.6: Graph between $(N_1)_{60cs}$ and angle of internal friction, ϕ

3.2.7 Comparison with Previous Proposed Relations

The correlations between $(N_1)_{60}$ and angle of internal friction, ϕ, given by different researchers is compared with the correlation obtained from the study and the graph is obtained from origin pro 8.5.

Table 3.8: Some relationships proposed by previous researchers

Relationships	References	Soil Type
$\Phi=27.1+0.3*N_{60}- 0.00054N_{60}^2$	Wolff (1989)	Sandy Soil
$\Phi=0.70N_{60}$	Mujtaba et al.(2017)	Sandy Soil

The following values of internal friction angles are obtained from table 3.7 considering the range of N-corrected from 10-25

Table 3.9: $(N_1)_{60}$ and Internal friction angle values as proposed by previous researchers

(N1)60	Internal friction angle(degree)by Wolff	Internal friction angle (degree) by Mujtaba et. al
10	30	25.0
11	30	25.7
12	31	26.4
13	31	27.1
14	31	27.8
15	32	28.5
16	32	29.2
17	32	29.9
18	32	30.6
19	33	31.3
20	33	32.0
21	33	32.7
22	34	33.4
23	34	34.1
24	34	34.8
25	35	35.5

The following comparative graph is obtained in origin pro 8.5

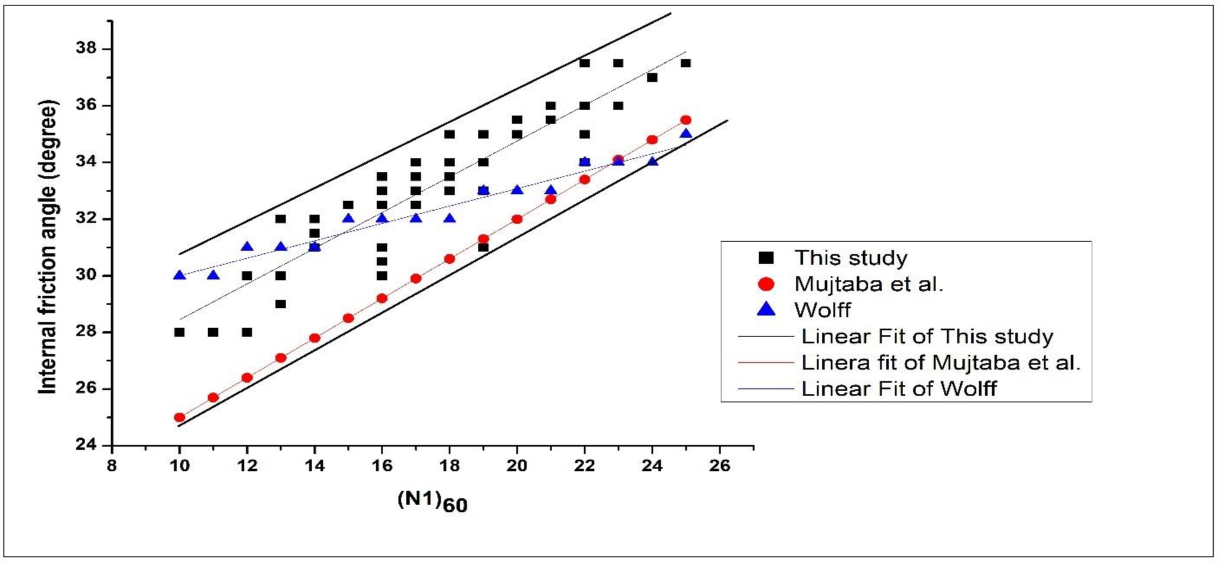


Figure 3.7: Comparison graph of the corelation of this study with previous studies

	A	B	C	D
1	Equation	y = a + b*x		
2	Weight	No Weighting		
3	Residual Sum of Squares	71.79544	3.78653E-29	1.31912
4	Pearson's r	0.90802	1	0.98008
5	Adj. R-Square	0.82206	1	0.95773
6			Value	Standard Error
7	THIS STUDY	Intercept	22.14074	0.60427
8		Slope	0.63103	0.03431
9	MUJTABA et.al	Intercept	18	1.61407E-15
10		Slope	0.7	8.91902E-17
11	WOLFF	Intercept	26.93382	0.30126
12		Slope	0.30735	0.01665

Figure 3.8: Result of linear regression of comparison graph

Range of Internal angle of friction for various corrected-N values as obtained from upper and lower bound of the graph in figure 3.7 are as follows-

Table 3.10: Range of internal angle for different N-corrected values

Corrected N value	**Internal friction angle value range (degree)**
10	24.5-30.5
11	25.5-31.5
12	26-32
13	26.5-32.5
14	27-33
15	28-33.5
16	28.5-34.5
17	29-35
18	30-35.5
19	30.5-36
20	31-36.5
21	32-37
22	32.5-38
23	33-38.5
24	33.5-39
25	34.5-40

3.2.8 Correlation between SPT-N corrected i.e. $(N_1)_{60cs}$, unit cohesion (c) and the angle of internal friction (ϕ) of soil.

With the data set obtained from the bore holes of different sites in Guwahati City, the following graph is plotted in origin pro 8.5 software.

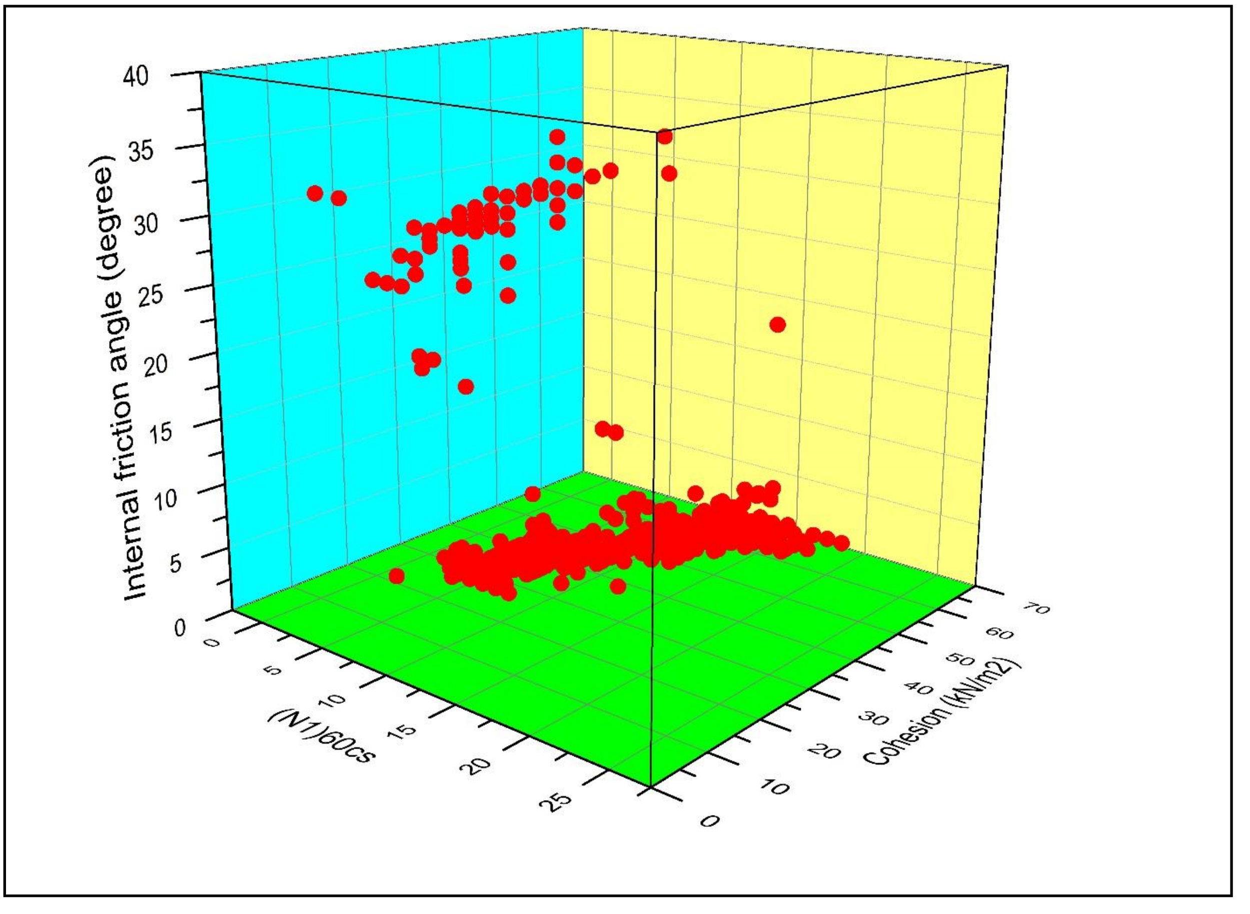


Figure 3.9: Graph between $(N_1)_{60cs}$, unit cohesion (c) and the angle of internal friction (ϕ) of soil.

The Non-linear curve fitting is done and the following result is obtained.

	A	B	C	D
1	Model	ExtremeCum		
2	Equation	z = z0 + B*exp(-exp(-(x - C)/D)) + E*exp(-exp(-(y - F)/G)) + H*exp(-exp(-(x - C)/D) -exp(-(y - F)/G));		
3	Reduced Chi-Sqr	4.36442		
4	Adj. R-Square	0.95408		
5			Value	Standard Error
6	Internal friction	z0	26.69865	1.25295
7	Internal friction	B	9.75792	1.93885
8	Internal friction	C	14.14544	0.85711
9	Internal friction	D	3.07929	1.05369
10	Internal friction	E	-24.24949	1.24261
11	Internal friction	F	12.3713	7693.59546
12	Internal friction	G	0.54673	6217.44266
13	Internal friction	H	-9.39588	1.97372

Figure 3.10: Result of Non-linear curve fitting analysis

The equation of the best fit surface is as follows –

$$\boldsymbol{z = 26.70 + 9.76e^{\left\{-e^{\left\{\frac{14.14-x}{3.08}\right\}}\right\}} - 24.25e^{\left\{-e^{\left\{\frac{12.37-y}{0.55}\right\}}\right\}} - 9.4e^{\left\{-e^{\left\{\frac{14.14-x}{3.08}\right\}}-e^{\left\{\frac{12.37-y}{0.55}\right\}}\right\}}} \quad \textbf{(3.6)}$$

where,

$x = (N_1)_{60cs}$

y = unit cohesion (c) in kN/m^2

z = Internal angle of friction in degree

$R^2 = 0.95$

CHAPTER 4

RESULTS AND DISCUSSION

4.1 RESULTS

The plot from $(N_1)_{60cs}$ and unit cohesion (c) with angle of internal friction, $\phi \leq 5^\circ$ gives us a correlation with **R^2 = 0.85**

The equation that we derive from the plot is as follows-

$$\mathbf{y = 82.4*ln(0.7*ln(x))}$$

where y= unit cohesion (c) in kN/m^2 and x=$(N_1)_{60cs}$

The equation can thus be written as-

$$\boxed{c = 82.4*\ln(0.7*\ln((N_1)_{60cs})),\ \text{For } \phi \leq 5^\circ} \qquad (4.1)$$

where, c= Unit cohesion in kN/m^2

$(N_1)_{60cs}$ = Corrected SPT-N values

ϕ = Internal angle of friction in degree

Thus, from the values of $(N_1)_{60cs}$, the unit cohesion in kN/m^2 can be calculated from Enq.6 in field itself.

The plot from $(N_1)_{60cs}$ and angle of internal friction, ϕ, with negligible cohesion, gives us a correlation with **R^2 = 0.84**

The equation that we derive from the plot is as follows-

$$\mathbf{y = 29.3*ln(1.1*ln(x))}$$

where y= angle of internal friction in degree

and x=$(N_1)_{60cs}$

The equation can thus be written as-

$$\boxed{\phi = 29.3*\ln(1.1*\ln((N_1)_{60cs}))} \qquad (4.2)$$

where, $(N_1)_{60cs}$ = Corrected SPT-N values
ϕ = Internal angle of friction in degree

Thus, from the values of $(N_1)_{60cs}$, the Internal angle of friction can be calculated from Enq.7 in field itself.

The plot between (N1)60cs, unit cohesion (c) and the angle of internal friction (ϕ) of soil gives us a correlation with $\mathbf{R^2 = 0.95}$

The equation of the best fit surface is as follows:

$$\boldsymbol{z = 26.70 + 9.76e^{\left\{-e^{\left\{\frac{14.14-x}{3.08}\right\}}\right\}} - 24.25e^{\left\{-e^{\left\{\frac{12.37-y}{0.55}\right\}}\right\}} - 9.4e^{\left\{-e^{\left\{\frac{14.14-x}{3.08}\right\}}-e^{\left\{\frac{12.37-y}{0.55}\right\}}\right\}}}$$

where, z= Internal angle of friction in degree

x= Corrected SPT-N values

y= Unit cohesion in kN/m^2

The equation can be written as:

$$\boxed{\boldsymbol{\phi = 26.70 + 9.76e^{\left\{-e^{\left\{\frac{14.14-(N1)60cs}{3.08}\right\}}\right\}} - 24.25e^{\left\{-e^{\left\{\frac{12.37-c}{0.55}\right\}}\right\}} - 9.4e^{\left\{-e^{\left\{\frac{14.14-(N1)60cs}{3.08}\right\}}-e^{\left\{\frac{12.37-c}{0.55}\right\}}\right\}}}}$$

(4.3)

where, c= Unit cohesion in kN/m^2
$(N_1)_{60cs}$ = Corrected SPT-N values
ϕ = Internal angle of friction in degree

Thus, from the values of $(N_1)_{60cs}$ and unit cohesion, the Internal angle of friction can be calculated from Enq.8 in field itself.

4.2 DISCUSSION

From the correlation (Equation 4.1), obtained from the result above, the value of cohesion in Kn/m^2, can be estimated from the SPT-N value ($(N_1)_{60cs}$). This equation is applicable for angle of internal friction, $\phi \leq 5^{\circ}$. The R^2 we have obtained for equation 4.1 is **0.85.**

The purpose of use of this statistical method for correlation determination is to give us a statistic known as the correlation coefficient which is a summary value of a large set of data representing the degree of linear association between two measured variables (R^2 of 0.70 to 1.0) are considered to be a strong relationship.

Hence, we can say that SPT-N value and unit cohesion of soil are strongly related to each. We can find unit cohesion of soil in the field from SPT-N values in case of situations where laboratory testing is not possible and a quick and reliable method is required for cohesion analysis.

Similarly, from equation 4.2, the corrected N-values gives the internal angle of friction for soil having negligible cohesion. The R^2 we have obtained is **0.84.** The graph is compared with the correlations of various other researchers and table 3.9 is obtained which gives the range of internal angle of friction possible for the N-corrected values.

Equation 4.3 shows the correlation between the corrected-N values with internal friction angle and unit cohesion; hence this correlation can be used to find the 3rd parameter when any two parameters are known in the field. The R^2 we have obtained for the correlation is **0.95**.

Since, the analysis is done for soil in different parts of Guwahati City, hence it can be stated that this correlation may be place specific.

These kinds of correlations which are obtained, helps in determining the geotechnical soil parameters quickly which can then be used to determine the bearing capacity of foundation, settlement calculations, ultimate load capacity of pile foundation, design safe load capacity of pile foundations etc.

CHAPTER 5

CONCLUSION AND SCOPE OF FURTHER STUDY

5.1 CONCLUSION

Due to equipment unavailability, financial considerations and limitations of time in a project, the different geotechnical parameters can be estimated by extracting SPT-N value from the in-situ SPT test. This test is considered to be the popular one to determine the sub soil parameters. Determining the soil geotechnical parameters and proposing practical relations using in-situ tests such as SPT could reduce the costs of construction projects considerably.

In general, various soil properties have been calculated using field and laboratory experiments, such as elastic and strength characteristics. There is an ability to discard conducting certain experiments in the absence of an appropriate budget, time constraints and a challenging field scenario. Instead, using data from adjoining sites or some statistical correlations are used to assess such soil properties. In the past, empirical correlations have been comprehensively used to estimate the soil properties for published data from various sources including the discrepancy of the test methods, test materials, and data explanation. The empirical soil correlations were established using field Standard Penetration Test values (SPT)-N value. The N value for its simplicity is commonly used as a simple strength assessment index value.

In this study, an attempt has been made to correlate the field SPT-N value after making necessary corrections, with that of the cohesion obtained from lab test. This correlation ultimately helps us to get a relationship between the SPT-N values with that of cohesion of soil taking into consideration the angle of internal friction. With the help of this correlation, we can assess the cohesion of soil in field itself without going into the laboratory tests. It is useful for preliminary investigation of the soil layers, where we intent to lay a foundation.

The Correlations obtained from this study are-

1. **$c = 82.4*\ln(0.7*\ln((N_1)_{60cs}))$, For $\phi \leq 5^{\circ}$**
2. **$\phi = 29.3*\ln(1.1*\ln((N_1)_{60cs}))$**
3. $$\boldsymbol{\phi = 26.70 + 9.76e^{\left\{-e^{\left\{\frac{14.14-(N1)60cs}{3.08}\right\}}\right\}} - 24.25e^{\left\{-e^{\left\{\frac{12.37-c}{0.55}\right\}}\right\}} - 9.4e^{\left\{-e^{\left\{\frac{14.14-(N1)60cs}{3.08}\right\}}-e^{\left\{\frac{12.37-c}{0.55}\right\}}\right\}}}$$

where, c= Unit cohesion in kN/m^2
$(N_1)_{60cs}$ = Corrected SPT-N values
ϕ = Internal angle of friction in degree

5.2 LIMITATION OF THE STUDY

In this study, the Standard penetration test is performed in boreholes of different places in Guwahati City, hence the correlation we have obtained may be valid for soil of Guwahati City. Since the values are not validated for other regions, hence the correlation may be region specific.

The correlation used for determining the cohesion can be used when there is limitation of time and budget constraints, further lab tests of the soil for finding the cohesion is always suggested for determination of the bearing capacity of the soil and for design of foundation.

5.3 SCOPE FOR FURTHER STUDY

This work effort can be extended to study the following topics-

1. Correlation between Liquid Limit and SPT-N value in cohesive soil.
2. Correlation between Plastic limit and SPT-N in cohesive soil.
3. Correlation between Plasticity index and SPT-N in cohesive soil.

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SOFTWARE

1. Origin Pro 8.5

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