DETERMINING RELATIONS BETWEEN SPT-N & SHEAR STRENGTH PARAMETERS OF DHAKA AND SYLHET SOILS USING MACHINE LEARNING APPROACH

by

170051021	**Nurul Amin Shuvo**
170051059	**Md. Ehsan Kabir**
170051070	**Md. Muftashin Muhim Bondhon**
170051088	**Shadman Rahman Sabab**

A THESIS SUBMITTED

FOR THE DEGREE OF BACHELOR OF SCIENCE IN CIVIL ENGINEERING

Department of Civil and Environmental Engineering

ISLAMIC UNIVERSITY OF TECHNOLOGY (IUT)

2021

THESIS APPROVAL

The thesis titled "**Determining relations between SPT-N & Shear Strength parameters of Dhaka and Sylhet soils Using Machine Learning Approach**" submitted by Nurul Amin Shuvo (170051021), Md. Ehsan Kabir (170051059), Md. Muftashin Muhim Bondhon (170051070) and Shadman Rahman Sabab (170051088) has been found as satisfactory and accepted as partial fulfillment of the requirement for the Degree Bachelor of Science in Civil Engineer.

Supervisor

Prof. Dr. Hossain Md. Shahin

Head of the Department

Department of Civil and Environmental Engineering (CEE)

Islamic University of Technology (IUT)

Board Bazar, Gazipur, Bangladesh

DECLARATION OF CANDIDATE

We hereby declare that the undergraduate research work reported in this thesis has been performed by us under the supervision of Professor Dr. Hossain Md. Shahin and this work has not been submitted elsewhere for any purpose (except for publication).

Nurul Amin Shuvo
Student No: 170051021
Academic Year: 2021-2022
Date:

Md. Ehsan Kabir
Student No: 170051059
Academic Year: 2021-2022
Date:

Md. Muftashin Muhim Bondhon
Student No: 170051070
Academic Year: 2021-2022
Date:

Shadman Rahman Sabab
Student No: 170051088
Academic Year: 2021-2022
Date:

DEDICATION

Our combined thesis work is dedicated to our respective parents. We also express our heartfelt gratitude to our respected supervisor Professor Dr. Hossain Md. Shahin for guiding us throughout the path with utmost sincerity as well as our parents who supported us throughout the journey.

ACKNOWLEDGEMENTS

"In the name of Allah, Most Gracious, Most Merciful."

All the praises to Allah (SWT) who has blessed us with the opportunity to complete this book. Our earnest gratitude towards our supervisor Professor Dr. Hossain Md. Shahin for giving us rightful instructions and for paying attention to us whenever needed throughout the research work. We are greatly indebted to him for enlightening us with his remarks and guidance to complete the thesis. We wish to show our gratefulness to ‘Prosoil Foundation Consultant’ for providing us the necessary data extracted from soil investigation that they have performed previously. We are much thankful to Tahsina Alam, Lecturer, CEE, IUT for her most valuable advice. We are thankful to Abdullah Sinha & Ahsan Ullah, M.Sc. students, CEE, IUT. We express our gratitude towards all of the departmental faculty members for their aid and support.

We also thank everyone who has contributed in any way to our work, as well as those who have sent words of encouragement, inspiration, and motivation. We are grateful for the help we have gotten during our project

ABSTRACT

The study is about establishing relationship between SPT N values, geotechnical parameters of soil & Angle of Internal Friction (φ), unconfined compressive strength (qu) for the region of Dhaka City & Sylhet region of Bangladesh using Machine learning technique. The relationship represents a formula for estimating Angle of Internal Friction φ & unconfined compressive strength qu from SPT N value. The relationship was formed previously by other researchers for regions of USA, Japan, Malaysia, UK and India. For Bangladesh a study was performed for areas of Joydevpur, Mymensingh, Jamalpur, Dhaka metro, Tangail & Madhupur. Though many countries have their own regional equation, for angle of internal friction (φ) finer content was not applied and for unconfined compressive strength (q_u) plasticity index was not applied in empirical equations, the study areas were also different. For this study About 300 samples have been collected from boreholes from Mirpur, Uttara, 300ft, and Purbachal area of Dhaka City & more than 400 samples were collected from Sylhet, Narsingdi, Habiganj, Brahmanbaria locations of Sylhet region. To develop the relation model & estimation linear regression, Multi Linear Regression (MLR), Structure Vector Regression (SVR) algorithms was used. According to R^2, RMSE & MSE value the best correlation is chosen among several combinations of N, N_{60}, $N_{1,60}$, depth & grain size data for Angle of Internal Friction (φ) and N, N_{60}, depth & plasticity data for unconfined compressive strength (qu). The best relation has later been compared with SVM model values where in some cases the MLR model comes out as better in terms of R^2, RMSE & MSE value and in some cases the SVM model comes out as better one in terms of R^2, RMSE & MSE value. Then the predicted values from selected MLR & SVM model have been compared with previously established empirical equations where the model shows better values of R^2, RMSE & MSE than previous established models. The better values from evaluation matrices indicates the better predicting ability of Angle of Internal Friction φ & unconfined compressive strength qu for the soils of Dhaka City & Sylhet region of Bangladesh.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 : INTRODUCTION

General

To its simplicity, the SPT, or Standard Penetration Test, is the most regularly utilized field investigation test throughout the world, including Bangladesh. It provides info about soil resistivity and qualities.

A soil's cohesive strength (C) is an important feature of its consistency. It refers to the cohesive force that exists between neighboring particles. Cohesion is defined in soil mechanics as "the shear strength (S_u) when the compressive stresses are equal to zero." We also considered the unconfined compressive strength (q_u) in this case. As Cohesion, we use half the value of q_u. The frictional angle is also an important element for determining soil lateral pressure and bearing resistance. Relationship between SPT-N & Shear Strength parameters of soil which helps engineers to adopt empirical methods and analyze soil performance.

Study Location

The samples were collected from two different zones of Bangladesh: - Dhaka & Sylhet. From Dhaka zone around 300 samples were collected from Kawlar, Purbhachal, Mirpur, 300 ft & Uttara. Dhaka zone Data's were collected from construction of Aga Khan Academy in Kawlar situated nearby of Shahjalal International airport & Ashkona, Geotechnical Survey for Excavation and Development (100'-0") Wide Khal Project which was conducted in the 300ft and Purbachal region of Dhaka, Dhaka Mass Rapid Transit Development Project Line 6 which covers the area of Uttara, Pallabi, Mirpur11, Mirpur10, Kazipara, Shewrapara, Agargaon, Bijoy Sarani, Farmgate, Kawranbazar, Shahbag, Dhaka University, Bangladesh Secretariat & Motijheel. From Sylhet zone about more than 400 samples were collected. Sylhet zone data were collected from Consultancy services for Development of Osmani International Airport Projects (Design Phase) which is located at 15 km northeast of Sylhet city, Geotechnical Investigation of Dhaka (Katchpur)-Sylhet-Tamabil Road (N2) which was located in Narsingdi, Brahmanbaria, Habiganj & Sylhet region.

From the collected samples of Dhaka zone for clay layer about 55% of samples were hard clay, 35% of samples were lean clay and other types of clays were 10%. For sand layer about 96% of samples were silty sands.

From the collected samples of Sylhet zone for clay layer about 87% of samples were lean clay & for sand layer about 99% of samples were silty sand.

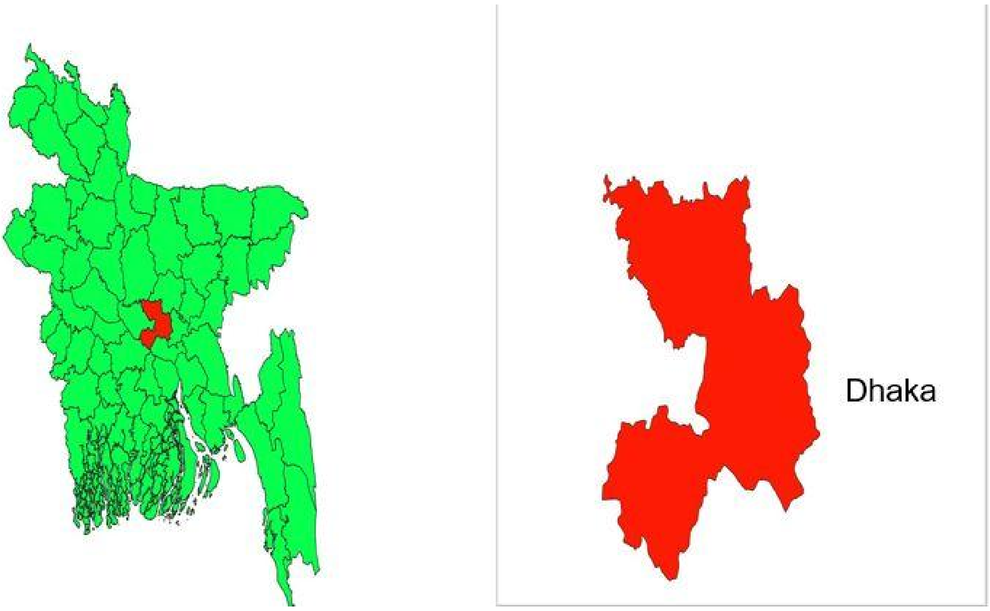


Figure 1: Dhaka study location generated by QGIS

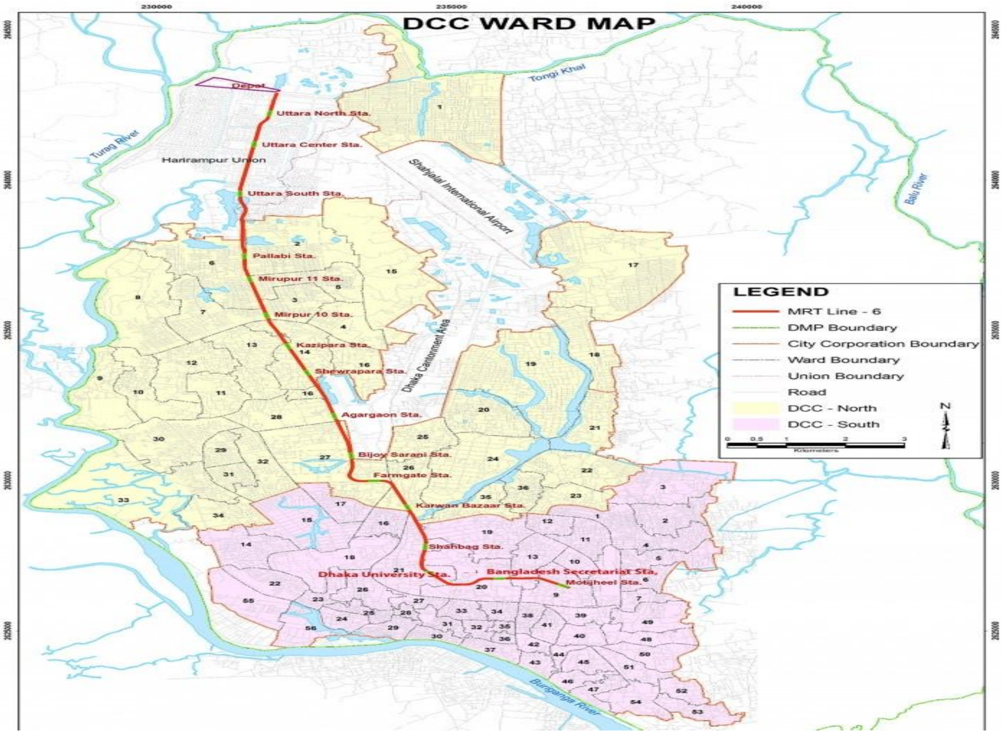


Figure 2: Dhaka study location of MRT Line 6 collected from Dhaka Mass transit company Limited.

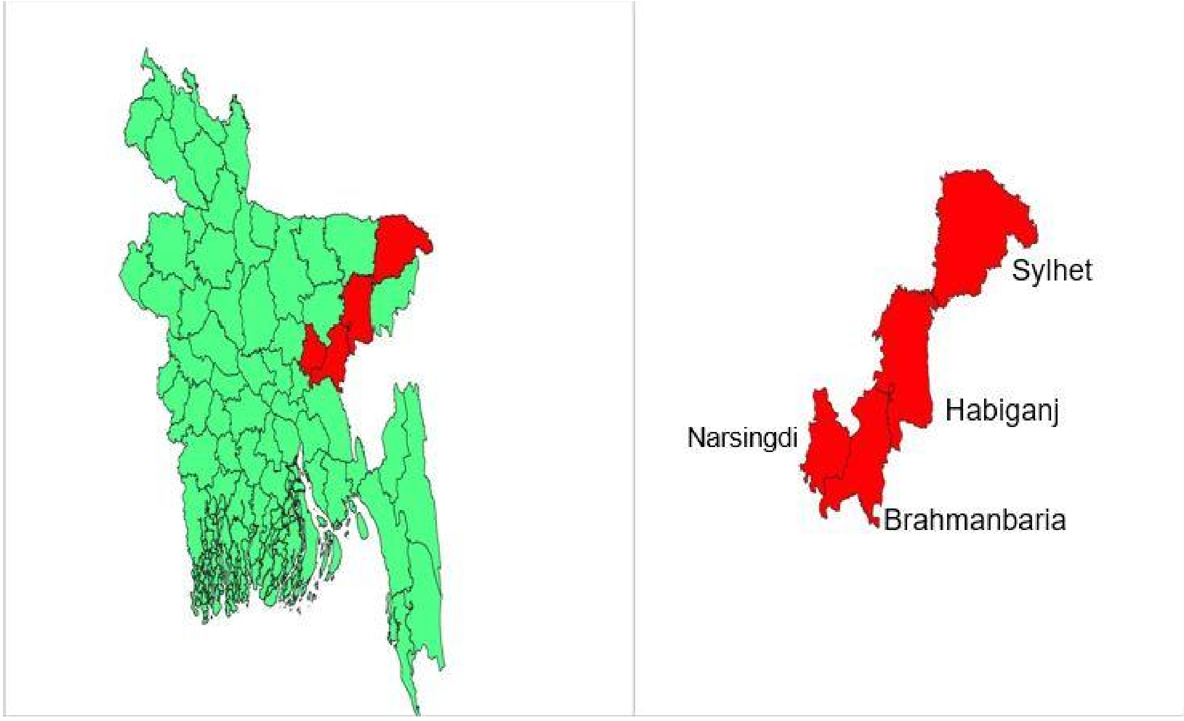


Figure 3: Study locations of Sylhet region

Background of the Study

Terzaghi introduced the SPT (Standard Penetration Test) in 1947 at the Texas Soil Mechanics Conference. It's commonly used to determine soil penetration resistance and correlate it with soil parameters including relative density, shear strength, bearing capacity, and liquefaction resistance. SPT has improved through time, by standardization-measurement of energy delivered to the drill rod from the hammer. Geotechnical studies and designs require the engineering properties of soils, which are calculated via in situ or laboratory tests. The connections should be such that in-situ soil strength may be anticipated using simple in-situ soil characteristics as SPT N value, water content, void ratio, relative density, shear strength, coefficient of compression, and so on. This will aid in the interpretation, verification, and reduction of the volume of a lengthy investigation program, or even the elimination of sub-soil investigation works with undisturbed sampling, as detailed soil investigation for each individual structure is not always possible or desirable, as it is laborious, time-consuming, and costly, and can sometimes cause difficulties for the investigator.

For many years, it has been mostly used in situ tests in Bangladesh. It is a low-cost and straightforward method for calculating relative density and shear angle. The unconfined compressive strength of cohesive soil is combined with the resistance of cohesion-less soils. The test is carried out in a borehole drilled to desirable sampling depth. A split spoon sampler is connected to the drill rod. This type of sampler is used to obtain disturbed samples.

These are based on the differences of operating technique, energy loss or friction. In this case, as the soil counters blow, thus becoming disturbed. The split spoon sampler which is penetrated through the layers of soil by specific blows by a hammer from at a height of 76 cm. Usually donut hammers of 63.5 kg are used. Besides, there are uses of other types of hammers as well. To the hole, the split spoon is bottom-lowered and after that driven 450mm (18 in.) into it, with the blows tallied, usually every 76mm (3 in.) of penetration. After driving, the split spoon is removed from the hole's base and the sample is stored in an airtight container. The number of blows required to drive the split spoon for the final 300mm (1ft) of penetration is the penetration resistance (N). Because the soil is believed to have been disturbed by the boring operation, the first 150 mm (6 in.) of penetration is excluded.

The SPT (Standard Penetration Test) (ASTM D 1586-99, 2006) is carried out in Bangladesh, as subsoil inquiry is the sole option with an in-depth investigation. Which is accompanied by only essential projects that are subjected to laboratory tests engineers in practice. Depending on SPT field test data and SPT-N value correlations with diverse soils parameters. Clients are sometimes hesitant to spend additional money on subsoil investigation. They will solely pay for the SPT field test. As a result, it's critical to understand the relationships between SPT-N and clay soil parameters so that SPT-N may be utilized to build foundations in soft clay with confidence. Several researches have been conducted on the soils of various Bangladeshi locales, including Serajuddin (1996), Bashar (2000), Ferdous (2001), and Munshi (2001) are some of the most well-known figures in the world. Several connections were discovered between soil parameters.

Three significant constraints plague the majority of the SPT team in Bangladesh are:

- Uncontrolled hammer fall height
- SPT Spoons that aren't standard
- For undisturbed sampling non-Standard Shelby Tube.

There is less reproducibility of field-testing data that limits those relationships. Because, gathered data from numerous sources without aiming for any research purpose. As a result, the correlation between SPT-N value and clay shear strength was established. Using inaccurate field data could result in erroneous correlation. This research was about SPT-N value and others parameters correlations.

Objectives of the Study

The following measures were used to complete this study:

i. To reduce high costing rates of laboratory Test
ii. To avoid unavailability of test specimen’s and insufficient laboratory facilities

iii. To deal with undisturbed soil samples that cannot be collected from all layers.
iv. To establish relationship between SPT-N & shear strength parameters q_u & Ø
v. To develop relationship between N_{60}, $N_{1,60}$, finer%, depth, q_u, Ø

Methodology

To achieve the objectives of this study following methodology will be adopted:

1. Field investigation:

Standard penetration test (SPT) following ASTM D1586 will be conducted at the selected locations

The test will be conducted using the wash boring technique. Up to 15 to 30 m from existing ground level, every 1.5 m depth interval, the SPT N-value will be recorded (EGL). A 623 N (140 lb.) hammer will be dropped from 0.762 m to pound the sampler into the earth (30 in). The number of blows required to drive the sample through three 0.15 m (6 in) intervals will be kept track of. The total number of strikes required to drive the last two 0.15 m (6 in) intervals is the SPT-N value. During drilling, disturbed and undisturbed samples will be collected.

2. Laboratory test:

For proper interpretation of soil characteristics following laboratory tests will be performed:

- Atterberg limit test,
- Specific gravity,
- Grain size distribution,
- Water content,
- Unit weight,
- Unconfined Compression Test

3. All samples will be classified according to ASTM D-2487's Unified Soil Classification System (USCS).
4. To compensate for changes in SPT blow count, field SPT-N data will be modified for dilatancy, adjustment for field procedures, and overburden.
5. Presently available correlations between Cohesion (C) and SPT-N of Terzaghi & Peck (1967), Sanglerat (1972), Stroud (1974) which included Plasticity Index, Sowers (1979), Nixon (1982), Sirvikaya & Toğrol (2002), Nandita rani saha (2013) for Bangladeshi soil and M. Serajuddin (1996) etc. empirical equations based on Liquid limit, Plasticity Index, Corrected SPT-N and Soil type will be checked for validation in Bangladeshi soft soil.
6. Multiple Linier Regression (MLR) and Support Vector Regression (SVR) analysis will be carried out to establish the correlation between Cohesion (C), SPT-N, Plasticity Index and fine content. N_{60}, $N_{1,60}$, finer%, depth, q_u, Ø and SPT-N & shear strength parameters q_u & Ø.

Angle of internal friction:

Angle of internal friction (ϕ) can be defined as a measure of a unit of soil's capacity to sustain shear stress. ϕ is an important factor in defining the frictional resistance of a pile, the slope stability of an earthen slope, and other geotechnical engineering design factors. It's used in finite element modeling to define constitutive soil (Mohr-Coulomb, Modified Cam-Clay, etc.) utilizing Critical State Soil Mechanics (Andrew and Wroth 1988) and define soil-structure interaction (FEM). On undisturbed soil samples, laboratory techniques called direct shear tests and triaxial tests are used to assess soil shear strength characteristics. Nonetheless, it is an expensive technique, and undisturbed soil samples from all levels cannot be acquired. In this case, estimating 0 from in-situ testing utilizing empirical equations may be a realistic option.

To begin, recognize that the circle shown below is determined by the confining stress point as well as the applied stress point. The distance between adjacent places is always the circle's diameter. The circle expands in bigger as the distance increases. As you can see, the circle is quite tiny for an applied stress that is very near to the confining stress. The circle grows in size as the applied tension increases. As the applied stress grows, the circle will finally hit the failure envelope line, indicating that the soil has failed.

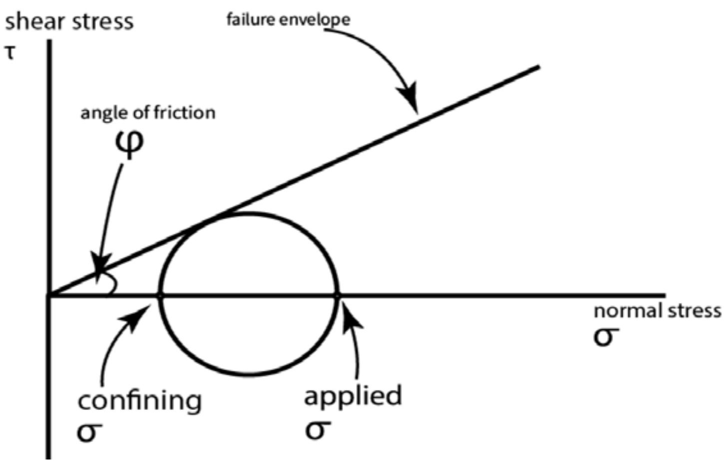


Figure 4: Shear Stress vs. Normal Stress

Surprisingly, as shown above, the friction angle of the soil dictates the inclination of the failure envelope. Obviously, the angle of friction is an important factor in a soils shear strength.

Unconfined Compressive Strength (q_u):

Maximum axial compressive stress that what a right-cylindrical specimen of material can bear under unconfined conditions is defined as the unconfined compressive strength. The unconfined compressive strength (qu) of a cohesive soil is the load per unit area towards which the cylindrical sample falls under compression.

$q_u = P/A$

here P= axial load at failure.

Corrected area $= \frac{A}{1-\varepsilon}$

Here A is the initial area of the sample

Ɛ= change in length/original length.

Undrained shear strength (s) of the soil is equivalent towards the one half of the unconfined compressive strength, $s = q_u/2$

Since it is the quickest & inexpensive methods of assessing shear strength. The unconfined compression experiment is commonly applied soil shear testing method. This is the most commonly employed to regain saturated, cohesive soils using thin-walled sample tubes. This experiment is ineffective on cohesionless or coarse-grained soils.

The strain is regulated in the unconfined compression experiment. While the soil specimen is loaded in a hurry, the pore pressures change in a way that does not allow them to spread. As a result, this is representative of soils on construction areas. Because of the quick rate of growth, pore liquid having no time to spread.

% Finer:

The soil under this sieve is a finer section of the specimen, with particle sizes smaller than the apertures of this sieve. The entire weight of this section, stated as a percentage of the whole soil sample, is the % finer of this sieve.

$$\% \text{ Finer} = \frac{W_5 + W_6 + W_{pan}}{W} \times 100$$

This can be easily calculated by simply removing cumulative percentage from the 100 % of soil sample

$$\% \text{ Finer} = 100 - \text{Cumulative \% retained}$$

Atterberg limit:

Atterberg's limitations contain the liquid limit, the plastic limit & the plasticity index.

Liquid limit:

The liquid limit is the moisture content where the groove made with a standard tool into a standard cup specimen of soil closes for 10 mm after 25 blows in a standard way. The soil has a lower shear strength at this limit. The liquid limit is the water concentration towards which the soil starts to function as liquid. To establish the liquid limit, a clay specimen is settled into a standard cup & a groove is formed with a spatula. Cup is lowered till the gap is gone. This sample is being applied to calculate the water content of the soil. This experiment should be restarted with greater water content. Soil with a low water content would produce more blows, whereas soil with a great moisture content would produce fewer blows.

Plastic Limit:

A soil's plastic limit is the moisture content where the soil commences to function like that a plastic substance. However, at water concentration, the soil crumbles while winding up 3.2mm (1/8in) threads. This article will give methods for evaluating plastic limit of soil in ASTM D 4148. Plastic limit analysis is concerned with determining the "strength" of a particular construction. It calculates the factor whereby the live load element must be increased in order for a physical crisis to occur in the form of plastic failure.

Plasticity Index:

The PI is defined like that moisture content range across whereby the soil deforms plastically. Thus, the PI is described as the variation between the LL and the PL.

PI= LL-PL

Thus, the PI is assessment of a soil's flexibility. As a result, the PI identifies the quantity and kind of clay in a soil.

• Commonly, clay-based soils have a high PI.

•That with a lower plasticity index likely to be silt, whereas that with a plasticity index close to zero have hardly any silt or clay (fines).

The plasticity index reflects, among other things, how much moisture must be removed from a soil to convert it from a liquid to semisolid state. It determines moisture limit towards which a soil becomes plastic. The plasticity index may be regarded of as a measure of the cohesion of a soil.

CHAPTER 2 : LITERATURE REVIEW

Introduction

The literature review contains the correlation of SPT-N with Cohesive Strength (C)/Unconfined Compressive Strength (q_u) & Friction angle (φ) in Bangladesh Region. To get the information on geological properties of soil The Standard Penetration Test (SPT) is done. In 1927, the Raymond Concrete Pile Company used a split barrel sampler to establish the Standard Penetration Test. The SPT has been performed all over the world since then. It is widely used in estimating the In Situ properties of Granular soil. Cohesive strength(C) of a soil is a crucial aspect of soil consistency. It refers to the cohesive force that takes place between adjacent particles. Again, in soil mechanics, cohesion means "the shear strength (Su) when the compressive stresses are equal to zero". Here we have taken the unconfined compressive strength (q_u) also in consideration. We are taking the half value of qu as Cohesion (C). The frictional angle (φ) is also a crucial parameter for its uses in finding the lateral pressure and bearing resistance of soil. For finding out these important soil parameters laboratory tests are required. Like Unconfined compressive strength test and triaxial test is related to finding the value of Cohesion (C). Again, by direct shear test we can get the value of internal friction angle (φ). These tests are not always available for their high costing rates, unavailability of test specimen's and insufficient laboratory facilities and may other issues. So empirical formulas have been used to find out these important parameters by correlating the relations of soil parameters with SPT N values which is always available in every soil taste. In past many other countries like Japan [2], Iran [5 & 10], Malaysia [12], Canada [11], Turkey [9] and Vietnam [2] have established their own correlations for their regions. In Bangladesh region M. Serajuddin & M. A. Alim Chowdhury at 1996 [3] and Nandita Rani Saha at (2013), A. Hossain, T. Alam, S. Barua & M. R. Rahman (2021) [26] has established correlations but they have some limitations in them. We have observed that the values we got from laboratory tests differs in a significant margin with the values we got from using empirical formulas of correlation provided by previous research works. The main reason behind that is the formulas were not generated for our country region and the soil property and behavior differs in every region respectively. As Bangladesh is now in a rapid phase of development so many upcoming projects need these important parameters which cannot be always found out by laboratory tests. So, for finding those parameters accurately by empirical formulas we are trying to establish a correlation for Bangladesh region in our research work.

Experiments

Soil parameters were obtained through various experiments.

Boring and Sampling

The borehole was made by the Percussion Method. Disturbed samples were collected during Standard Penetration testing at 1.5 m intervals mainly (ASTM D 1586). A total of twenty (20) boreholes were drilled in the proposed project shown in Appendix–A1 Borehole Location Plan.

The subsoil within the vicinity of the borehole consists mainly are shown in the bore log. The borings were drilled vertically through soil approximately 15.0 to 26.0 meters deep.

Undisturbed Samples

Undisturbed soil samples were collected from boreholes under Site Engineer supervision. A 75mm O.D thin-walled UD sampler with length 1000mm was used. After removing the sampling, the length of the recovered specimens was measured, documented, as well as the recovery ratio was calculated. While sampling below the water table perhaps after cleaning the casing with water, the water table was kept at the top of the casing till the sampler was withdrawn. To get a jar specimen from the top and bottom of the tube, a maximum of 50mm of undisturbed material was extracted from each. Following specimen preparation, all ends of the specimen were covered with a non-shrinking wax to guarantee an airtight seal.

Standard Penetration Test

Standard Penetration Tests (SPT) were performed to determine the consistency of the soil as well as to gather disturbed specimens for visual examination and lab testing. The number of blows necessary for 12-inch (300mm) penetration measured just after seating drive of 150 mm is known as the N value. A free fall hammer was employed in the Standard Penetration Test (SPT). This mechanism was made out of a hollow cylindrical mass that slid over a steel rod. It works by raising the bulk using a wire. When the bulk reaches the proper height (760 mm), it is automatically released, forcing the split spoon into the dirt. While collecting the disturbed samples gathered from the split spoon sampling in a plastic zipper bag, they were visually evaluated.

Unconfined Compression Test

The unconfined compaction is performed in accordance with ASTM D2166. This test measures the unconfined compressive strength of cohesive soil in its natural, remolded, and compacted state. It is accomplished by pressing cylindrical specimens until failure, with failure occurring whenever the shear stress to shear strength ratio reaches its maximum.

Direct Shear Test

The ASTM D 3080 direct shear testing is used to assess the shear strength of topsoil or any discontinuity in soil or rock masses. It is carried out on three or four specimens from an undisturbed soil specimen. A sample is placed in a shear box, which includes two stacked rings to retain the sample; the contact between two rings is about at the sample's mid-height. A confined tension is supplied to the sample vertically, and the upper ring is dragged laterally until the sample fails or passes through a defined strain. The maximum force and strain are recorded at regular intervals to create a stress–strain curve for each confining stress.

Tri-axial Test (UU)

A cylindrical piece of soil contained in an impermeable membrane is confined pressured and then forced horizontally to failure in compression in the Tri-axial Compression Test, ASTM D 2850-95. In a tri-axial chamber, the samples are subjected to restricting fluid pressure. By spinning the knob, the cell pressure is increased to a predefined amount, and the sample is pushed to failure by raising the vertical stress by maintaining a single rate of axial strain. Such testing is normally performed on three identical specimens exposed to varying confining forces. Because all specimens are presumably saturated, the shear strength is consistent throughout all experiments. The data are presented as primary stress difference vs strain curves. For greatest primary conditions variation in maximum main stress Mohr circles were displayed as a function of total stress.

Atterberg Limit test

The Atterberg limits test would be a fine-grained clay and silt various soil testing that detects the moisture content during which particles transition among phases. The Atterberg limits test is performed in accordance with ASTM D 4318-00 on the fraction of soil that will enter through a No. 40, 425m, or 0.425mm sieve. The experiment aids in the classification of soil, the plasticity qualities, as well as the assessment of near-surface soil shrink/swell potential. It can be used to differentiate between the various types of silt and clay, and to determine the shrinkage limit (SL), plastic limit (PL), and liquid limit of a soil specimen (LL). The moisture content of clay soil affects its firmness and behavior. The soil can be in one of four stages depending on the moisture content: solid, semi-solid, plastic, or liquid.

Parameters

Several parameters were considered to establish the correlation. Soil characteristics, plasticity index, grain size value etc. were considered.

Soil Classification

Usually determined by grain size and soil consistency. There are a few categorization systems. Casagrande created the Unified Soil Classification Method in 1942, which was later refined and accepted by the United States Bureau of Reclamation and the United States Army Corps of Engineers. The method is now employed in almost all geotechnical operations.

Here we follow UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

USCs Divided into two broad categories:

• Coarse Grained Soils Gravels (G) and Sands (S)< 50% passing through #200 sieve (i.e., >50% retained on #200 sieve)

• Fine Grained Soils Silts (M) and Clays (C) ≥ 50% passing through #200 sieve

Criteria for assigning group symbols and group names using laboratory tests[a]				Soil classification: Group symbol	Soil classification: Group name[b]
Coarse-grained soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[c]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[e]	GW	Well-graded gravel[f]
			$C_u < 4$ and/or $1 > C_c > 3$[e]	GP	Poorly graded gravel[f]
		Gravels with Fines More than 12% fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[d]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[e]	SW	Well-graded sand[i]
			$C_u < 6$ and/or $1 > C_c > 3$[e]	SP	Poorly graded sand[i]
		Sand with Fines More than 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
Fine-grained soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A" line[j]	CL	Lean clay[k,l,m]
			PI < 4 or plots below "A" line[j]	ML	Silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OL	Organic clay[k,l,m,n] / Organic silt[k,l,m,o]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[k,l,m]
			PI plots below "A" line	MH	Elastic silt[k,l,m]
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$	OH	Organic clay[k,l,m,p] / Organic silt[k,l,m,q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

Figure 5: unified soil classification chart (after ASTM,2011) (Based on ASTM D2487-10: Standard practices for classification of soils for Engineering purposes (Unified Soil Classification).

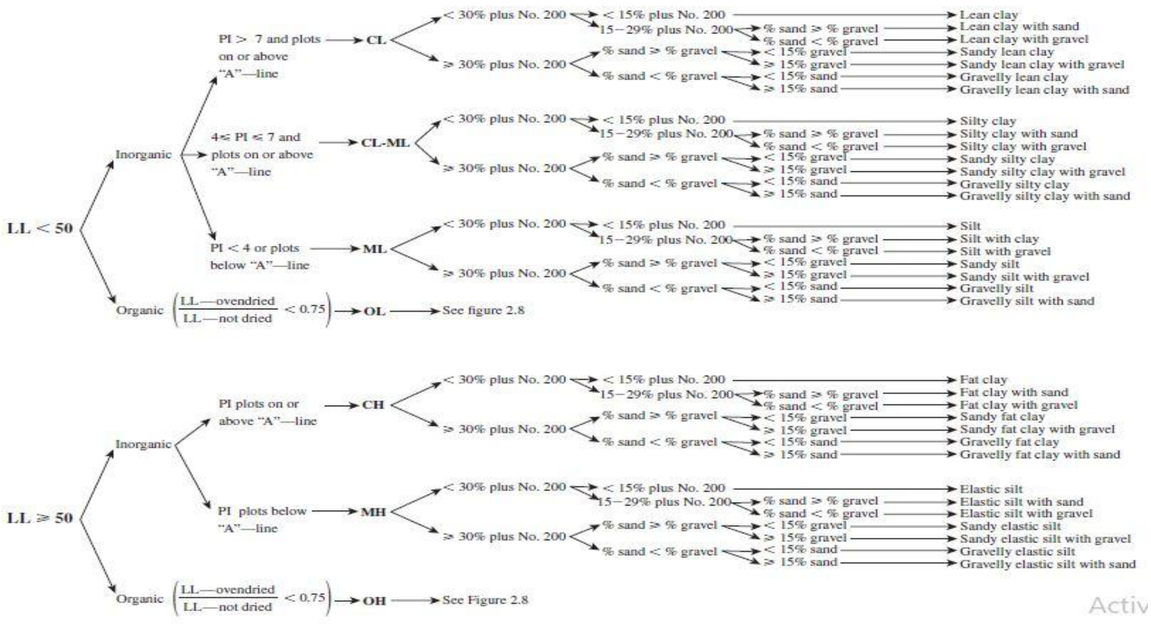


Figure 6: Flowchart for classifying fine grain soil (based on ASTM D2487-10: Standard practice for classification of soils for Engineering purposes (Unified Soil Classification).

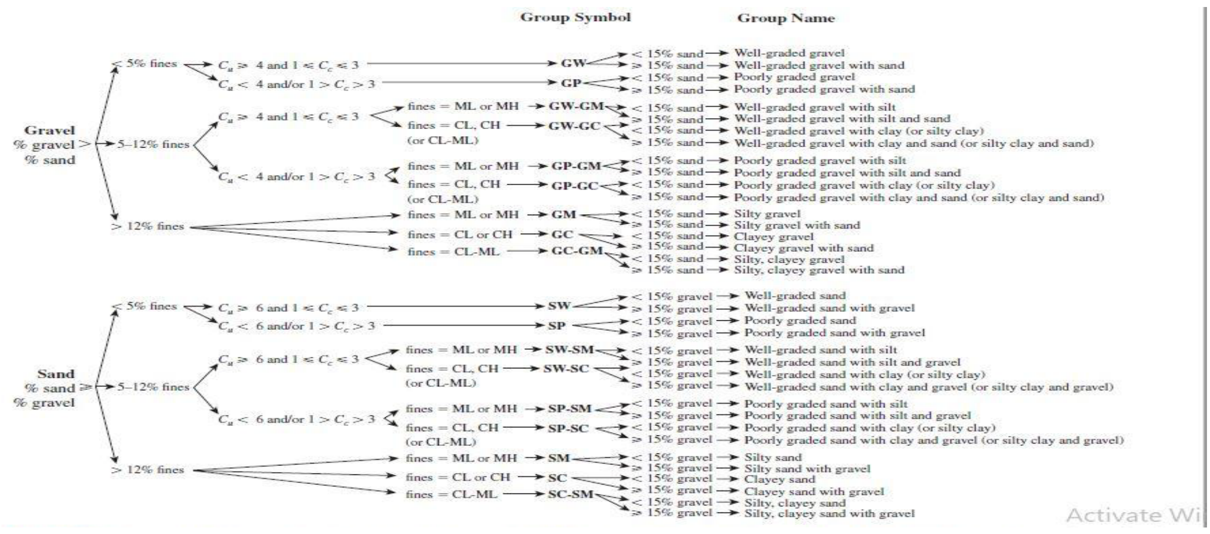


Figure 7: Flowchart for classifying coarse grained soils (based on ASTM D2487-10: Standard practices for classification of soils for engineering purposes (unified Soil classification)

Correction of Standard Penetration Test (SPT)

SPT Value Correction for Field Methods Based on field experiences, it is plausible to standardize the field SPT value as a consequence of the driving energy input and its dispersion around the sampling and surrounding soil. Differences in testing methodologies can be corrected for at least partially by translating the measured N to N60 as follows.

Were

$$N_{60} = (E_H\ C_B\ \ C_S\ C_R\ N)/.60$$

N_{60} = Corrected SPT N-value for field procedures

E_H= Hammer efficiency

Hammer Energy Efficiency Correction (C_E):

SPT N values should be converted to an equivalent standard penetration resistance

(N) equivalent to a delivered energy of 60%, using

C_B= Borehole diameter correction

It required when inner diameter is 150mm (6 inches) or greater. Large inner diameters of boreholes reduce confinement making it easier for spoon to penetrate soil.

C_S= Sampler correction

C_R= Rod length correction

N= Measured SPT N-value n field

If soil type is cohesive then it refers as N_{60}(corr)

Correction of SPT value for over burden pressure

The application of the SPT correction factor is frequently perplexing. Field method corrections (Energy Adjustment) are always suitable, however overburden pressure adjustment may or may not be makeup on the processes adopted by those who devised the analytical technique under discussion. Overburden pressure adjustment is not required for cohesive soil. Initially, overburden pressure is corrected for soils with low cohesion.

$(N1)_{60}= C_N \times N_{60} \leq 2\ N_{60}$

C_N = overburden pressure correction factor

Then if it is fine sand or silt under water table with N value >15, *dilatancy (water table) correction* is made. For coarse sand dilatancy correction is not required

Correction of SPT Value for Water Table

In additional to overburden adjustments, scientists proposed SPT-value adjustments for water table in the event of fine sand or silt below the water table. Owing to the dilatancy effect, high N-values may be seen, especially whenever the value obtained is more than 15.

$N_{1,60}$= 15+0.5(N'-15)

Liquid limit

The liquid limit is the water concentration at which the soil begins to act like a liquid. It is calculated by putting a clay specimen on a standard cup and separating it with a spatula. The cup is placed until the separation disappears. The water content of this specimen can be determined. The experiment is done with the water content increased. Soil with a low water content would produce greater blows than soil with a high moisture content. Geotechnical Engineering Calculations and Rules of Thumb (Second Edition), Ruwan Rajapakse PE, CCM, CCE, AVS, 2016.

Plastic Limit

Plastic limit is defined as the water moisture content at which a thread of soil with 3.2mm diameter begins to crumble. Offshore Pipelines (Second Edition), 2014

Effective Stress:

It is the stress transferred via grain to grain at the contact point by soil mass and is indicated by'. Whenever a soil mass is pressed, the weight is transmitted to the soil grains via their point of contact. Whereas if effective load at the point of contact is higher than the resistance of the grains, the soil mass will compress. This is owing to elastic compression of the grains just at site of contact, and also relative particle sliding. Effective stress is the load per unit area of soil mass that is prone to soil mass displacement.

Cohesive strength(C) of a soil is a crucial aspect of soil consistency. It refers to the cohesive force that takes place between adjacent particles. Cohesion means "the shear strength (Su) when the compressive stresses are equal to zero". Here we have taken the unconfined compressive strength (q_u) also in consideration. We are taking the half value of q_u as Cohesion (C).

The frictional angle (φ) is also a crucial parameter for its uses in finding the lateral pressure and bearing resistance of soil.

Existing Correlation between SPTN & Cohesion C

(1) First study to determine the relationship between C and SPT-N was done by Terzaghi & Peck (1967). They proposed following equation for fine grained soil

$C = 6.25\ N$

Where, C = Cohesion in KPa

(Here N=field SPT N value for limited range)

	q_u (kPa)					
Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
N_{60}	<2	2–4	4–8	8–15	15–30	>30
q_u	<25	25–50	50–100	100–200	200–400	>400

Figure 8: Relation of Consistency of Clay, Number of Blows N60 on Sampling Spoon and Unconfined Compressive Strength

(2) In 1971 Hara proposed a relation between SPT N & Cu using atmospheric pressure Pa.

$C_u = Pa * 0.29 N_{60}^{0.72}$

Pa= Atmospheric pressure (100 KN/m^2)

C_u= Undrained shear strength or Cohesion kPa.

(3) Sanglerat (1972) was the first researcher who presented C and SPT-N correlation according to the type of fine-grained soils

$C = 12.5\ N$, For Clay

$C = 10\ N$, For Silty Clay

Where, C = Cohesion in kPa

Here corrected SPT-N $N_{1,60}$ was used.

$N_{1,60} = 15+0.5(N'-15)$

(4) Stroud (1974) examined different relationships and proposed correlation in terms of Plasticity Index (PI) for the first time.

This research shows increase in Plasticity Index decreases the value of Cohesion (C)

C = (6-7) N, When PI<20
C = (4-5) N, When 20<PI<30
C = 4.2 N, When PI>30

Where, C = Cohesion in kPa
and PI = Plasticity Index (Here N=field SPT N value for limited range)

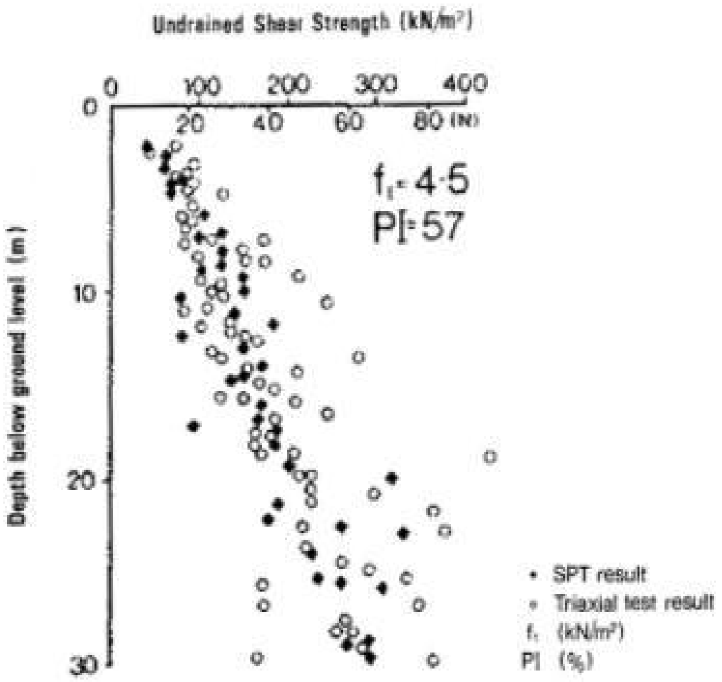


Figure 10: plot of Cu vs SPT-N with depth for 'London clay site 6'

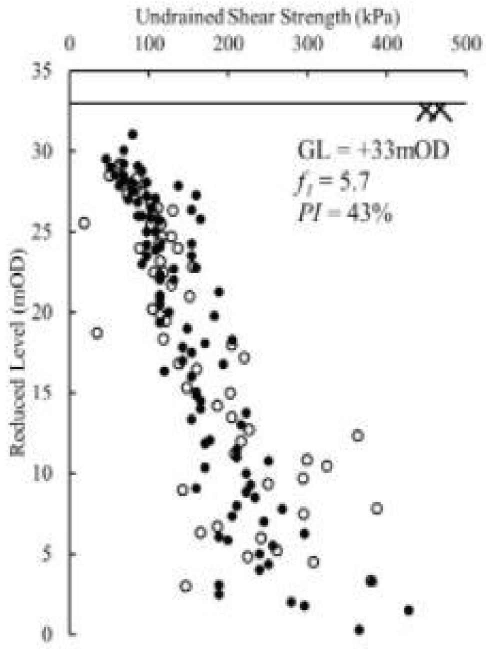


Figure 9: plot of Cu vs SPT-N with depth for f_1=5.7, PI=43%

Table 2. Summary of f_1 and PI (%) for all sites.

Site*	f_1 (by eye)	f_1 (proposed method)	PI	Location
Site 1 (B)	5.7	5.4	45	Finsbury
Site 1 (A3)		5.1	43	Park
Site 1 (A2)		6.6	42	
Site 2 (B)	5.5	5.3	51	Pimlico
Site 2 (A3)		5.5	44	
Site 2 (A2)		5.6	42	
Site 3	5.0	4.5	41	Victoria
Site 4	5.5	6.0	45	W Brompton
Site 5	6.1	7.0	44	Soho
Site 6	5.6	6.4	46	Liverpool St
Site 7	5.8	6.4	46	White City
Site 8	6.5	7.2	46	Earls Court
Site 9	5.8	5.5	45	NWR
Average	5.7	5.9	45	All sites

*London Clay unit shown in brackets where assessed.

Figure 11: Summary of f_1 7 PI % for all sites

(5) The results of Sowers (1979) shown that C increases with increase in plasticity index.

C = 12.5 N, For Highly plastic soil
C = 7.5 N, For Medium plastic soil
C = 3.75 N, For Low plastic soil

Where, C = Cohesion in kPa

(6) Nixon (1982) proposed a general equation for all Clay soil.

C = 12 N

Where, C = Cohesion in kPa

(Here N=field SPT N value for limited range)

(7) Decourt (1990) proposed a relation between SPT-N & N_{60} & Cohesion kPa for S. Paul o over consolidated, insensitive clays

Cu= 12.5 N kPa.

Cu= 15 N_{60} kPa.

(8) In 1996 M. Serajuddin & M. A. Alim Chowdhury studied to find the correlation between SPT-N & Unconfined compressive strength (q_u) of Bangladesh soil deposit

Total of 420 soil sample was collected from different locations of Bangladesh especially from Dhaka metropolitan, Tangail and Modhupur.

Atterberg limit test, Sieve analysis & Unconfined Compression test was done.

The liquid limit and plasticity index for all the soil samples has been plotted in the Casagrande plasticity chart to define the classification of soil sample by USCS.

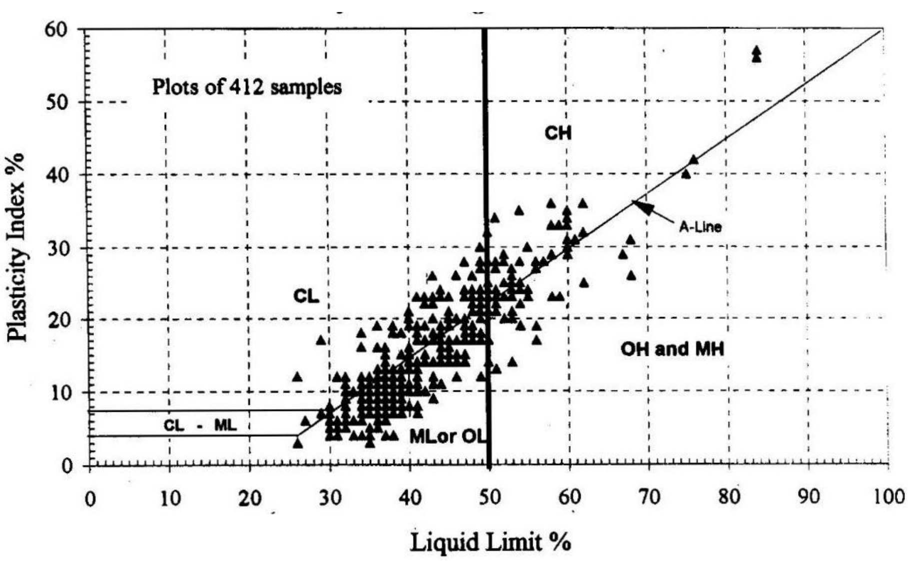


Figure 12: Plot on Casagrande plasticity chart to define the classification of soil sample by USCS

At first N vs qu for all the soil samples has been plotted to find an average K (Approximate proportionality factor) for all type of soil.

From the graph researcher found the equation as q_u = KN
Where, K=16.8

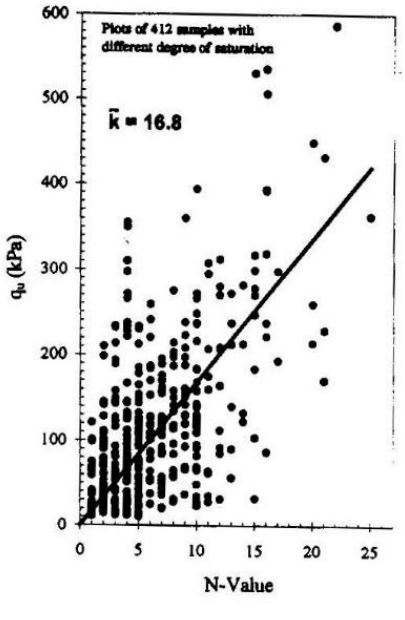


Figure 13: Correlation between qu and N (for 412 soil sample of different degree of saturation)

After that soil sample of different degree of saturation was plotted to find K value for different degree of saturation.

From the graph's researcher found the equation as q_u = KN
Where, K=16.5 for degree of saturation is 95-100%
and, K=15.8 for degree of saturation is 100%

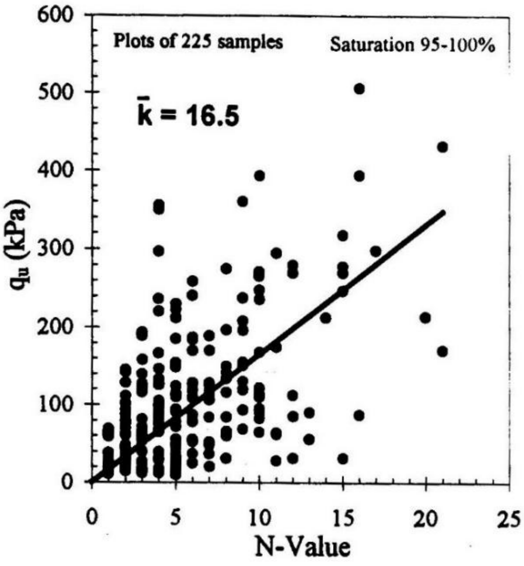


Figure 14: Correlation between q_u and N (for different degree of saturation)

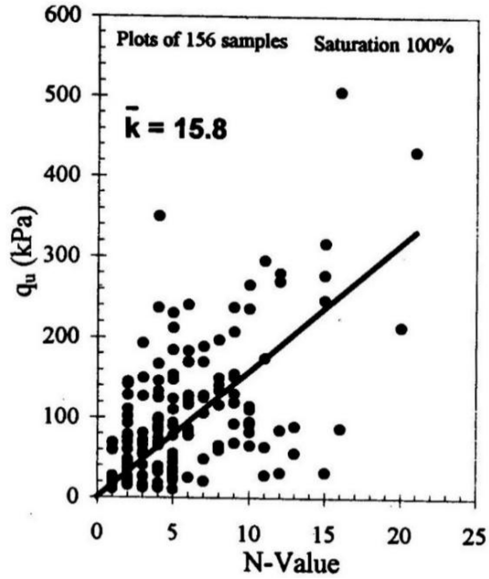


Figure 15: Correlation between q_u and N (for different degree of saturation)

Finally soil sample of different range of liquid limit was plotted to identify the K value for a range of liquid limit.

From the graphs the equation was found as, $q_u = KN$
Where, K=14.3 for Liquid limit <= 35%
K=16.9 for Liquid limit = 36 to 50%
and, K=17.8 for Liquid limit >= 51%

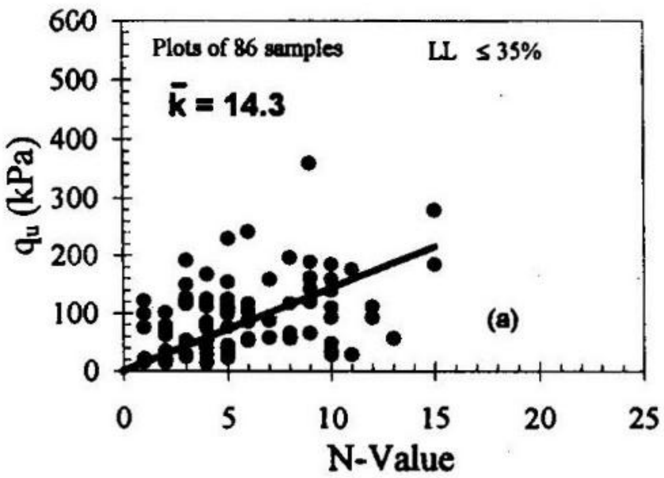


Figure 16: Correlation between q_u and N (for Liquid limit range)

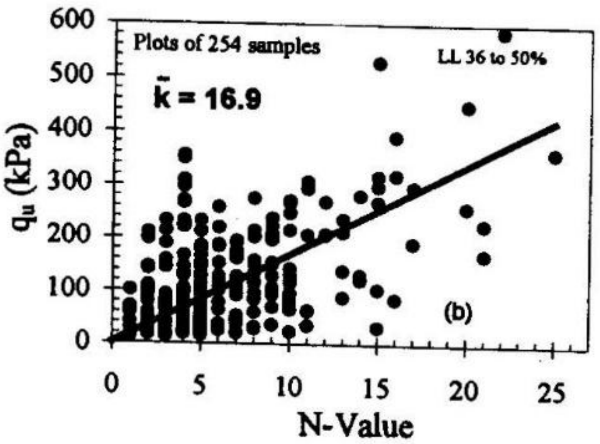


Figure 17: Correlation between q_u and N (for Liquid limit range)

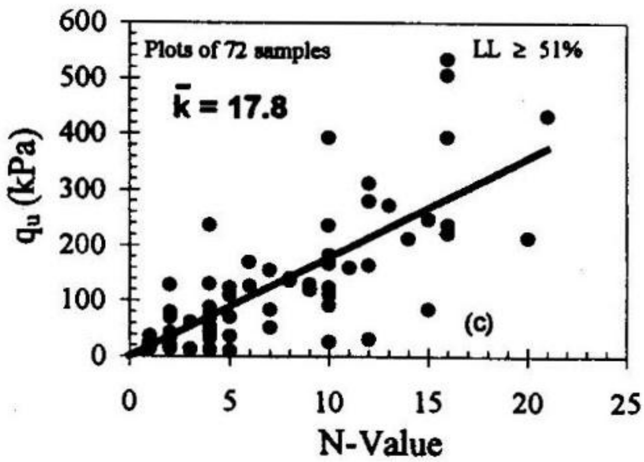


Figure 18: Correlation between q_u and N (for Liquid limit range)

(9) Sirvikaya & Toğrol (2002) made a wider study on different fine-grained soils using results of UCS experiment and presented new correlation.

$S_U = 5.50\ N_{field}$ for Highly plastic clay
$S_U = 7.80\ N_{60}$ for Highly plastic clay
$S_U = 3.70\ N_{field}$ for Low plastic clay
$S_U = 5.35\ N_{60}$ for Low plastic clay
$S_U = 4.75\ N_{field}$ for Clays
$S_U = 6.90\ N_{60}$ for Clays
$S_U = 4.45\ N_{field}$ for Fine grained soil
$S_U = 6.35\ N_{60}$ for Fine grained soil

Where, S_U = Undrained shear strength in KPa

(Here N_{field} =field SPT-N value)

(Here N_{60} =Corrected SPT-N value for 60% hammer efficiency)

N" = Corrected value of SPN-N

Soil types and numbers analyzed for relations between SPT-N and S_u

Soil type	Number of data (n)		
	UC	UU	FV
Highly plastic clay (CH)	113	80	13
Low plastic clay (CL)	72	67	11
Clay	185	147	24
Fine-grained soil	226	190	62

Figure 19: soil types & numbers for relation between SPT-N & Su

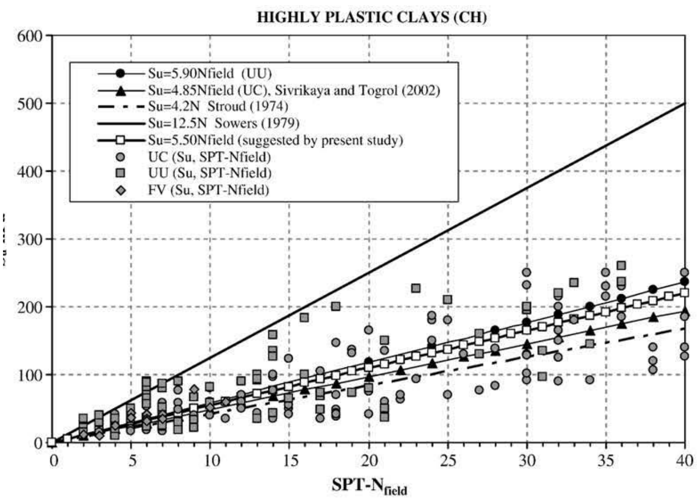


Figure 20: Correlation between Su and SPT-N & comparison with previous data for Highly plastic clay

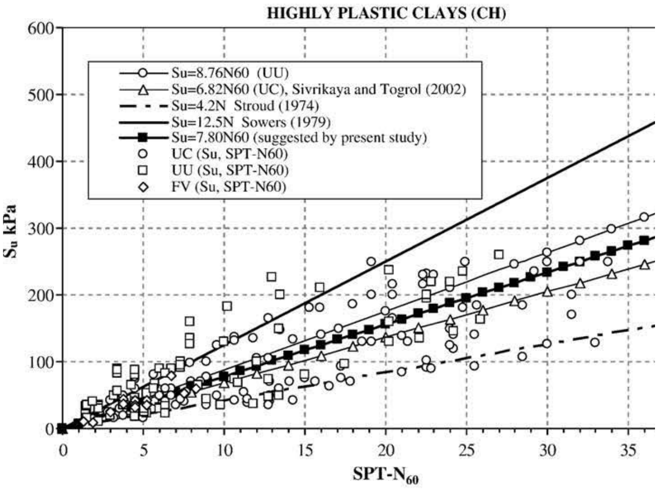


Figure 21: Correlation between Su and SPT-N & comparison with previous data for Highly plastic clay

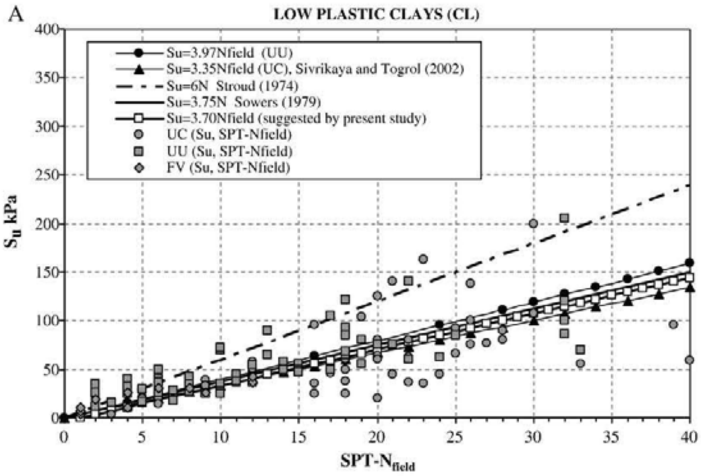


Figure 22: Correlation between Su and SPT-N & comparison with previous data for Low plastic clay

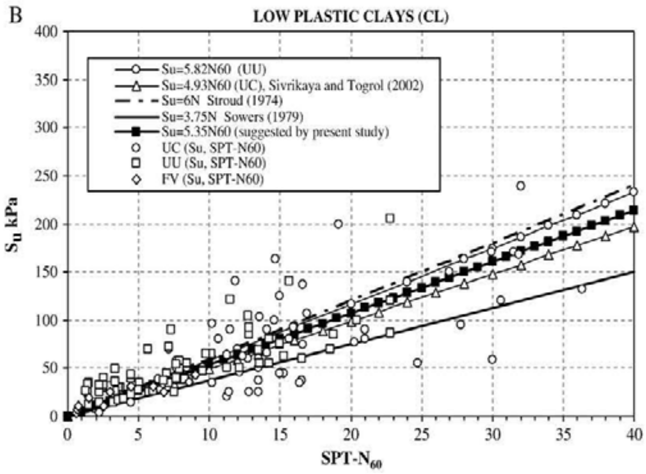


Figure 23: Correlation between Su and SPT-N & comparison with previous data for Low plastic clay

In first phase of analysis simple linear regression analysis was performed to establish a linear equation between Su and SPT. The following equation was found:

$Su = 1.6\ N_{field} + 15.4\ (r=0.72)$
$Su = 2.1\ N_{60} + 17.6\ (r=0.72)$

In the second phase the parameters of natural water content (Wn), liquid limit (LL) and plasticity index (PI) considered as independent effective parameters in addition to N (SPT)

$Su = 1.5\ N_{field} – 0.1\ Wn – 0.9\ LL + 2.4\ PI + 21.1,\ (r = 0.8)$
$Su = 2\ N_{60} – 0.4\ Wn – 1.1\ LL + 2.4\ PI + 33.3,\ (r = 0.81)$

Where, Su = Undrained shear strength (kPa)
Nfield = Field SPT value
N60 = Corrected SPT value for 60% hammer efficiency.

r = regression co-efficient

Regression analysis was performed with a total of 26 data available from the tests. The following equation was found:

C_U (kPa) = 8.42 N_{60} (R^2 = 0.80), for Silty clay till
C_U (kPa) = 8.22 N_{60} (R^2 = 0.34), for Clayey silt till
C_U (kPa) = 8.32 N_{60} (R^2 = 0.79), for All soil

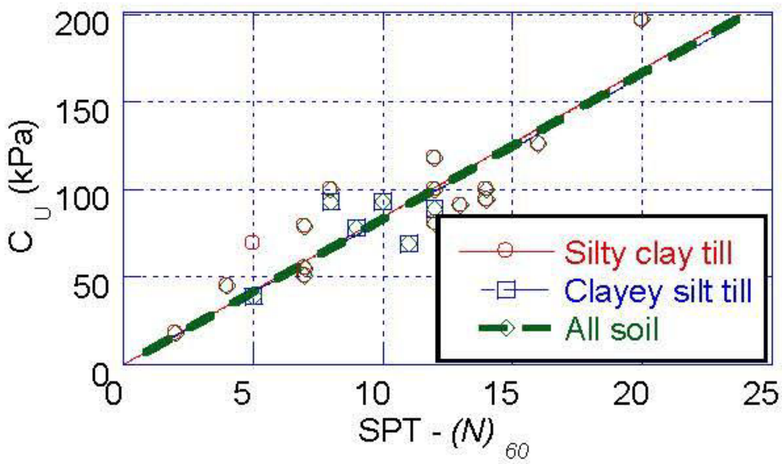


Figure 24: Correlation between Cu and SPT-N

(10) In 2013 Mostafa Abdou Abdel Naiem Mahmoud studied the reliability of using SPT test to predict the properties of silty clay with sandy soil.

The research area was Tabarjal – Al-Jouf, KSA.

Standard penetration test was carried out in 100 locations to depths between 10-15m below ground surface.

Specimens with different field SPT numbers have been taken for laboratory test.

To determine the shear strength parameters (C & Ø), direct shear test was done.

From the regression analysis the following equation was found:

C = 0.014 N”-0.18

Were,

C = Cohesion in Kg/cm^2

N’= Corrected N value.

(11) In 2016 Kumar proposed a relation using random number generation method between field SPT-N value & Cohesion KPa.

C= -2.2049+6.484N

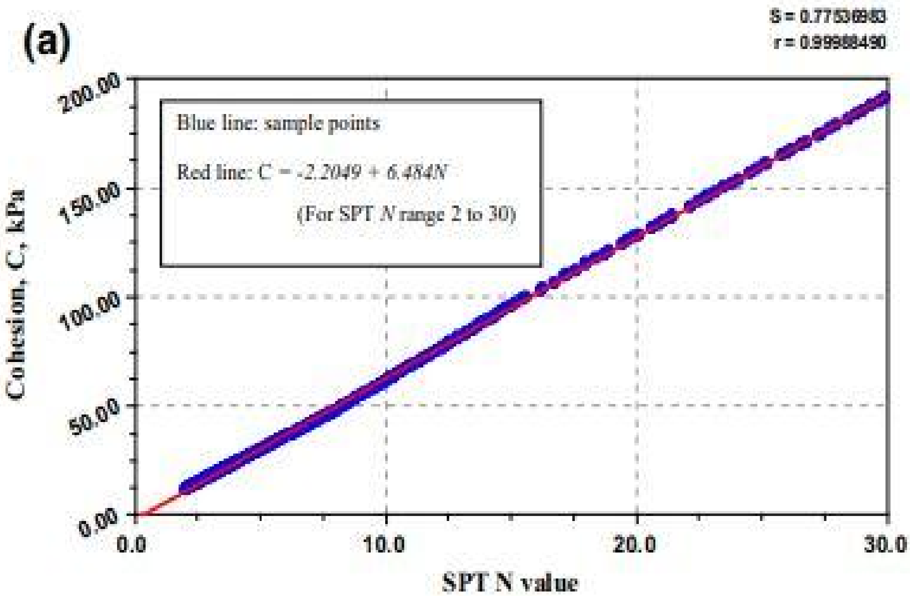


Figure 25: plot of 200 pair of data points of SPT-N & Cohesion of Cohessive Soil

(12) In 2018 N.Q.A.M. Yusof & H. Zabidi studied the soil of Pahang, Malaysia to check the reliability of pretending properties of soil from SPT-N

The study area is located in the State of Pahang, in the district of Cameron Highlands.

Samples collected from 14 boreholes.

Borehole depth was 1.5 to 15m.

Triaxial test was conducted in the laboratory

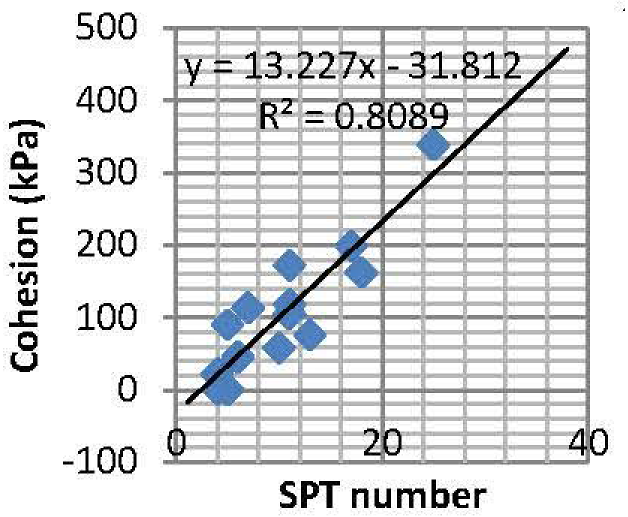


Figure 26: After performing regression analysis using the test results between SPT-N & Cohesion.

After performing regression analysis using the test results

The regression co-efficient was found more than 0.7 which

Indicates the reliability of pretending cohesion from SPT-N.

Existing Correlations between SPT N & frictional angle

The angle of friction is estimated by using empirical equation using the SPT N value from field. The existing equations on this parameter are mentioned bellow:

(1) The empirical equation developed by Peck in 1953

$\Phi = (0.3N)^{0.5} + 27$ (Here N=field SPT N value for limited range)

(2) The equation of Peck (1974) was later approximated by Wolf 1989. Which is-

$\Phi = 27.1 + 0.3N_{60} - 0.00054(N_{60})^2$

(N60 = Corrected SPTN value for 60% hammer efficiency)

(3) Schmertmann (1975) included σ' into a N− **Φ** relationship. This correlation was later approximated by Kulhawy and Mayne (1990) where pa =100 kPa is the atmospheric pressure, σ'= effective overburden stress. According to this-

$\Phi = \tan^{-1} \{N_{60}/ [12.2 + 20.3(\sigma'/p^a)]\}^{0.34}$

(N_{60} = Corrected SPT-N value for 60% hammer efficiency)

(4) Japan Road Association (1990). Hatanaka and Uchida (1996) tested high quality, undisturbed frozen samples from few sites in a standard triaxial apparatus and each σ' was compared against the corresponding N_{60}. They proposed Equation to estimate **Φ** where Cn is a correction factor. According to that-

$$\Phi = \sqrt{(20 C_n N_{60})} + 20$$

(N_{60} = Corrected SPT-N value for 60% hammer efficiency)

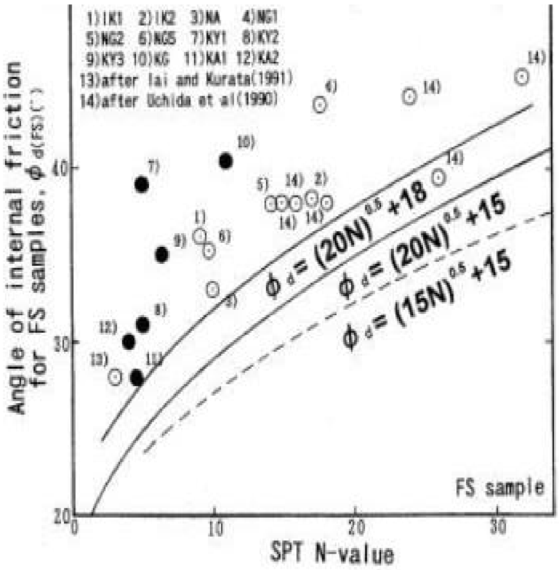


Figure 27: Relation between the angle of internal friction for FS samples and SPT-N values

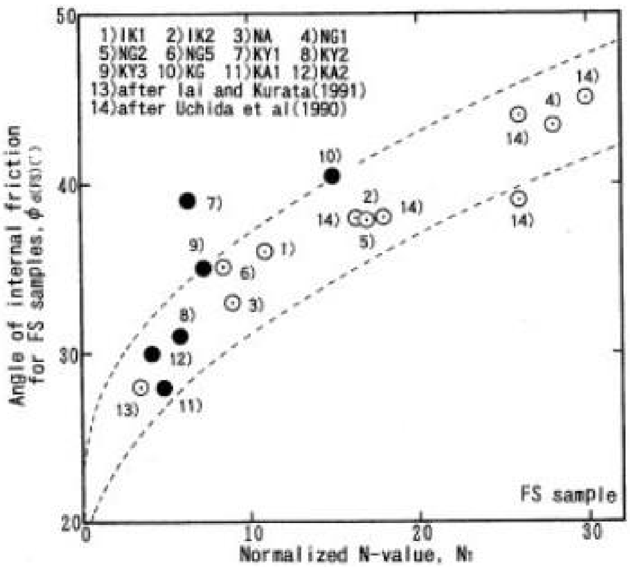


Figure 28: Relation between the angle of internal friction for FS samples and normalized N values N_1

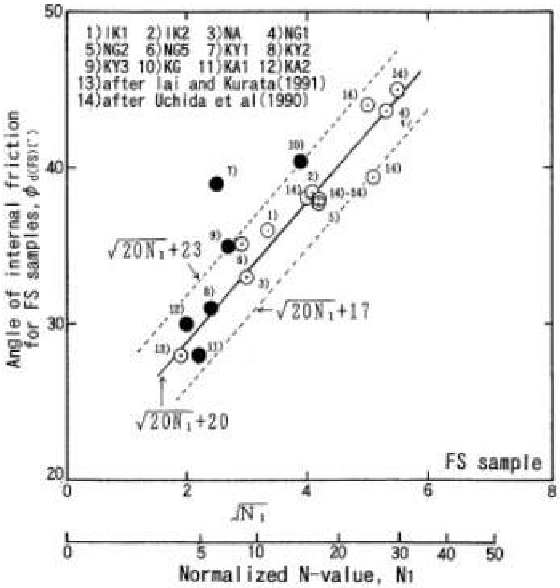


Figure 29: Relation between SPT-N of FS samples & $(N_1)^{0.5}$

(5) According to (Ohsaki et al, 1959) the correlation is:

Φ $=(20N)^{0.5}+15$

(Here N=field SPT N value for limited range)

(6) According to (Dunham, 1954) the correlation is:

For Angular and well-grained soil particles

Φ = $(12N)^{0.5}+25$

For Round and well-grained or angular and uniform grained soil particles

Φ = $(12N)^{0.5}+20$

For Round and uniform-grained soil particles

Φ = $(12N)^{0.5}+15$

In all three cases (Here N=field SPT-N value for limited range)

(7) According to Japan Road Association, 1990 the correlation is:

Φ = $(15N)^{0.5}+15 \leq 45$

Here the N value is considered as Field SPT-N value which is over 5. (N>5)

(8) Shioi and Fukui (1982) has stablished three equations

For roads and Bridges

$\Phi = \sqrt{N_{70}'} + 15$

For buildings

$\Phi = 0.36\, N_{70}' + 27$

For general purposes

$\Phi = 4.5\, N_{70}' + 20$

Here N_{70} = (Corrected SPT N value for 70% hammer efficiency)

(9) Brown 2008 has established an equation where 36 standard penetration tests conducted at 24 different boreholes in Oconto and Marinette County, WI, in 2005 were used to estimate parameter. These data produced an average value of

$1/\beta = 0.3818$ with a 0.018 standard deviation. The correlation is:

$\Phi = \tan^{-1}(.25 N_{60}\, p^a / \sigma')$

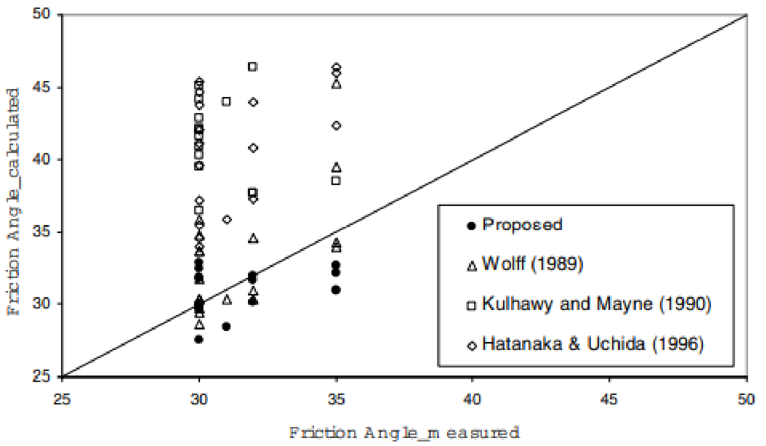


FIG. 1. Comparison of calculated friction angle with the measured.

Figure 30: Correlation of calculated frictional angle with previously measured results.

(10) N Puri conducted a study on Haryana of north India. From 1053 borehole locations he obtained samples. The results of relation were obtained from Linear regression, ANN, SVM, M5 Tree, Random Forest methods of machine learning. The proposed relationship was based on Field SPT-N value.

$\phi = 0.3125 * N + 26.1261$

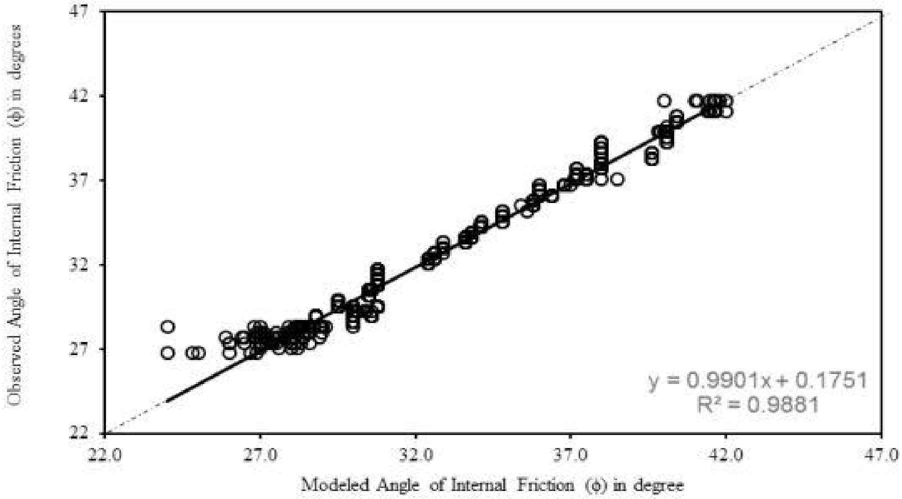


Figure 31: Performance analysis of SVR model for predicting internal angle of friction using SPT N

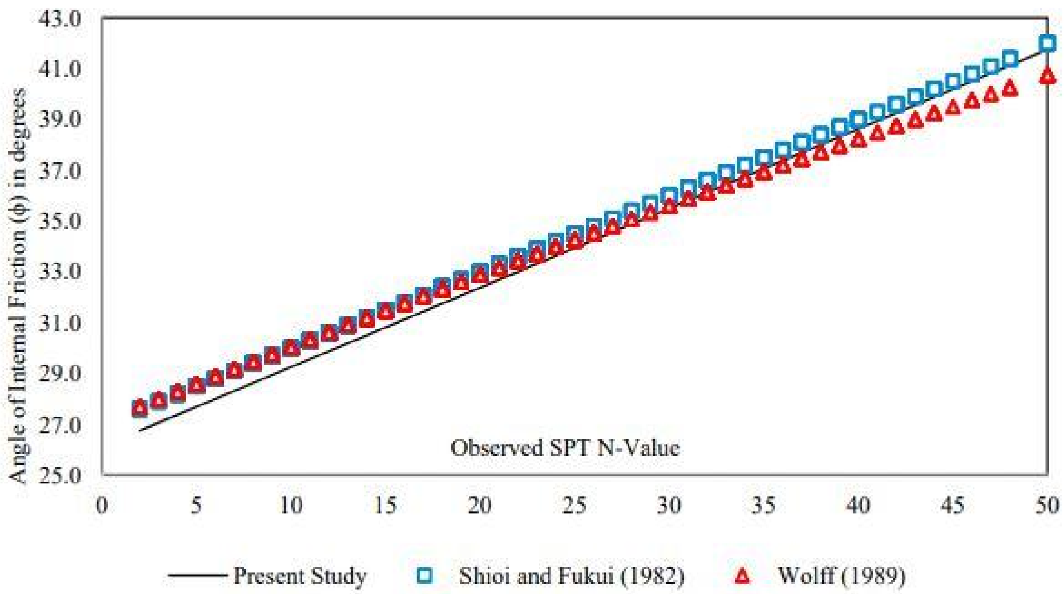


Figure 32: Comparison between prediction models of angle of internal friction.

(11) In 2021 A. Hossain, T. Alam, S. Barua & M. R. Rahman conducted a study on Joydevpur-Mymensingh-Jamalpur section of Bangladesh. Samples were obtained from 210 boreholes. The relationship results were obtained from using multiple linear regression, SVR, ANN methods of machine learning. The proposed relationship was between angle of internal friction & N_{60} (Corrected SPTN value for 60% hammer efficiency), grain size analysis properties, depth & soil properties. The proposed formula was

$\Phi = 28.5985+9.5493N_{60}+0.6882 - 1.0256D_{10} - 0.4613D_{30} + 2.5435D_{60} -1.1907C_u-0.7696Sand-0.4126Silt-0.4607Clay-0.3182Gradiation$

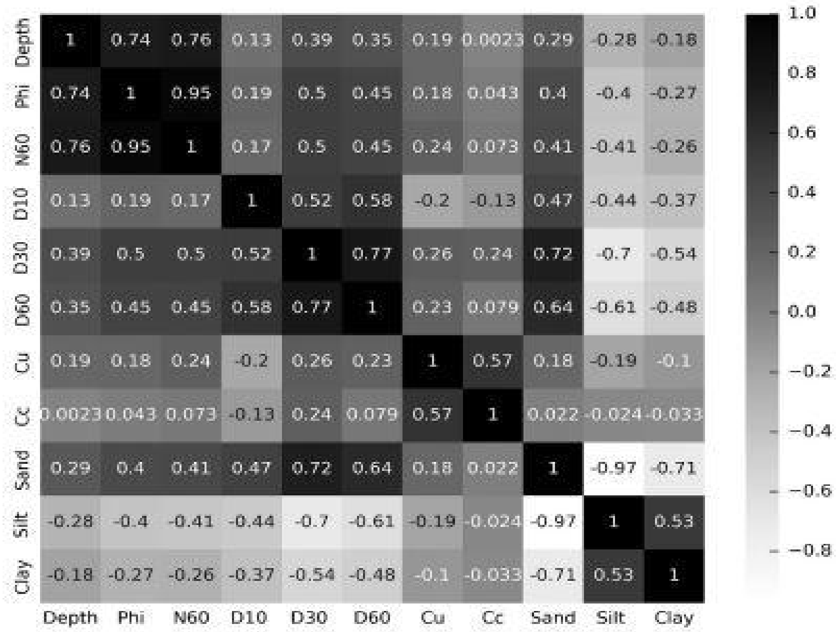


Figure 33: Correlation matrix of dataset

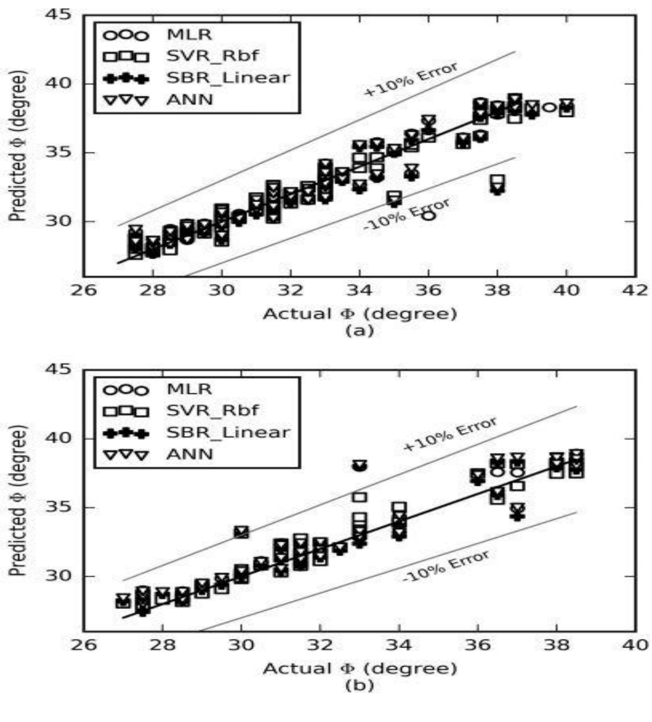


Figure 34: relation between actual and predicted phi by MLR, SVR & ANN for (a) training, (b) testing dataset

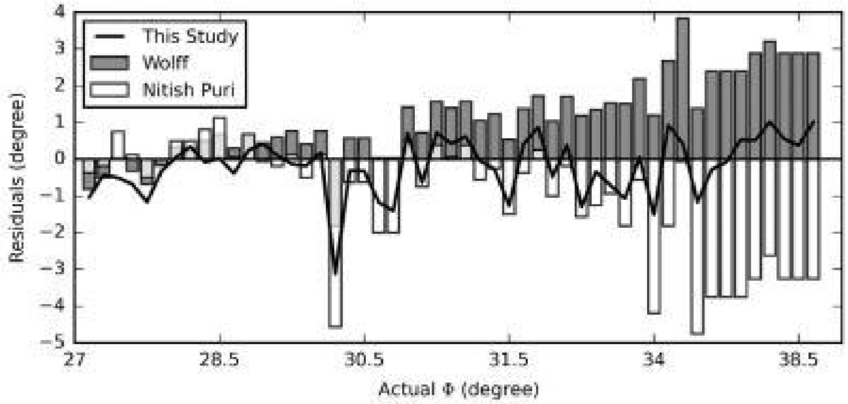


Figure 35: residual ϕ comparison of present study, wolf and Puri.

(12) In 2016 Kumar proposed relation using random number generation & curve expert for 200 samples and obtained the relation based on field SPT N value ranging 4 to 50.

$\phi = 0.2857 * N + 27.12$ ($r^2=0.998$)

The higher r^2 value represents higher predictability of the model.

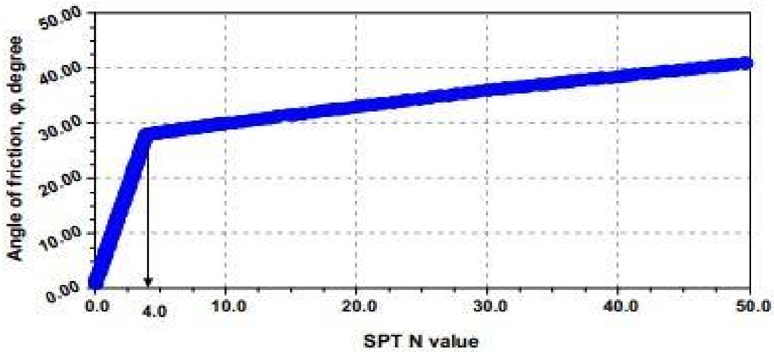


Figure 36: plot of 200 data points of SPT-N value & angle of friction.

(13) In 2018 Yusof conducted study on 32 samples obtained from 14 boreholes of Malaysia's Pahang state.

The relation was proposed based on field SPT-N value.

$\Phi = 0.481N + 29.174$, $R^2 = .7903$

The higher R^2 value indicates higher predictability of proposed model.

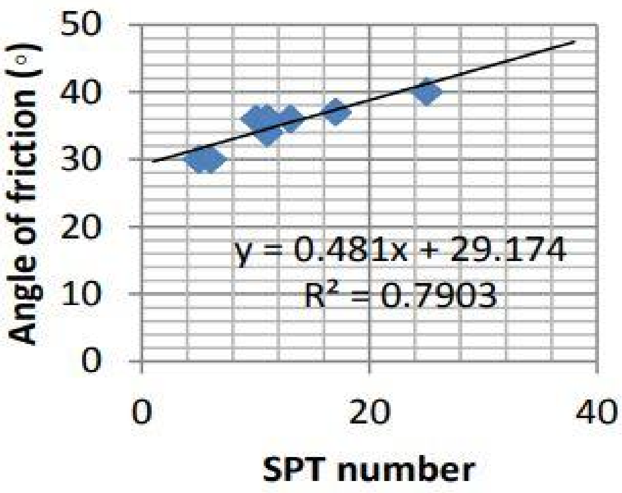


Figure 37: the relation between SPT-N number & angle of internal friction.

Conclusion

From existing correlations provided by different researchers we have found different empirical equations. If we use those formulas for our region the values of C & Φ differs from the value we obtain from lab experiments. Sometimes the values are too high and sometimes they are too low. So, we have to do regression analysis between the value we get from lab result & the values from empirical formulas. As a final product we have to produce empirical equation where the standard deviation is low and the level confidence is high between the values of lab result & formula in Bangladesh region.

CHAPTER 3 : METHODOLOGY

General

This chapter describes how the samples for the data were obtained. The undisturbed soil samples were collected using a 75mm O.D thin-walled UD sampler with length 1000mm and Standard Penetration Resistance was measured at the same depth of collection of undisturbed samples at 1m intervals. SPT is determined and samples are collected at three different places. Tests were carried out in the geotechnical laboratory of PROSOIL Foundation Consultant to determine Atterberg limits, specific gravity, index properties, sieve analysis, unconfined angle of friction and compressive strength of the soil obtained from the locations: 300ft, Purbachal, Uttara, Mirpur, Kawlar of Dhaka and from Narshingdi, and Habiganj of Sylhet.

Mistakes occurring in SPT procedure

There are three reasons due to which the mistakes can occurs during SPT. The major errors that occur in the field when conducting the process, height from which SPT hammer is dropped, the thickness of spoon cutting shoe and using of non-standard Shelby tube.

Height of fall of SPT Hammer

Height of fall of hammer has a major impact on the value of SPT. According to ASTM D1586-08 the height of the fall should be more or less within 76cm. So, when the hammer falls from a large height, a large energy is released and smaller SPT is obtained and vice versa. Therefore, the correct SPT-N value will not be collected. For greater height, the SPT-N value will be lower thus the cost for foundation would be excessive. While, if height of the hammer falling is less than 76 cm, higher SPT-N value would be obtained and the foundation design would be risk bearing. Therefore, the height of fall has to be strictly maintained. On field the height of fall can be maintained by hiring skilled labor, who can carry out the task by either labeling and using nut on top of the guide rod or by using an auto trip hammer.

Thickness of SPT spoon cutting Shoe

Split spoon sampler was used to collect the disturb samples. It is made up of a tube that split into two halves: a shoe and a sampler head combined with holes to release air screwed onto the ends. During collection, the shoe and head: the two halves of the tube are detached in order to remove the sample. It has an outer diameter of 51mm and inside diameter of 35mm while the total length of the spoon is 460mm.

Non-standard Shelby tube:

The thin-walled open-drive non-standard Shelby tube was first used in USA in late 1930s. National Tube Company of the United States was the first to produce ‘Shelby Tubing’. There were two types of early instruments. The thin-wall sampler tube was spot welded to a short length of heavy tube, which was inserted into the head by the US Engineer Office, Boston District sampler. Another way to secure the tube to the head is to utilize tubing that fits neatly over the lower section of the sampler head and is secured to the head with two Allen set screws that, when engaged, lie flush with the sampler tube's outer surface. This design has been incorporated into a large number of samplers, and is now in use worldwide (ASTM D 1587--74).

Field Test and Sample Collection

To evaluate subsoil stratification and collect disturbed and undisturbed soil samples from various depths and locations, sampling sites were chosen. Purbachal, Dhaka MRT project line 6- Depot, and Bashundhara are all in Dhaka. Other locations include the Dhaka-Sylhet-Tamabil route and the Osmani Airport in Sylhet. Airports at Barisal, Saidpur Airport, Nilphamari.

Field Investigations

Rotary drilling was used to complete the bore. For choking action, a strong string of drill rod is employed. Water jetting is used to progress the boring, which is pushed through the hollow drilling rods. Drilling mud is injected into the drill hole by the drill rod. Then it removes the shattered rock or soil pieces, which are gathered in a settling tank for recirculation. The water comes from a little wetland. The land water from the borehole is released into a similar reservoir, where the coarse materials settle out. Then, at this stage, so-called 'wet samples' can be obtained. Hardly water could be utilized unless the level is shallow as well as the subsoil is stable. As these aid in the stabilization of the wellbore so Drilling fluid is helpful. Drilling fluids is a clay-and-groundwater slurry. Because of its higher specific gravity Drilling fluid does have a securing effect just on bore hole. In comparison to water, this is partially attributable to the creation of drilling mud on the pit's sidewalls. There is no casing because fluids have a stabilizing influence. In most rocks, this approach suitable for boring holes with such a size 10 cm, or more preferable 15 to 20 cm is needed if drilling fluid is employed. Cuttings can be inspected to determine the depth of distinct strata. Cuttings that have settled in the circulation tank are collected and disposed of on a regular basis. Uncased boreholes of 100 mm diameter are bored. To get 75 mm diameter undisturbed samples from the cohesive soil layer. For digging on land, a 5m long temporary casing is being employed. After the investigation is completed, all boreholes will be backfilled.

Boring equipment was used to develop a borehole 89mm in diameter for the clayey/sandy layer on the Dhaka-Sylhet-Tamabil route A wash boring is moved forward by spraying water through cylindrical drilling rods. Circulating water dispelled cuttings from the hole. Drilling rods are moved vertically and horizontally by a driller spinning back and forth while pressing and lessening the rope. The water is drawn from a little wetland. The borehole's land water is dumped into a similar storage. The coarser grains then dropped in and were collected as moist samples. In soft or low-cohesion soils, an NX 89mm casing is necessary. However, it is frequently eliminated in stiff, cohesive soils while mere few specimens are required. Variation in soil character is influenced in part by the driller's feel and, in part, by inspecting the treasures in the water because it exits the shell. However, only while specimens are extracted from the drill bottoms can a definitive characterization of the soil be made.

Undisturbed Sampling and SPT in Field

Four Districts were selected for sampling Dhaka, Sylhet, Barisal, Nilphamari. According to the site circumstances, Holes were sampled for undisturbed soil. The sampler utilized was a 75mm O.D thin-walled Undisturbed sampling including a length of 1000mm. The lengths of the restored sample were evaluated when the sampling was retrieved from the hole. The recovery ratio was determined to be calculated. While sample underneath the ground water, the water level remained maintained at the casing's surface until the sampler was withdrawn. To get jar specimens for both the upper and bottom ends of the tube, a max of 50mm of undisturbed soil were removed. To create an airtight seal, both ends of these samples were treated with a non-shrinking wax after they were prepared.

Standard Penetration Tests (SPT) were performed to determine the consistency of the ground. To gather Disturb samples for visual examination and laboratory testing.

The N value was described as a number of blows necessary for a 12inch (300mm) penetration measured after a 150 mm seating drive. The amount of blow for a reported penetration was documented in the case of premature rejection situations. Here we used free fall hammer. This method was made out of a hollow cylindrical mass that slid above a steel rod. It works by elevating a mass using a wire. When the bulk reaches the proper height (760 mm), it is set free, forcing the split spoon into the dirt. When the mass reaches the proper height (760 mm), it is set free, forcing the split spoon into the dirt. During the Standard Penetration Test, disturbed samples were taken from the split-spoon sampler and visually examined before being stored in a polypropylene zipper bag for laboratory testing.

For the Dhaka-Sylhet-Tamabil Road here used 50mm O.D. x 600mm length and thick split sampling tubing pushed through undisturbed subsoil. Here force in a rolling hammer carrying 63.5 kg is used in the SPT. The 'N' value of penetration depth is stated as the number of blows to the head required to complete a 300mm penetration after a 150mm beginning puncture. The sampling tube is withdrawn then disassembled just after test and provide a "disturbed" yet random group. In terms of load carrying capacity, we advise multiplying the number of strikes per 300mm 'N' on a 760mm free - falling based by 10 to provide a hint of the load carrying retention in KN per square meter based on the Sampler Adequacy Test findings.

Laboratory Test

Soil samples were subjected to geotechnical laboratory tests. In order to categorize and evaluate mechanical qualities. Soil samples recovered from boreholes excavated at the site. It was subjected to a thorough laboratory examination. Both undisturbed and disturbed samples were transferred to Prosoil's geotechnical laboratory for testing. Here are some laboratory testing programs. These are Atterberg limits, grain size analysis, unconfined compression test, Direct Shear test, natural water content and Specific Gravity test. The laboratory tests were all completed in compliance with ASTM Standards. In addition, the soil samples were categorized using (USCS).

On undisturbed cohesive soil samples, unconfined compressive strength tests were performed. ASTM D 2166 is utilized to carry out the unconfined compression test. The test is carried out by compressing cylindrical samples until they fail. Failure usually happens whenever the shear stress-to-shear strength ratio is at its highest. The sample's cohesion is defined as 1⁄2 the unconfined compressive strength.

Utilizing the direct shear (ASTM D 3080) test to determine the shear strength qualities of soil material. four samples from a moderately undisturbed soil sample are tested. A sample was put in a shear box with two stacked bands to retain it. The connection between the two bands is about at the sample's half. A vertical confining tension is measured by applying. The top band is yanked lateral until the failure occurred. The generated strain is measured at periodically to create a stress–strain curve for every confining stress.

Sieve analysis experiments were carried out in accordance with ASTM D 422. Sieving for particles remained on a 0.075 mm sieve was performed on oven-dry samples. The amount of soil kept on every sieve is calculated. In sieve analysis, it reported as a percentage of the overall mass of the sample. On a logarithmic scale, the grain size is shown. As a consequence, two soils with the similar degree of homogeneity are described by distribution plot curves. The settling of soil particles in water is the basis for the hydrometer investigation of fine materials. While a soil sample is disseminated in water, the particles are deposited at varying rates. It depends on shape, size & weight.

On typical samples collected of cohesive soils, Atterberg limits (ASTM D 4318) were calculated. The Atterberg limits are randomly determined borders between the liquid and plastic phases. The liquid limit is the water content in a standard cup that a portion of soil contains. When the cup is exposed to 25 normal shocks, the grooves cut by a standard grooving tool would flow together from the base of something like the groove.

Detail of Instruments

The Standard Penetration Test (SPT) is carried according to ASTM D 1586. Here the In-situ testing is done by Rotary Percussion method. Here both offshore & onshore drilling was applied due to various geographical conditions. The whole procedure needs a SPT testing setup which is consists of derrick, power unit, winch, pump, drill head, hammer weight and other components. Here fluid (bentonite slurry) is also used to provide some support to sides of the hole if no casing is used, fluids are also used to carrying out the debris and cooling of drilling rod. The safety hammer and the donut hammer are two types of hammers that are commonly used. The safety hammer is relatively lengthy and so has a small diameter. The internal striking ram of the safety hammer considerably decreases the chance of injury. The donut hammer has a smaller diameter than the safety hammer due to its shorter length. The mean energy ratio delivered by a safety hammer was determined to be around 60% in energy calibration research by Kovacs et al. (1983), whereas the mean energy ratio delivered by a donut hammer was about 45 percent. The hammer efficiency also differs in case of manual & auto-trip hammers.

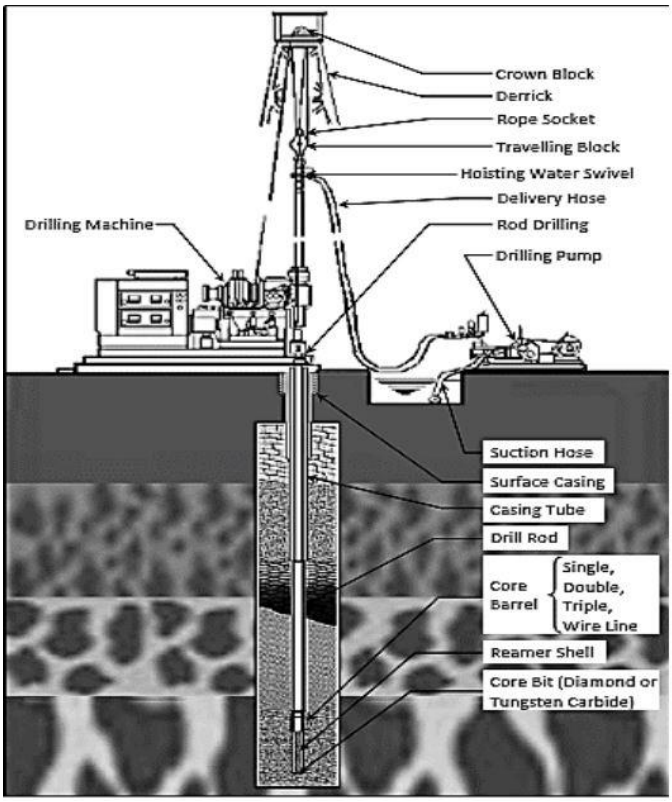


Figure 38: SPT Test instruments

Rotary drilling

Here a spinning drilling bit cuts or grinds the soil material at the bottom of the hole. Water is driven down through the drilling rods and out of little holes in the bit. It then rises, bringing the soil fragments among with it through the annular gap between casing & drilling rod. It overflows into a sedimentation pond via a T connection at the top of the casing, where it is fed back into the drilling rod by a hose. A swivel coupling is supplied between the water hose and the drilling rod, allowing the drilling rod and drill bit to revolve on the dirt at the bottom of the hole which makes cutting easier. It is not necessary to have a casing more than 3.0m or 4.5m beneath the ground's surface. The diameter of exploratory drilling rotary borings typically ranges from 75mm to 150mm. The drilling bit is replaced with a sampler to get a sample. Uncased boreholes of 100mm diameter must be drilled to collect 75mm diameter undisturbed samples from the cohesive soil layer.

Drilling fluid

Drilling fluid, which is made up of a slurry of clay and water with bentonite added, is used to keep the borehole from collapsing during rotary drilling for site exploration. To prevent slurry loss, drilling mud covers and supports the borehole wall and seals off permeable stratum.

Drilling Bit

The diamond or tungsten carbide bit makes an annular hole in the material and an intact core enters the barrel to be removed as a sample in core drilling, which is employed in rocks and hard clays. The most common core diameters are 41, 54, and 76 millimeters, however they can go up to 165 millimeters.

Piston Sampler

In case of pure clay layer where the SPT N<5 the piston sampler is used to obtain the undisturbed soil sample. It is made out of a thin-walled tube with a piston. The piston is connected to a rod that runs through the sampler head and into the hollow-boring rod. The sampler is lowered into the borehole using a piston at the tube's bottom end during the sampling procedure. A clamping device located at the top of the rods holds the tube and piston together. The vacuum created by the piston and the sample keeps the soil in the tube. As a result, the piston acts as a non-return valve, collecting soil samples while preventing soil backflow. The piston sampler should be pushed into the soil rather than forced. Hydraulic force is used for pushing the sampler. The sampler's diameter typically runs from 40 to 100 mm, although it can be as large as 250 mm.

Figure 39: Piston sampler

Mazier Sampler

The Mazier sampler is a three-tube swivel retractor barrel whose efficiency is based on the fact that the amount of inner barrel protrusion is controlled by a spring in the device's upper section. A brass liner is included in the inner barrel and can be used to transport or store samples to the laboratory. The cutting shoe on the bottom of the inner barrel is thick, making it far less likely to be damaged than a thin-walled seamless tube, but it also introduces issues of disruption when the high area ratio shoe moves ahead of the core bit.

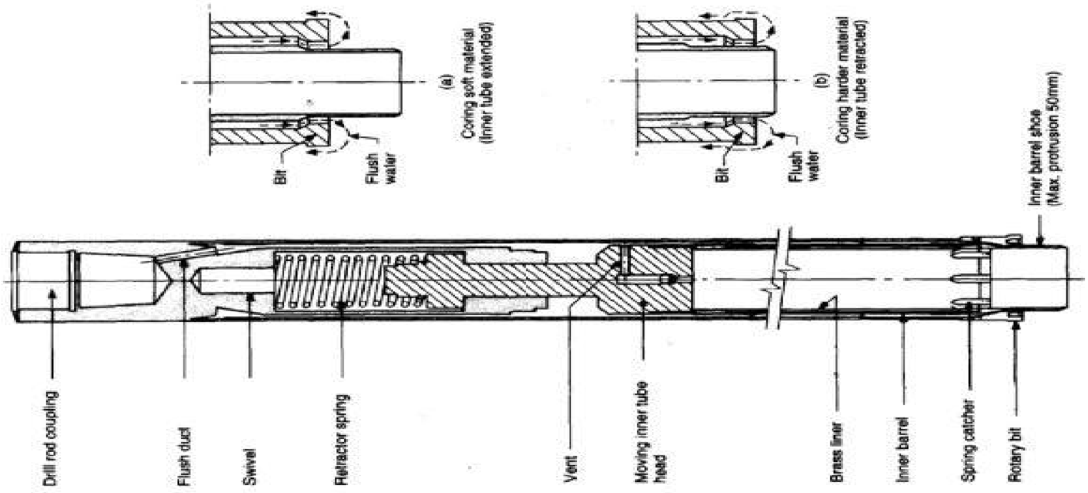


Figure 40: Mazier sampler

Split Spoon Sampler

The SPT test is used to determine some parameters of soils, particularly in soils with low cohesion, for which undisturbed samples are difficult to collect (ASTM D 1586). The SPT makes use of a split spoon sampler. It's a tube with an exterior diameter of 51 mm and an inner diameter of 35 mm. It measures 457-610 mm in length and is divided longitudinally. The sampler will be linked to the drilling rod's bottommost section. Using a drop hammer, the drilling rod is forced into the soil. The sampler is driven to a depth of 457 mm into the soil using a hammer with a 63.5 kg weight dropping from a height of 762 mm. Each of the three 152 mm increments is driven with a different set of blows. The Standard Penetration Resistance value is the sum of all blows utilized in the last 305 mm. As a result, the N-value represents the number of strikes necessary per millimeter of depth. The sampler can be removed once all of the blow counts have been recorded. The container is then entirely opened to obtain a disrupted sample for further testing and evaluation.

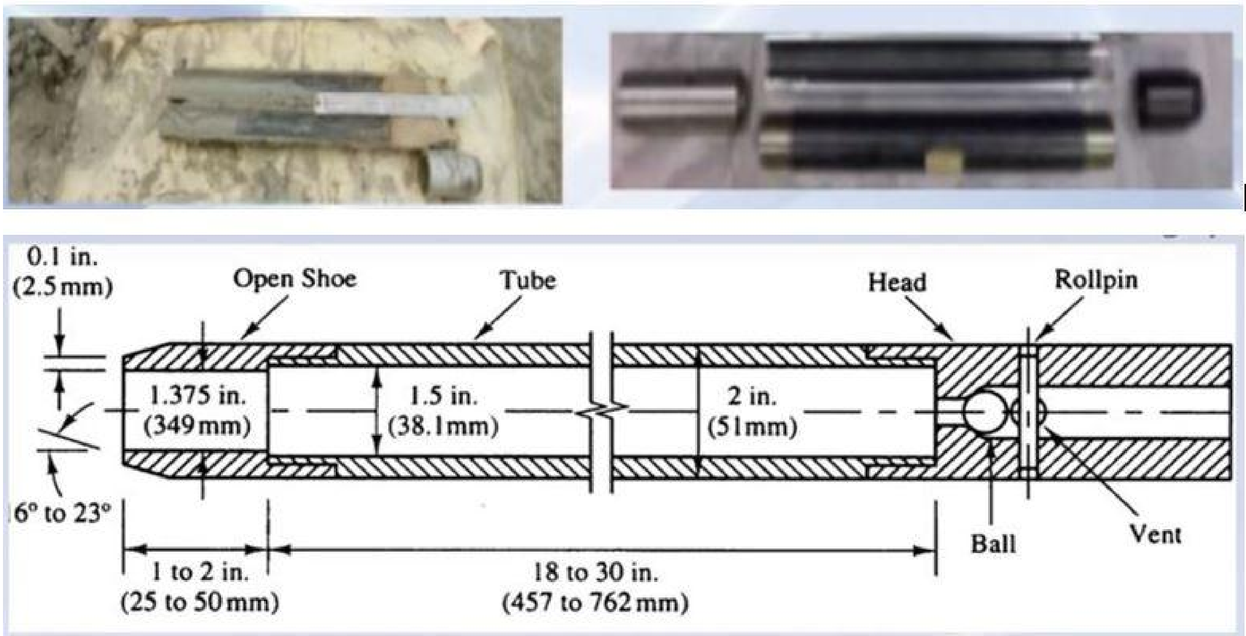


Figure 41: Split spoon sampler

Auto Trip Hammer

In case of manual hammer the falling height of 762mm is not properly maintained. In this case the rope is rarely entirely separated from the pulley, the quantity of energy delivered depends on the operator's competence, the smoothness of the cathead like amount of rust, and the number of times the rope is originally looped around the pulley. According to Kovacs et al. (1982) to reduce the impact of the number of turns and operator characteristics as determinants of the delivered energy, two turns of the rope around the pulley should be employed. To reduce the energy deficiency auto trip hammer is used where the falling height distance of 762 mm is automatically maintained. For performing the Standard Penetration Test, the Auto trip Hammer meets the standards of BS 1377: Part 9: 1990. (SPT). The hammer is made up of a 63.5 kg weight with a pick-up and self-tripping mechanism that ensures the weight has a precise free-fall of 760 mm. The inner shaft serves as a guide, allowing the weight to fall with minimal resistance and ensuring that the weight lands squarely on the anvil. The drive rods are securely screwed into the anvil's base, which is equipped with a 1.1/2" B.S. whit worth or BW rod box connection. When the hammer is not in use or during transportation, the sliding outer sleeve is secured to the inner guiding rod by a safety cross bolt. The hammer's overall length is 1.8 meters when not expanded and 2.6 meters when fully extended. The total amount of weight. The energy transfer ratios "ER" generated by the Auto Trip Hammer are normally around 90%. Lower sensitivity and larger energy correction coefficients, CE, result from this efficiency.

Figure 42: Auto Trip hammer

Shelby Tube Sampler

ASTM D 1587 utilized, which refers to 'Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes'. Generally, wall of the sampler tube is thin and at the toe, there's a cutting edge. A sampler contains a check valve, the drill rod is attached to the head of the tube, and pressure vents. Usually, not more than 152.4 mm (6") the length of the tube is used for sampler which leads to the cohesive soils. The tube cohesion of the sample causes retention of the sample after withdrawal of the tube and the check valve creates the vacuum. Standard ASTM dimensions are; 127mm (5") OD, 1371.6 mm (54") long, 11-gauge thickness; 50.4mm (2") OD, 914.4 mm (36") long, 18-gauge thickness; and 75.6 mm (3") OD, 914.4 mm (36") long, 16-gauge thickness. Also, must be acknowledged that ASTM allows other diameters of the standardized tube designs as long as the length of the tube is proportionally acceptable for field conditions and they are. An undisturbed soil sample is required.

Nonstandard Shelby Tube

In the scenario of Bangladesh, the most used sampler contains a high area of the ratio of the Shelby tube, cross-sections are irregular, the cutting edge has no specification and a very rough inner surface. The sampler is pressed to the soil by impact loading. It has been referred that the sample will jam in the tube if side friction becomes too great. Besides the inconvenience of very high levels of disturbance is associated with low percentages of recovery. (Clayton and Siddique, 1999). The undisturbed soil sample causes more foundation costs because of the very conservative design of foundations.

Fabrication of Standard Shelby Tube

ISSMFE (International Society for Soil Mechanics and Foundation Engineering) recommended a standard tube sampler about area ratio, edge of leading, and the ratio of length to diameter (ISSMFE, 1965). It has already discussed the current practices are happening in Bangladesh of soil sampling, Siddique (2000). It has already been discussed about the comparison of the unconfined compressive strength and consolidation properties of undisturbed soil samples in this study. The collection was done by using currently practiced and available local Shelby tube samplers. Then, it was compared with Modified Shelby tube samplers. The Shelby tube sampling generated good quality in the field of Bangladesh at a first implementation. Fabricated Modified Shelby tubes were recommended as per ISSMFE (1965). It has an inside clearance ratio of 0.0%, taper of leading-edge angle 600 up to the thickness of 0.3 mm. Also, has a 72 mm inside diameter, the thickness of the wall is 1.9 mm. The ratio of area is 10%, the angle of cutting shoe taper is 120 and the ratio of B/t is 38. The inner/outer surface needs to be smooth.

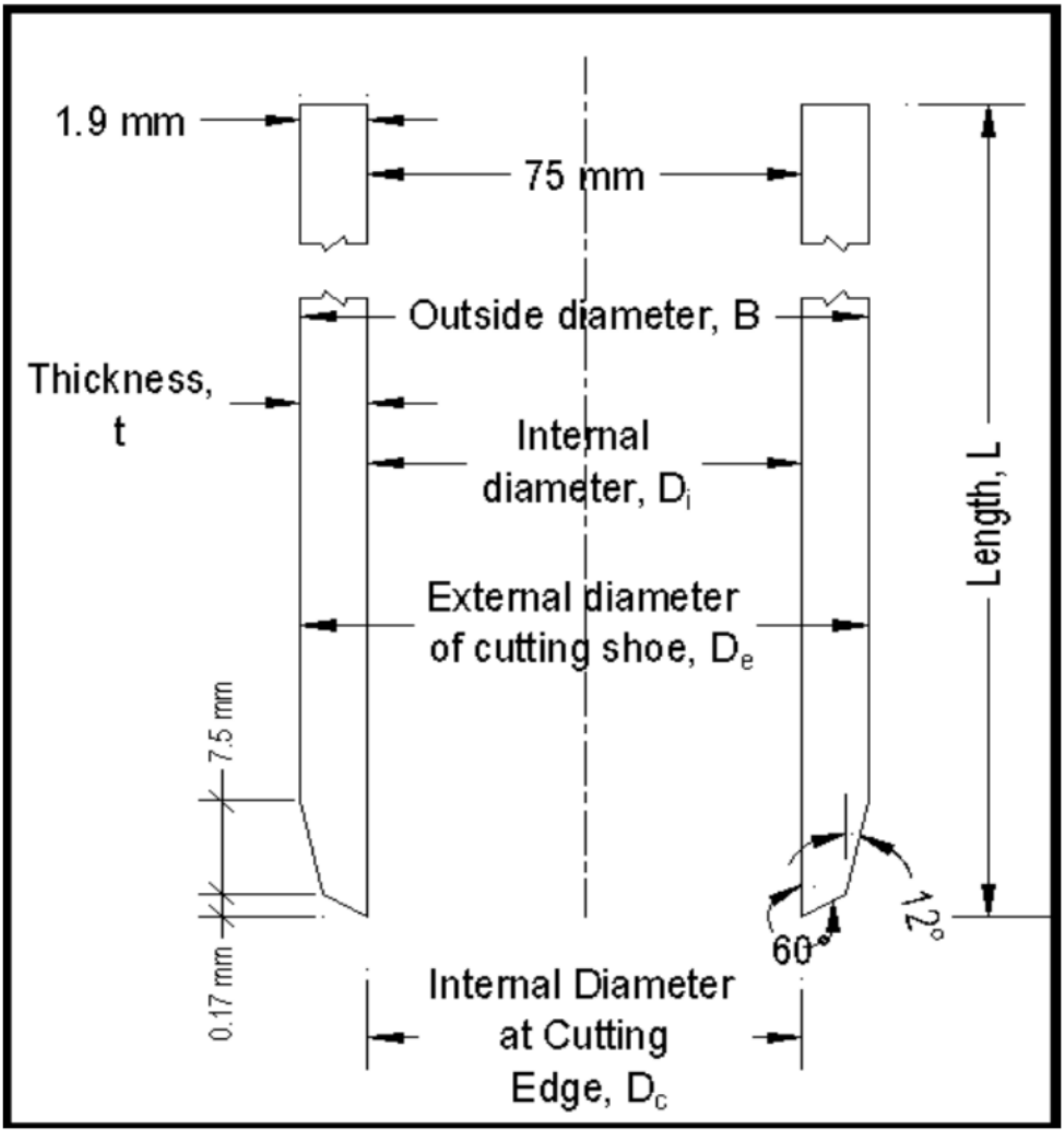


Figure 43: Standard (Modified) Shelby tube schematic diagram

Table 1: Standard Shelby Tube sampler quality parameters used in this study

Parameter	Standard Shelby Tube
Ratio of Area	10%
Material	Stainless Steel
Length to Diameter ratio	8
Roughness of Surface	Smooth
Cutting shoe taper angle	12°
Angle of Leading-edge taper	60° up to 0.3 mm

CHAPTER 4 : DATA ANALYSIS & RESULTS

General

The results that were obtained in the whole investigation is explained in this chapter. This study was carried out based upon the relations between SPT N with q_u & Ø although depth, PI, fines% and N_{60} were included in some cases. In Dhaka, about 300 samples were collected from 300 ft, Purbachal, Uttara, Mirpur, Kawlar these are mostly Fine sand with clay deposit at uppermost layer while for Sylhet, about 400 samples were obtained from Narshingdi, Brahmanbaria, and Habiganj which consist of mainly Clay & Silty Clay. Machine Learning Methods like Multiple linear regression (MLR) and Support Vector Regression (SVR) have been done to find the relationship between the variables. Python and MATLAB has been used to find this relationship.

Data Distributions

The distribution for each Dhaka and Sylhet region using variables: q_u – 134 & Ø – 139 for Dhaka region while q_u – 168 & Ø – 249 for Sylhet region are as follows:

Sylhet clay soil

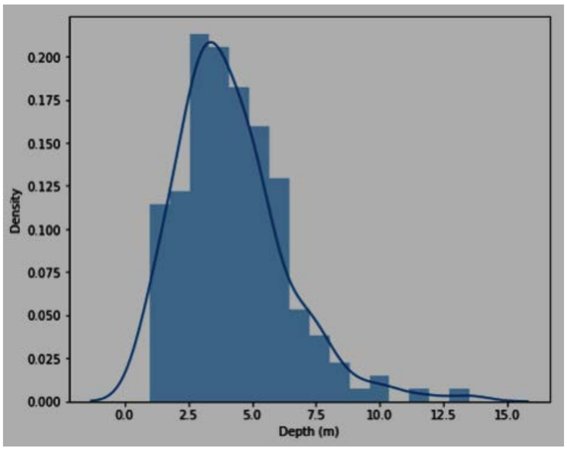


Figure 1 Data distribution of depth

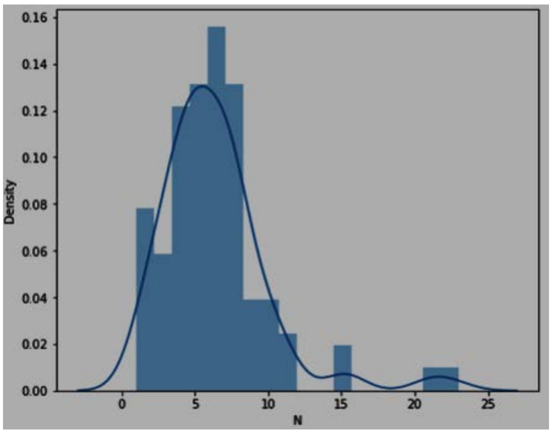


Figure 2 Data distribution of N

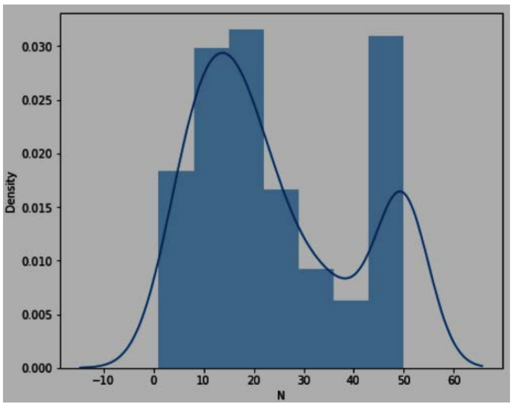


Figure 3 Data distribution of q_u

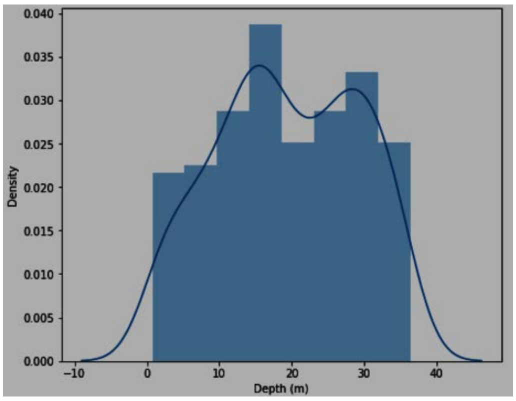


Figure 4 Data Distribution of PI

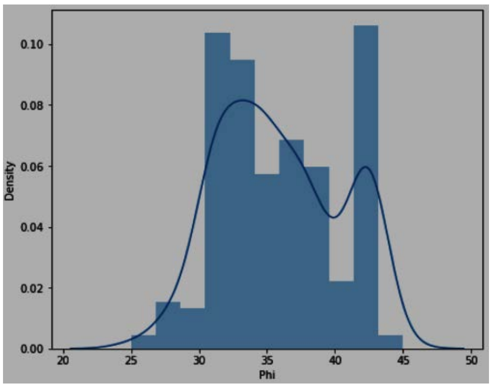


Figure 5 Data distribution of N

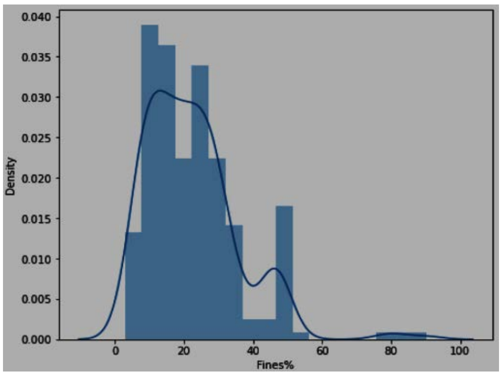


Figure 6 Data Distribution of depth

Sylhet silty sand soil

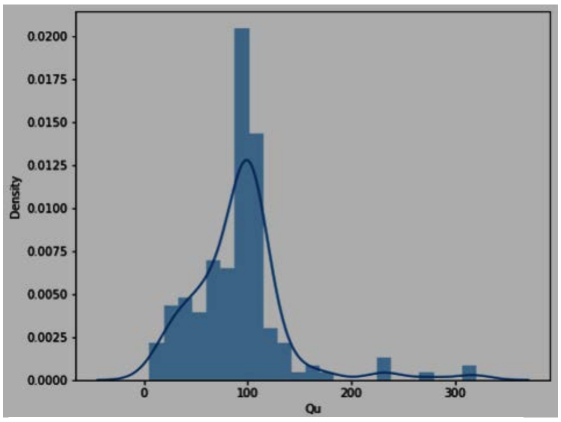


Figure 7 Data distribution of Ø

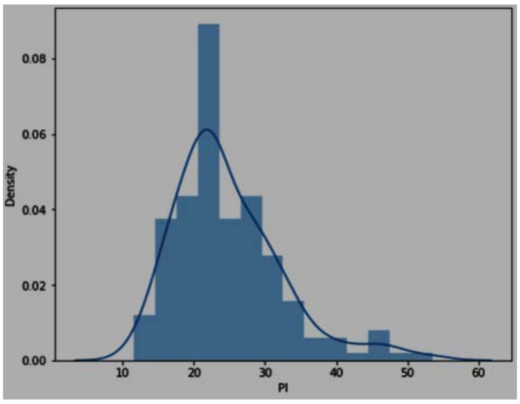


Figure 8 Data distribution of Fines%

Dhaka clay soil

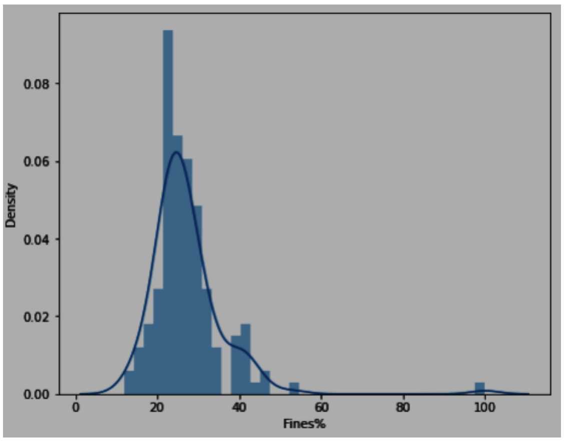


Figure 9 Data distribution of PI

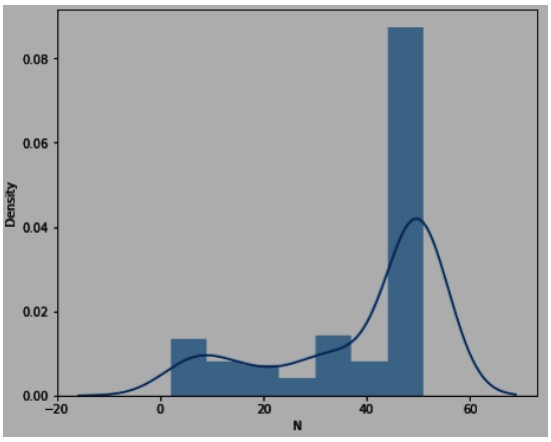


Figure 10 Data distribution of depth

Dhaka silty sand soil

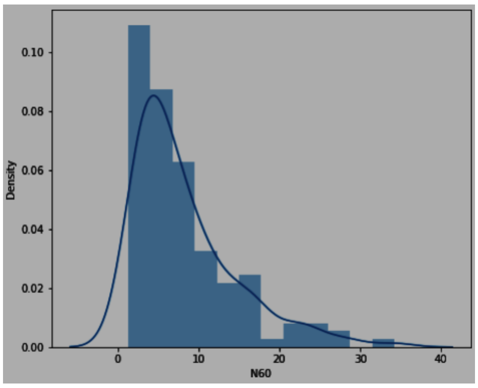


Figure 11 Data distribution of PI

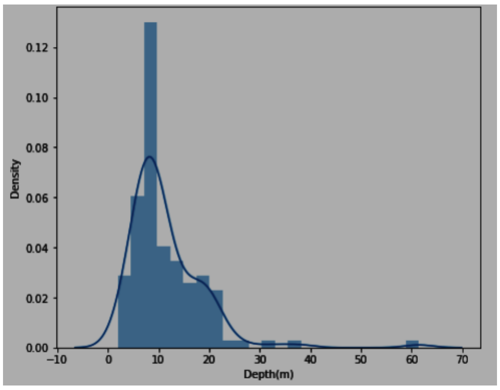


Figure 12 Data distribution of q_u

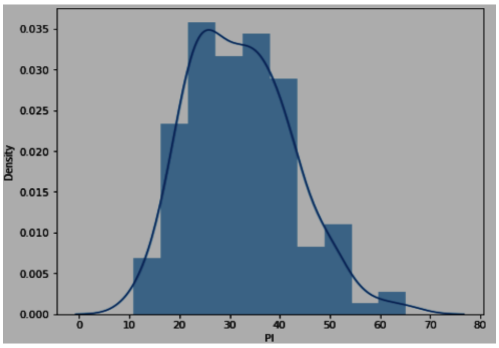


Figure 13 Data distribution of N

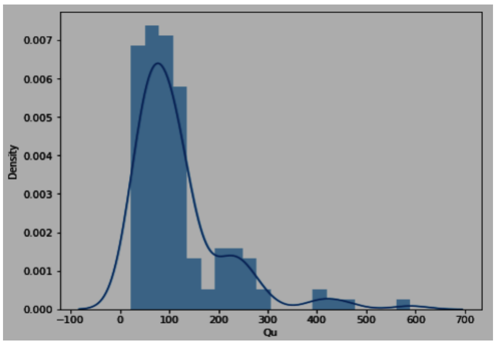


Figure 14 Data distribution of fines%

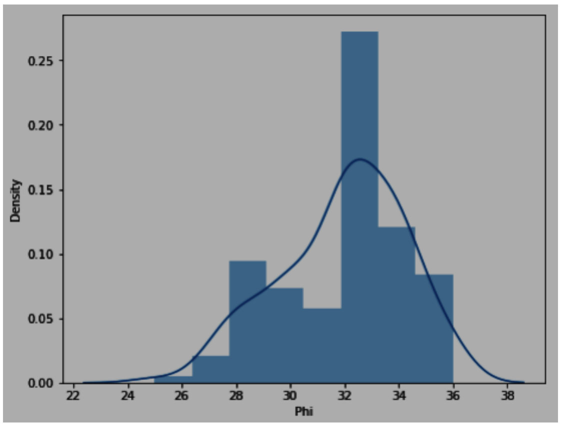


Figure 15 Data distribution of Ø

From here we can see almost all the data except a few are close to normal distribution. Regressions like MLR and SVR are significantly affected by skewness of data. Closer the data to normal distribution, more significant the relationship between the independent and dependent variables.

Correlation between variables

The correlation of qu and Ø between the independent variables of Dhaka and Sylhet regions are as follows:

Dhaka

From the correlation heatmap, we can see that in determining the relationship for q_u, there is significant correlation between q_u with N_{60} while there is insignificant correlation between q_u with depth. While there is also correlation between depth and N_{60} observed.

In determining relationship for Ø, there is significant correlation between Ø with N while there is insignificant negative correlation between Fines% with Ø.

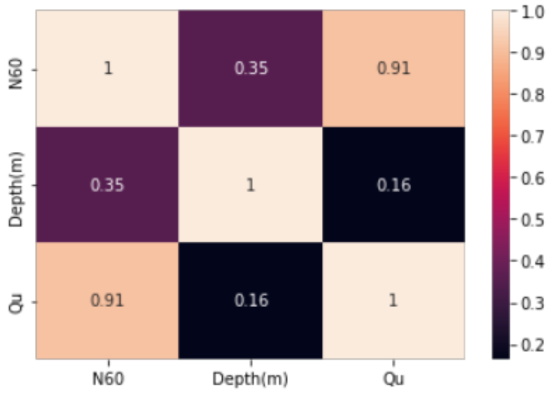


Figure 15 Heatmap for q_u

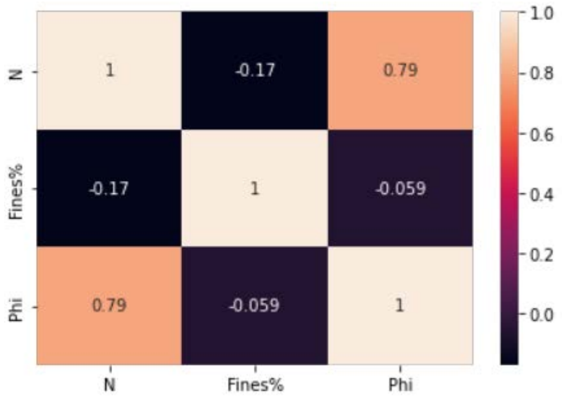


Figure 16 Heatmap for Ø

Sylhet

From the correlation heatmap, we can see that in determining the relationship for q_u, there is significant correlation between q_u with N while there is insignificant correlation between q_u with PI (plasticity index) and Depth.

In determining relationship for Ø, there is significant correlation between Ø with N, depth and fines%. There is also correlation observed between N with fines% and depth.

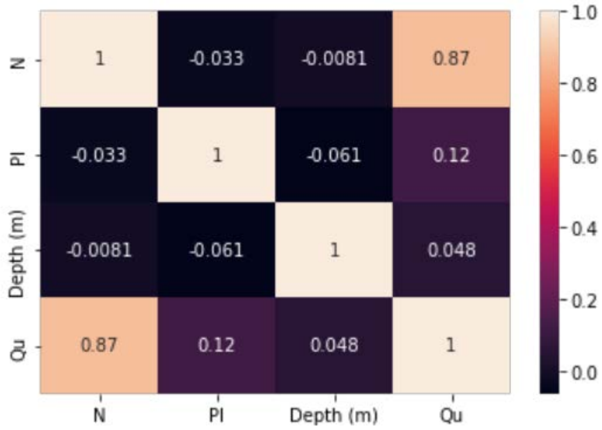


Figure 17 Heatmap for q_u

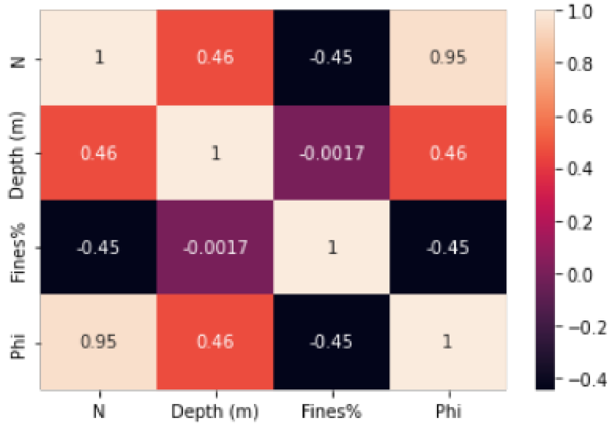


Figure 18 Heatmap for Ø

MLR models

Unconfined compressive strength qu

For both regions, we made eight different models using different combinations of independent variables to determine q_u and chose the best model best model based on R^2 and RMSE

Table 2 different combination of regressions for predicting q_u of Sylhet

Sylhet

	N vs q_u	N, Depth vs q_u	N, PI vs q_u	N, PI, Depth vs q_u	N_{60} vs q_u	N_{60}, Depth vs q_u	N_{60}, PI vs q_u	N_{60}, PI, Depth vs q_u
R^2	0.743	0.747	0.772	0.775	0.727	0.725	0.752	0.751
RMSE	21.05521674	21.43928464	21.0649355	21.02753199	22.08429	21.92037389	21.899721	21.76038979

Table 3 different combination of regressions for predicting q_u of Dhaka

Dhaka

	N vs q_u	N, Depth vs q_u	N, PI vs q_u	N, PI, Depth vs q_u	N_{60} vs q_u	N_{60}, Depth vs q_u	N_{60}, PI vs q_u	N_{60}, PI, Depth vs q_u
R^2	0.797	0.827	0.796	0.828	0.794	0.83	0.793	0.831
RMSE	35.27746606	33.85959385	35.55098733	34.49771892	35.97883	33.89516797	36.1850782	34.4602637

For Sylhet, the model we chose is N, PI, Depth vs q_u and for Dhaka, the model we chose is N_{60}, PI, Depth vs q_u.

Angle of internal friction Ø

For both regions, we made six different models using different combinations of independent variables to determine Ø and chose the best model best model based on R^2 and RMSE

Table 4 different combination of regressions for predicting Ø of Sylhet

Sylhet

	N vs Ø	N, Depth vs Ø	N, Fines% vs Ø	N, Fines%, Depth vs Ø	$N_{1,60}$ vs Ø	$N_{1,60}$, Fines% vs Ø
R^2	0.898	0.898	0.899	0.899	0.804	0.804
RMSE	1.251201	1.270144431	1.274102566	1.25774559	1.88355	1.892607258

Table 5 different combination of regressions for predicting Ø of Dhaka

Dhaka	N vs Ø	N, Depth vs Ø	N, Fines% vs Ø	N, Fines%, Depth vs Ø	$N_{1,60}$ vs Ø	$N_{1,60}$, Fines% vs Ø
R^2	0.629	0.626	0.63	0.627	0.232	0.224
RMSE	1.413334	1.415967622	1.401273162	1.403544751	1.8352206	1.835920356

For Sylhet, the model we chose is N, Fines%, Depth vs Ø and for Dhaka, the model we chose is N, Fines% vs Ø and for Dhaka we chose N, Fines% vs Ø.

MLR models

Out of our chosen models below is the description in details of the models. In the regression analysis, 70% of the data were taken for training and 30% for testing. The regression was conducted using the OLS method.

Dhaka

Table 6 MLR model for predicting q_u of Dhaka

MODEL	OLS	R-SQUARED	0.833	No. Observation	93
DEPENDENT VARIABLE	q_u	ADJ. R-SQUARED	0.830	F-STAT	225.0
PROB (F-STAT)	9.67e-36	SKEW	1.007	KURTOSIS	7.042
	Coef.	Std.Err	t	P>[t]	
constant	28.8868	7.6049	3.7985	0.0003	
N_{60}	13.9646	0.6592	20.9247	0.0000	
Depth(m)	-2.2666	0.5068	-4.4726	0.0000	

$$q_u(\mathrm{kPa}) = 13.9646\ N_{60} - 2.2666\ \mathrm{Depth(m)} + 28.8868$$

Here 93 variables of the training set were used to develop this model. After training the adjusted R squared decreased to 0.830, which indicated 83.0% of the variability in q_u is explained by N_{60} and depth(m). The F statistic is highly significant, which means at least one of the independent variables has a significant relationship with q_u. Here, all the independent variables have a p value of less than 0.05 for the t-stat, thus they all have a significant relationship with q_u at 5% level of significance. The skew value of 1.007 suggests that the data is moderately skewed. And a kurtosis value of 7.0442 indicates that the dataset has heavier tails than a normal distribution.

Table 7 MLR model for predicting Ø of Dhaka

MODEL	**OLS**	**R-SQUARED**	**0.638**	**No. Observation**	**97**
DEPENDENT VARIABLE	Ø	**ADJ. R-SQUARED**	**0.630**	**F-STAT**	**82.72**
PROB (F-STAT)	**1.90e-21**	**SKEW**	**-0.037**	**KURTOSIS**	**2.904**
	Coef.	**St. Err**	**t**	**P>[t]**	
constant	**26.8495**	**0.6074**	**44.2060**	**0.0000**	
N	**13.9647**	**0.6699**	**20.8450**	**0.0000**	
Fines%	**0.0165**	**0.0146**	**1.1321**	**0.2605**	

$$Ø = 13.9647\ N + 0.0165\ Fines\% + 26.8495$$

Here 97 variables of the training set were used to develop this model. After training the adjusted R squared decreased to 0.630, which indicated 63.0% of the variability in Ø is explained by N and fines%. The F statistic is highly significant, which means at least one of the independent variables has a significant relationship with Ø. Here, only N has a p value of less than 0.05 for the t-stat, thus have a significant relationship with Ø at 5% level of significance. The skew value of -0.037 suggests that the data is fairly symmetrical. And a kurtosis value of 2.904 indicates that the dataset has lighter tails than a normal distribution.

Sylhet

Log transformation of Depth(m): before caring out the regression analysis for determining Ø, the dataset of depth(m) has been converted to log so that it is closer to that of a normal distribution and the effect of skewness on the regression can be reduced.

Before **After**

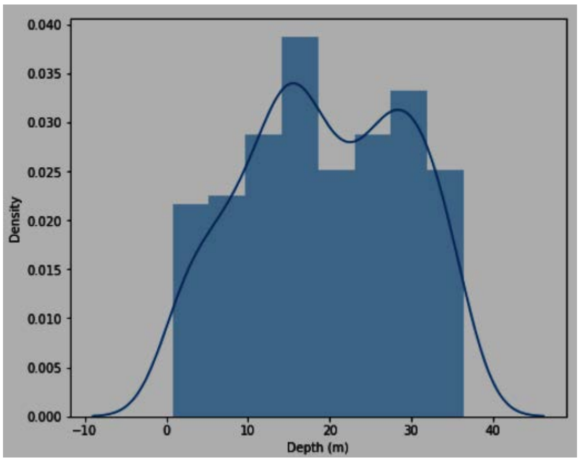


Figure 22 Data distribution of Depth

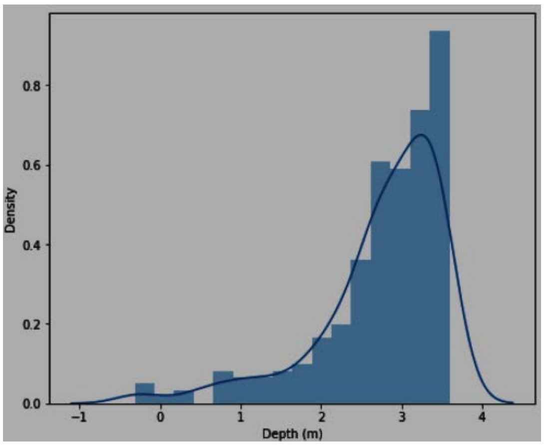


Figure 22 Log transformation of Depth

Table 8 MLR model for predicting Ø of Sylhet

MODEL	OLS	R-SQUARED	0.901	No. Observation	174
DEPENDENT VARIABLE	Ø	ADJ. R-SQUARED	0.899	F-STAT	513.1
PROB (F-STAT)	6.24e-85	SKEW	-0.784	KURTOSIS	7.345
	Coef.	Std.Err	t	P>[t]	
constant	29.6132	0.4266	69.4116	0.0000	
N	0.2601	0.0091	28.5754	0.0000	
Depth(m)	0.1369	0.1609	0.8506	0.3962	
Fines%	-0.0181	0.0101	-1.7940	0.0745	

$$Ø = 0.2601N + 0.1369\text{Log}(\text{Depth(m)}) - 0.0181\text{Fines\%} + 29.6132$$

Here 174 variables of the training set were used to develop this model. After training the adjusted R squared decreased to 0.899, which indicated 89.9% of the variability in Ø is explained by N, Depth(m) and fines%. The F statistic is highly significant, which means at least one of the independent variables has a significant relationship with Ø. Here, only N has a p value of less than 0.05 for the t-stat and Fines% also has p value close to 0.05 thus they both have a significant relationship with Ø at 5% level of significance. The skew value of -0.784 suggests that the data is moderately skewed. And a kurtosis value of 7.345 indicates that the dataset has heavier tails than a normal distribution.

Log transformation of PI: before caring out the regression analysis for determining q_u, the dataset of PI has been converted to log so that it is closer to that of a normal distribution and the effect of skewness on the regression can be reduced.

Before

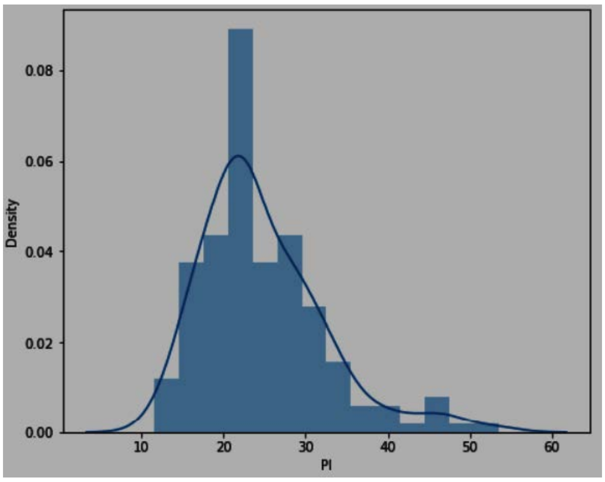


Figure 22 Data distribution of PI

After

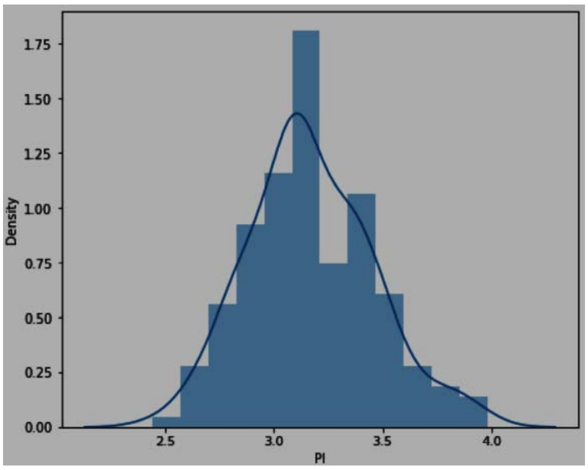


Figure 23 Log transformation of PI

Table 9: MLR model for predicting q_u of Sylhet

MODEL	OLS	R-SQUARED	0.781	No. Observation	117
DEPENDENT VARIABLE	q_u	ADJ. R-SQUARED	0.775	F-STAT	134.5
PROB (F-STAT)	3.86e-37	SKEW	0.460	KURTOSIS	3.802
	Coef.	St. Err	t	P>[t]	
constant	-73.5638	23.8579	-3.0834	0.0026	
N	10.7554	0.5428	19.8158	0.0000	
Depth(m)	1.6036	0.9279	1.7281	0.0867	
PI	28.6540	7.3066	3.9217	0.0002	

$$q_u = 10.7554N + 1.6036Depth(m) + 28.6540Log\ (PI) - 73.5638$$

Here 117 variables of the training set were used to develop this model. After training the adjusted R squared decreased to 0.775, which indicated 77.5% of the variability in q_u is explained by N, Depth(m) and PI. The F statistic is highly significant, which means at least one of the independent variables has a significant relationship with q_u. Here, only N and PI has a p value of less than 0.05 for the t-stat and Depth(m) also has p value close to 0.05 thus they all have a significant relationship with q_u at 5% level of significance. The skew value of 0.460 suggests that the data is fairly symmetrical. And a kurtosis value of 3.802 indicates that the dataset is closer to a normal distribution.

SVR models

Of our chosen four MLR models we carried out SVR then determined which model to keep. Below is the tabulation of the regression done by two of the platforms of our chosen models.

Table 11 Model for predicting Ø of Sylhet

Sylhet

N, Fines%, Depth vs Ø		
R^2	0.899	0.92
RMSE	1.25774559	1.2311
Platform	Multi-Linear Regression	SVR

Table 10 Model for predicting q_u of Sylhet

Sylhet

N, PI, Depth vs q_u		
R^2	0.775	0.83
RMSE	21.96333465	26.189
Platform	Multi-Linear Regression	SVR

Table 12 Model for predicting Ø of Dhaka

Dhaka

N, Fines%, vs Ø		
R^2	0.64	0.58
RMSE	1.410423197	1.4845
Platform	Multi-Linear Regression	SVR

Table 13 Model for predicting q_u of Dhaka

Dhaka

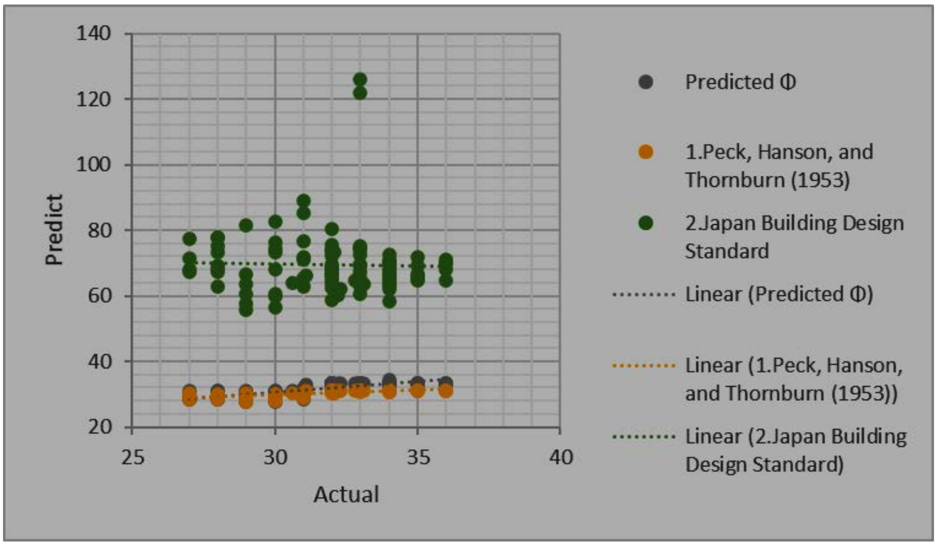
N_{60}, Depth vs q_u		
R^2	0.83	0.92
RMSE	36.29372922	27.246
Platform	Multi-Linear Regression	SVR

Here, we can observe that only for N, Fines%, vs Ø model we have chosen MLR due to higher R^2 and smaller RMSE while for other three models we have chosen SVR.

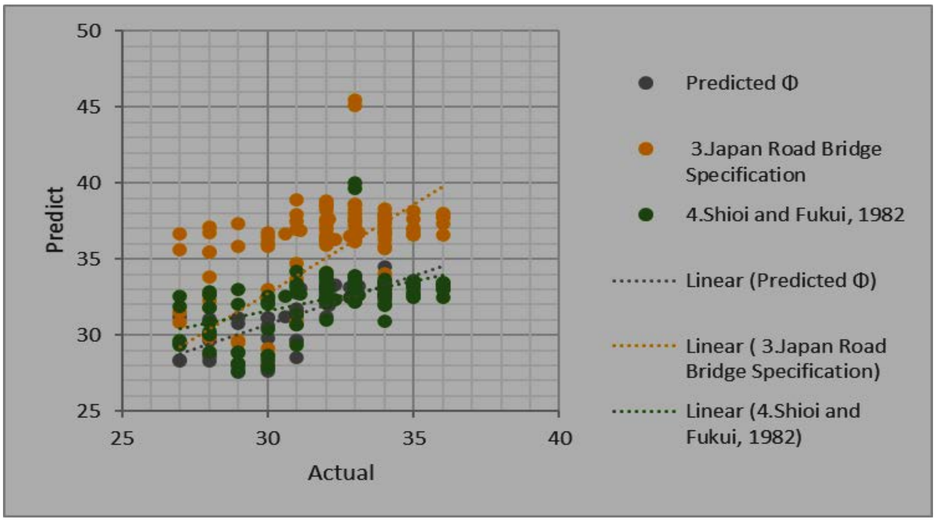
Comparison with others

The predicted results that we obtained, we plotted it against the actual results and obtained the R^2 and RMSE. Then we compared it with other published models and verified that our models have provided better results in predicting q_u and Ø.

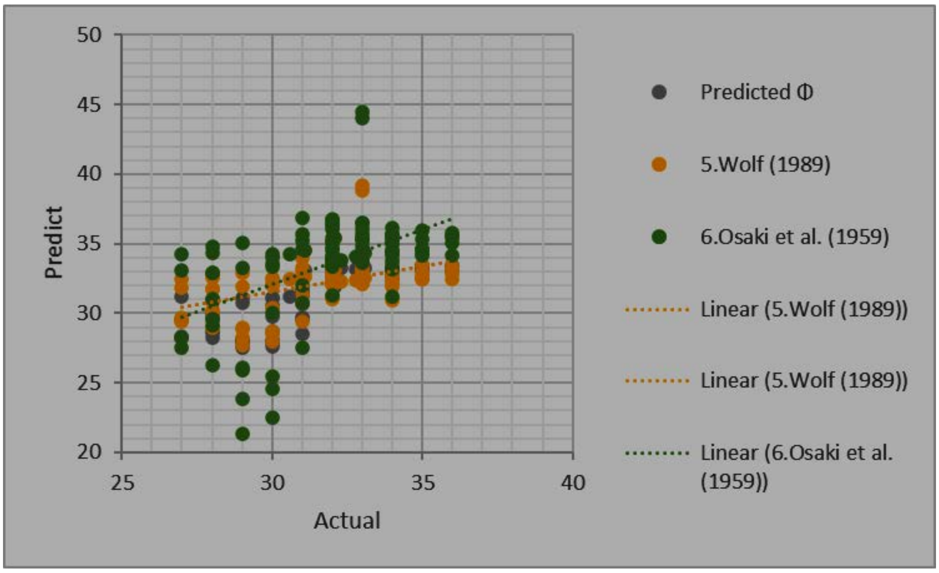
Ø of Dhaka



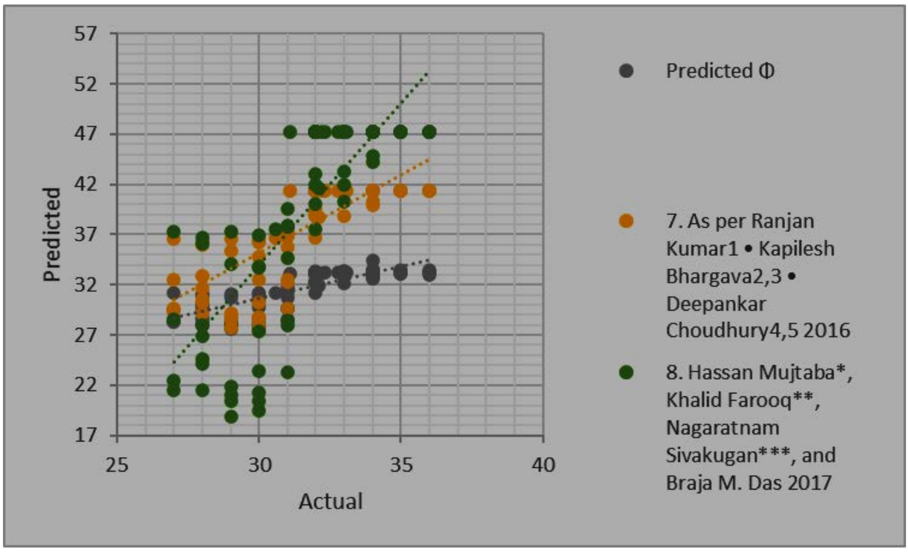
140
120
100
80
60
40
20
Predict
25
30
35
40
Actual
Predicted Φ
1.Peck, Hanson, and Thornburn (1953)
2.Japan Building Design Standard
Linear (Predicted Φ)
Linear (1.Peck, Hanson, and Thornburn (1953))
Linear (2.Japan Building Design Standard)



50
45
40
35
30
25
Predict
25
30
35
40
Actual
Predicted Φ
3.Japan Road Bridge Specification
4.Shioi and Fukui, 1982
Linear (Predicted Φ)
Linear (3.Japan Road Bridge Specification)
Linear (4.Shioi and Fukui, 1982)



50
45
40
35
30
25
20
Predict
25
30
35
40
Actual
Predicted Φ
5.Wolf (1989)
6.Osaki et al. (1959)
Linear (5.Wolf (1989))
Linear (5.Wolf (1989))
Linear (6.Osaki et al. (1959))



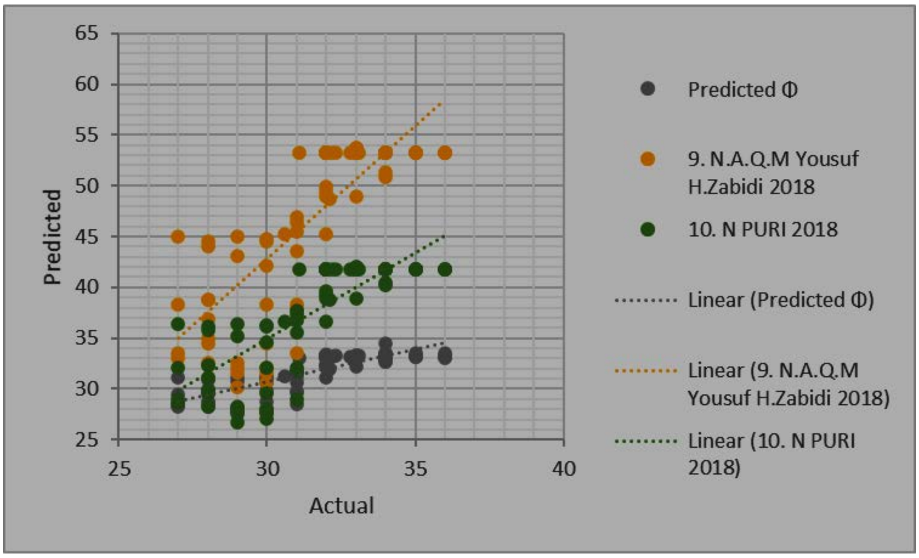


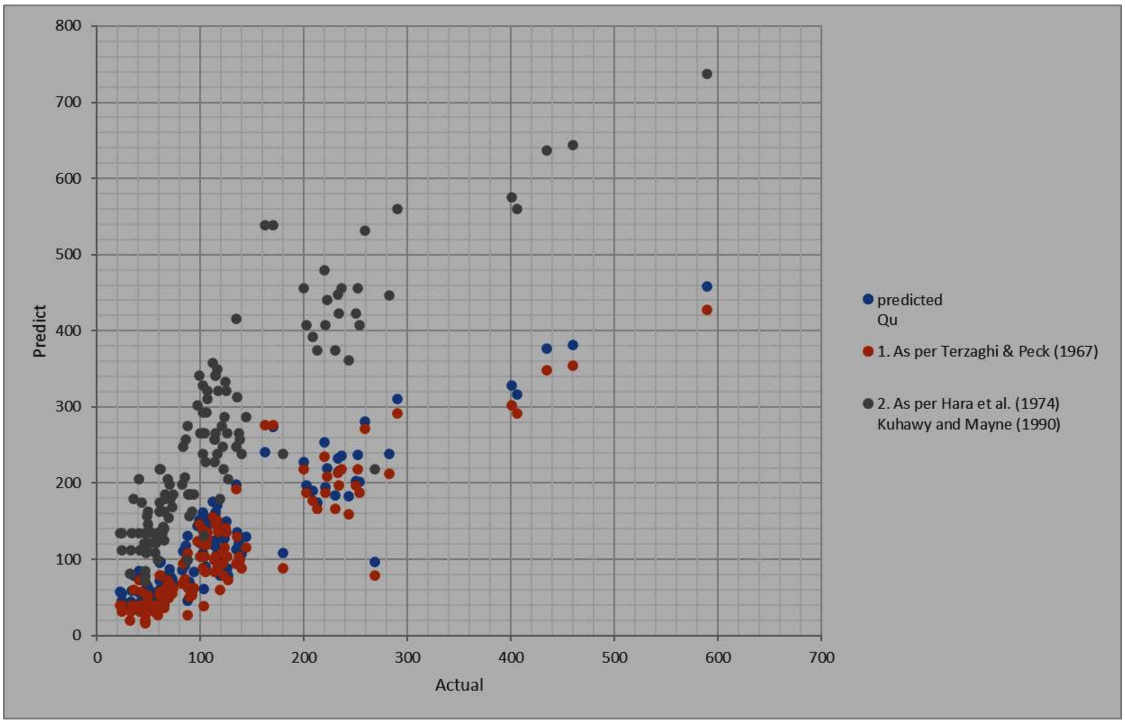
Figure 24 Relation between actual and predicted ϕ by MLR

Table 14 Tabulation of the relation between actual and predicted

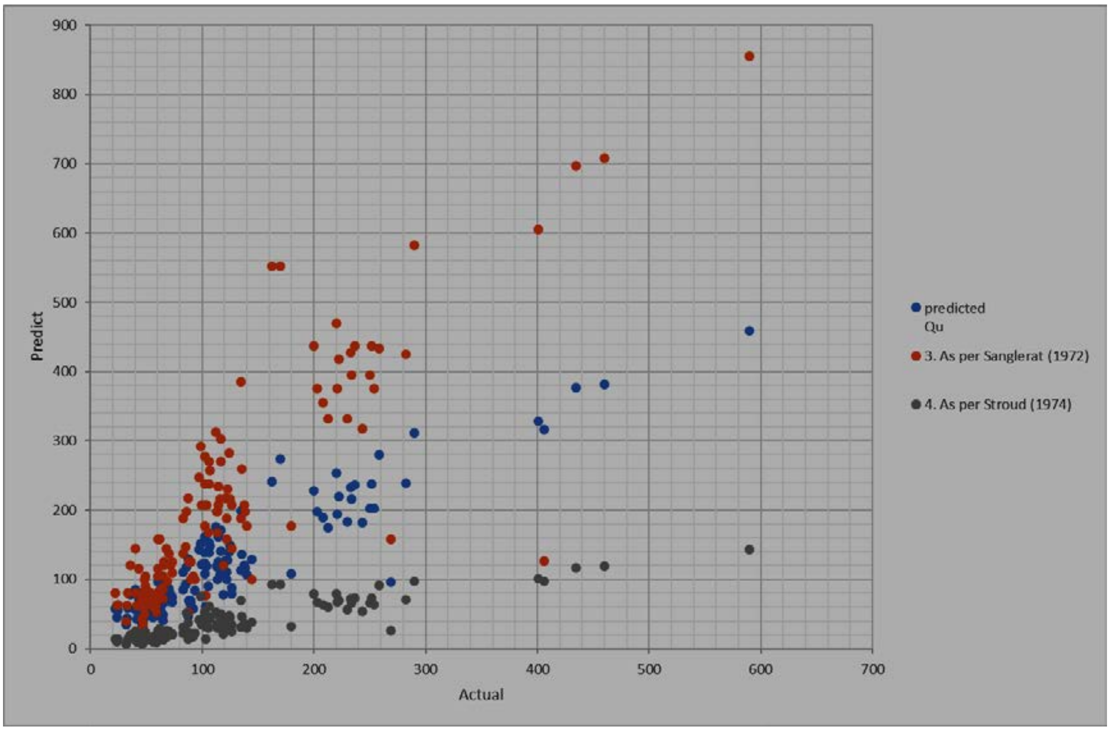
	Ø of Dhaka										
	Predicted	**1. As per Peck, Hanson, and Thornburn (1953)**	**2. As per Japan Building Design Standard**	**3. As per japan Road Bridge Specification**	**4. As per Japan " Design Standards for Structures" (Shioi and Fukui, 1982)**	**5. As per Wolf (1989)**	**6. As per Osaki et al. (1959)**	**7. As per Schmertmann (1975) Kubawy and Mayne (1990)**	**8. Hassan Mujtaba*, Khalid Farooq**, Nagaratnam Sivakugan***, and Braja M. Das 2017**	**9. N.A.Q.M Yousuf H.Zabidi 2018**	**10. N PURI 2018**
R^2	0.6377	0.5682	0.0009	0.1272	0.2398	0.2641	0.2628	0.6327	0.6368	0.6327	0.6327
RMSE	1.41	2.394	39.084	7.792	2.211	2.149	3.546	6.938	11.308	16.903	7.148

Here, we can see that in terms R^2, our model has the highest value and in terms of RMSE, our model has the smallest value so we can conclude that our model has the highest predictability rate for determining Ø of Dhaka.

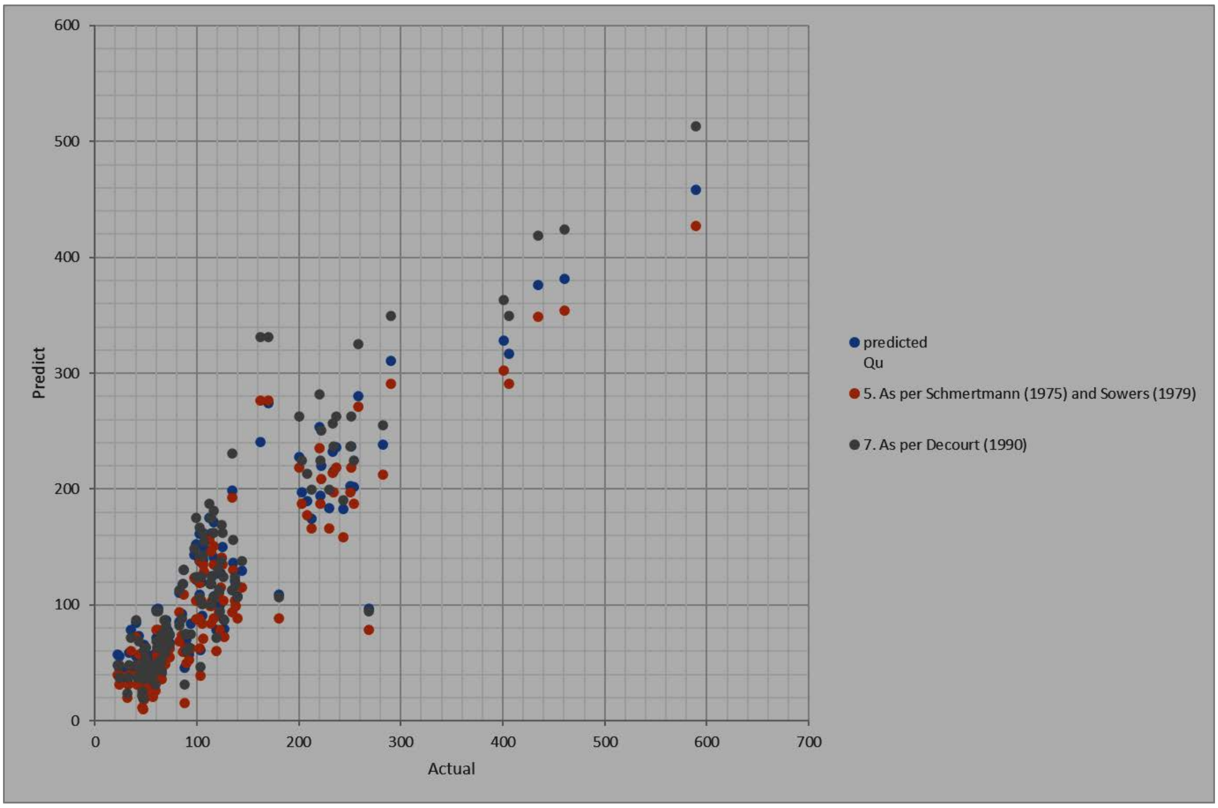
qu for Dhaka



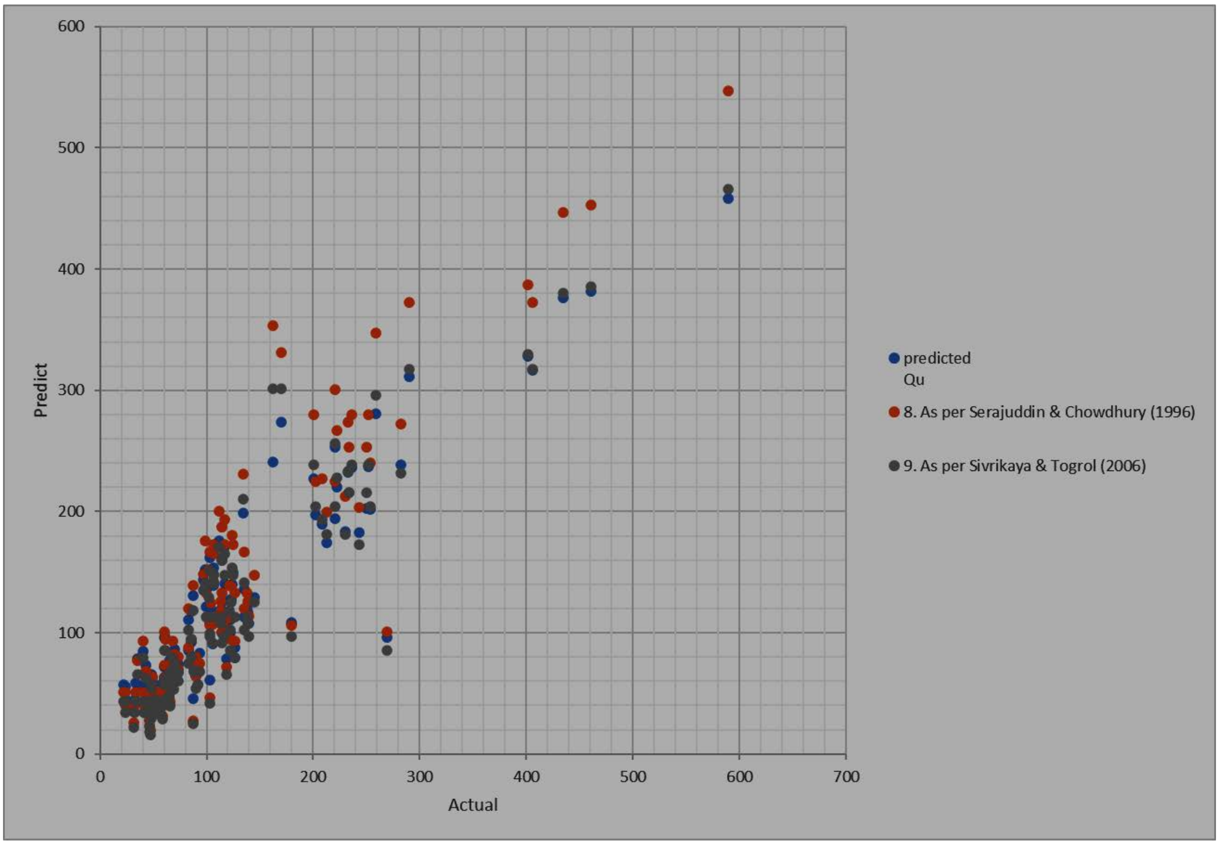
800
700
600
500
400
300
200
100
0
Predict
0
100
200
300
400
500
600
700
Actual
predicted Qu
1. As per Terzaghi & Peck (1967)
2. As per Hara et al. (1974) Kuhawy and Mayne (1990)



900
800
700
600
500
400
300
200
100
0
Predict
0
100
200
300
400
500
600
700
Actual
predicted Qu
3. As per Sanglerat (1972)
4. As per Stroud (1974)



Predict
Actual
predicted Qu
5. As per Schmertmann (1975) and Sowers (1979)
7. As per Decourt (1990)



Predict
Actual
predicted Qu
8. As per Serajuddin & Chowdhury (1996)
9. As per Sivrikaya & Togrol (2006)

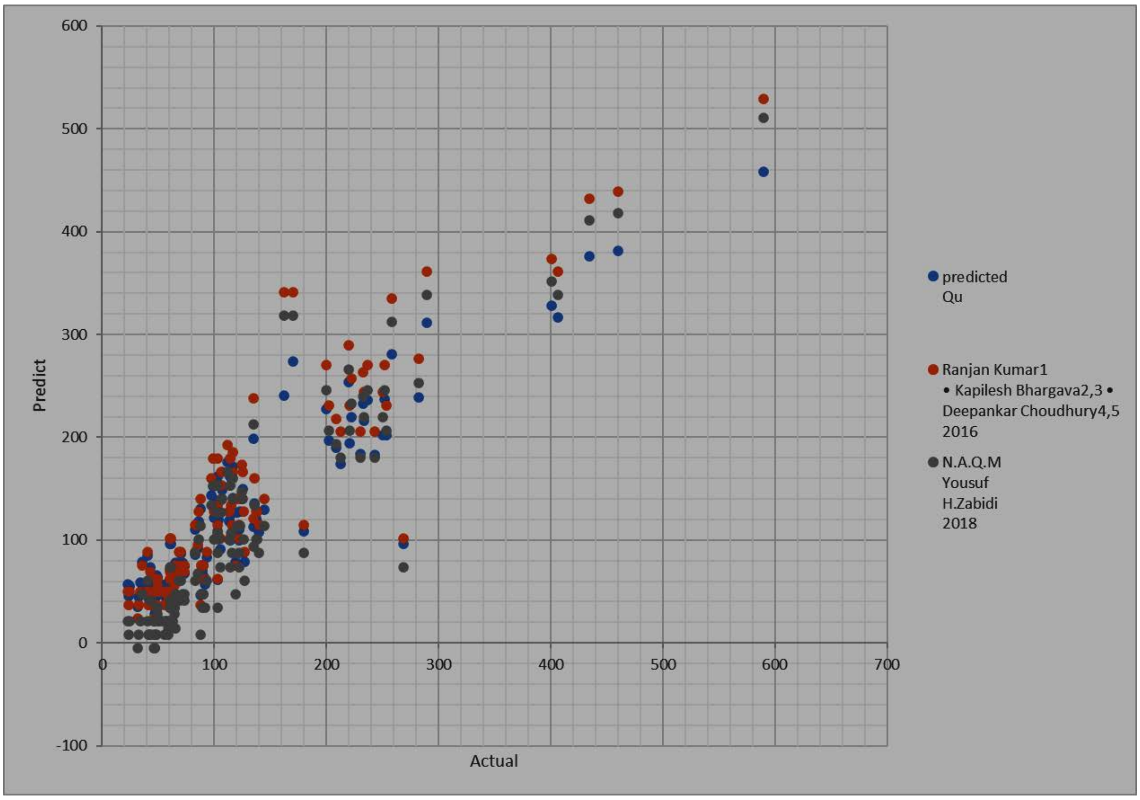


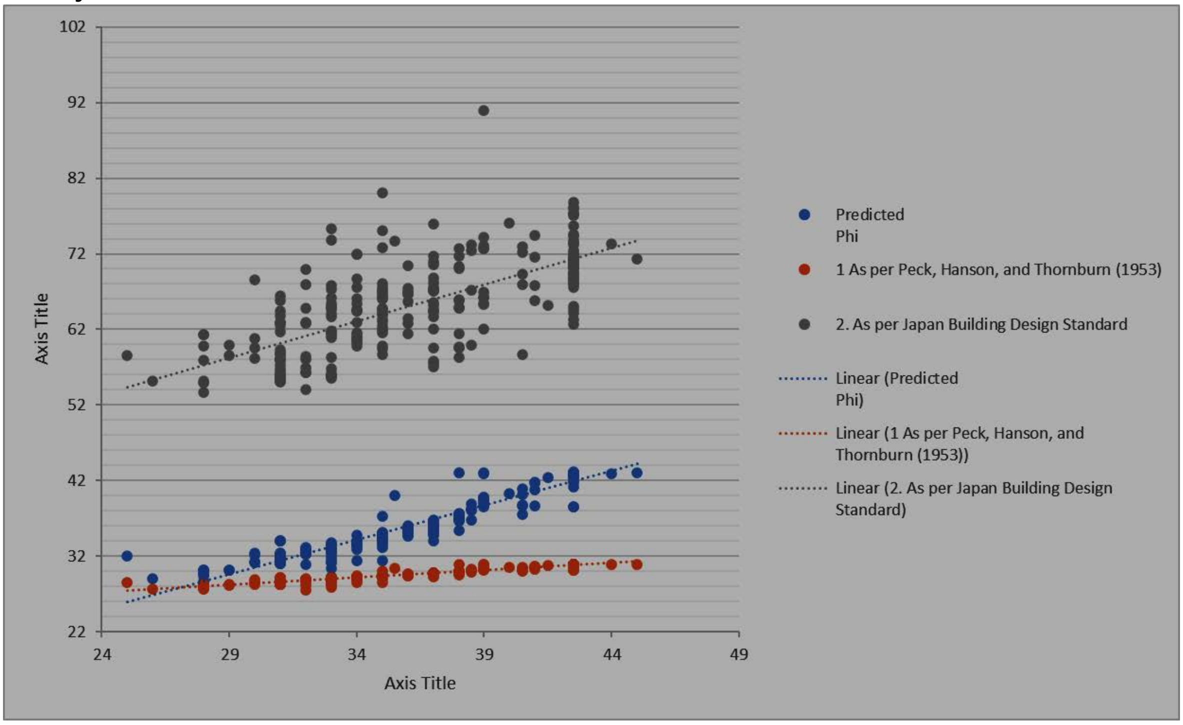
Figure 25 Relation between actual and predicted q_u by SVR

Table 15 Tabulation of the relation between actual and predicted

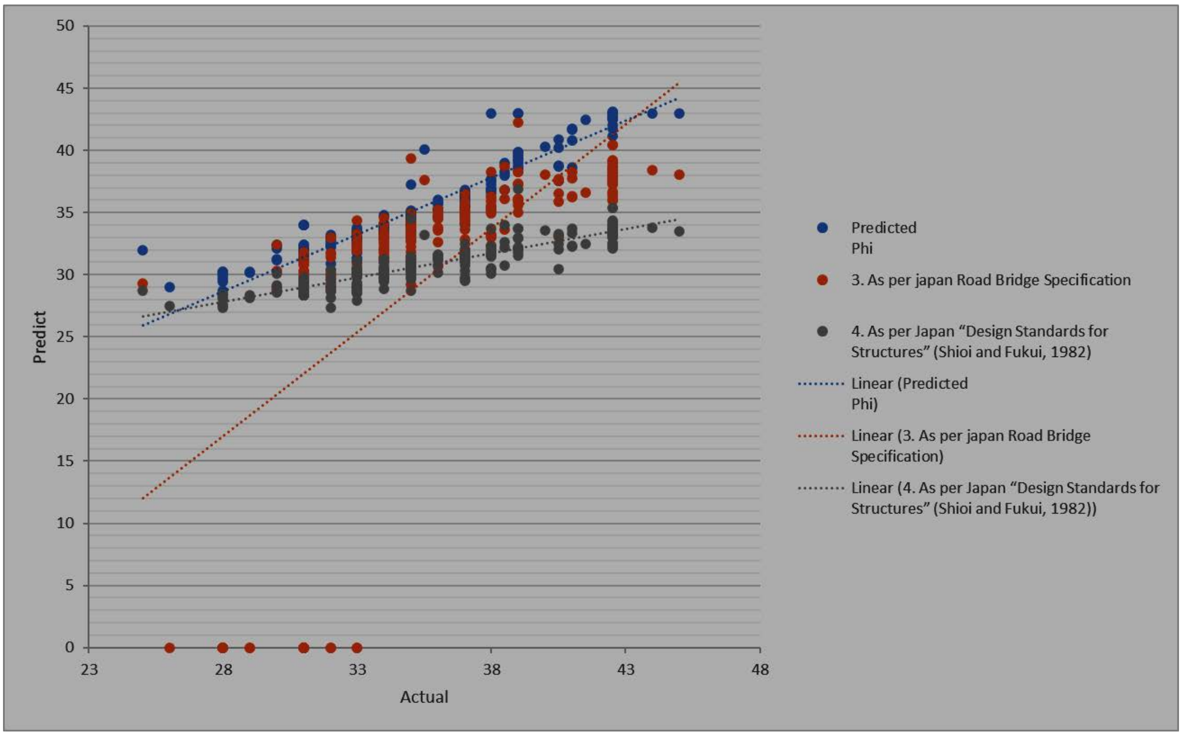
	q_u of Dhaka										
	Predicted	**1. As per Terzaghi & Peck (1967)**	**2. As per Hara et al. (1974) Kuhawy and Mayne (1990)**	**3. As per Sanglerat (1972)**	**4. As per Stroud (1974)**	**5. As per Schmertmann (1975) and Sowers (1979)**	**7. As per Decourt (1990)**	**8. As per Serajuddin & Chowdhury (1996)**	**9. As per Sivrikaya & Togrol (2006)**	**Ranjan Kumar1 • Kapilesh Bhargava2,3 • Deepankar Choudhury4,5 2016**	**N.A.Q.M Yousuf H.Zabidi 2018**
R^2	0.8495	0.8275	0.7948	0.741	0.7935	0.8294	0.8294	0.8275	0.83	0.8312	0.8317
RMSE	36.5	41.997	45.071	119.186	108.334	42.563	118.644	40.276	38.98	41.557	44.384

Here, we can see that in terms R^2, our model has the highest value and in terms of RMSE, our model has the smallest value so we can conclude that our model has the highest predictability rate for determining q_u of Dhaka.

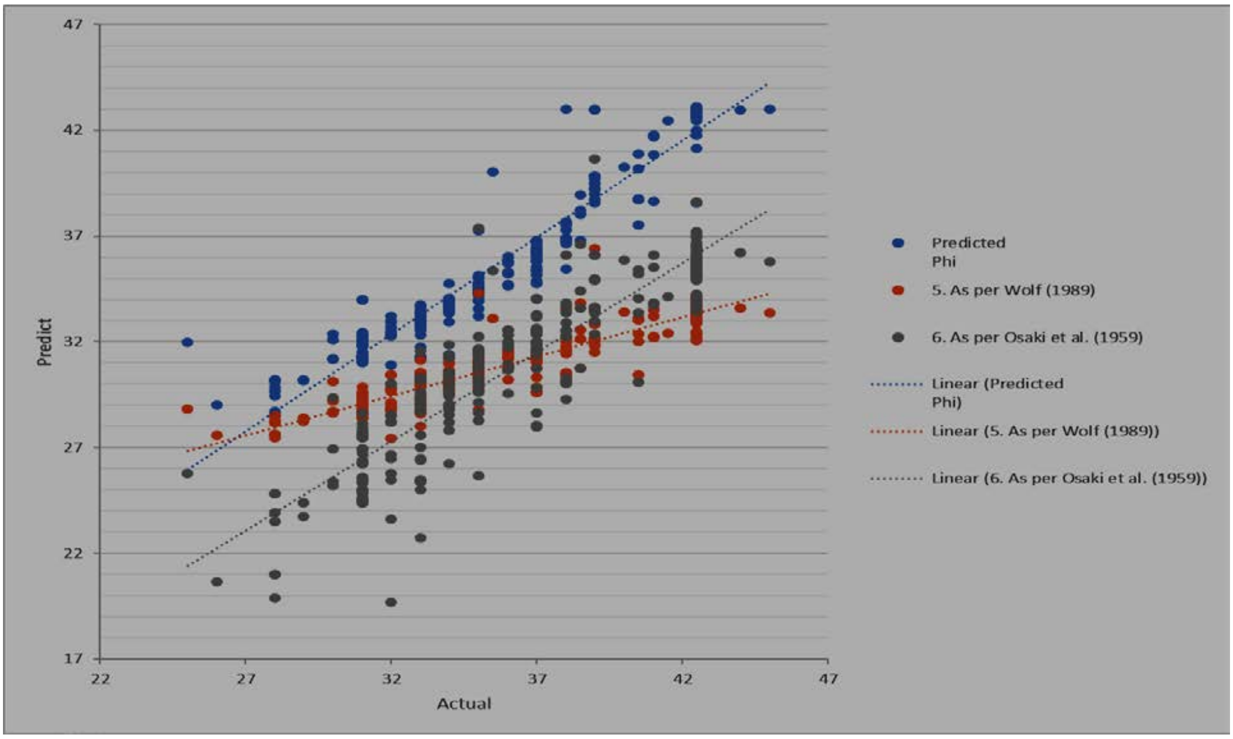
Ø of Sylhet



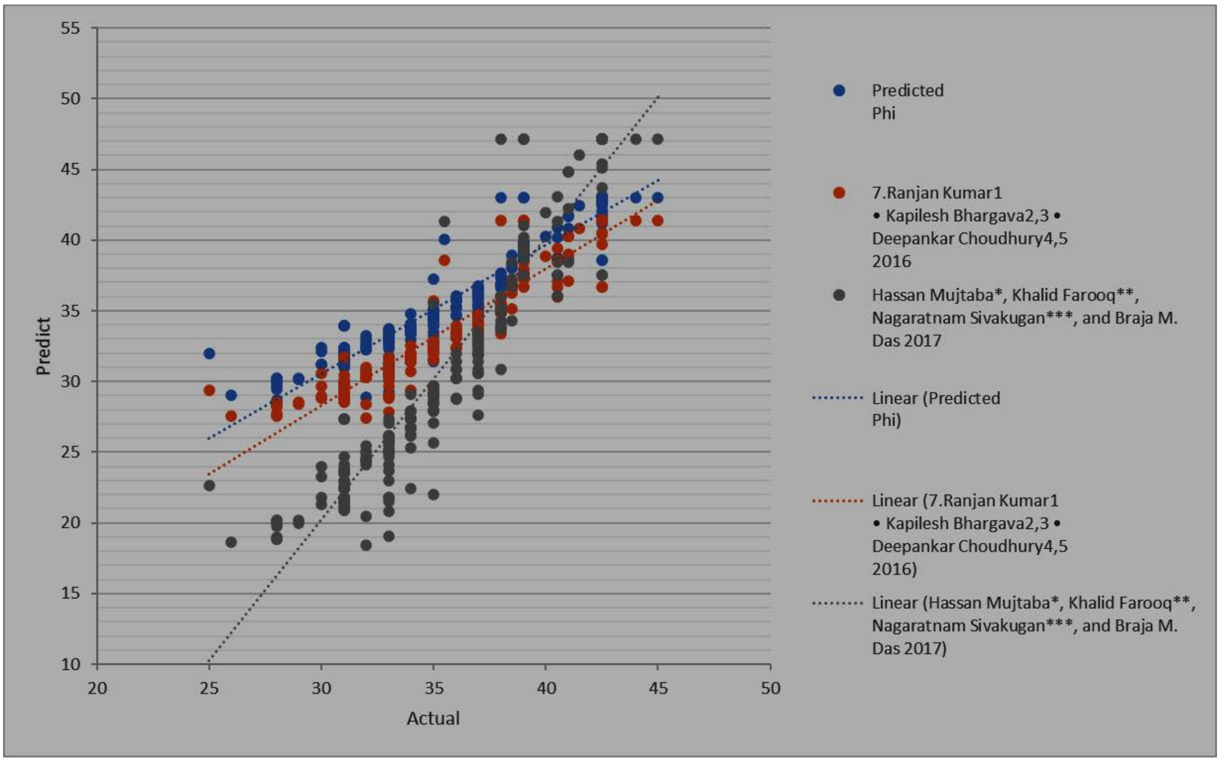
102
92
82
72
62
52
42
32
22
24
29
34
39
44
49
Axis Title
Axis Title
Predicted Phi
1 As per Peck, Hanson, and Thornburn (1953)
2. As per Japan Building Design Standard
Linear (Predicted Phi)
Linear (1 As per Peck, Hanson, and Thornburn (1953))
Linear (2. As per Japan Building Design Standard)



50
45
40
35
30
25
20
15
10
5
0
23
28
33
38
43
48
Actual
Predict
Predicted Phi
3. As per japan Road Bridge Specification
4. As per Japan "Design Standards for Structures" (Shioi and Fukui, 1982)
Linear (Predicted Phi)
Linear (3. As per japan Road Bridge Specification)
Linear (4. As per Japan "Design Standards for Structures" (Shioi and Fukui, 1982))



47
42
37
32
27
22
17
Predict
22
27
32
37
42
47
Actual
Predicted Phi
5. As per Wolf (1989)
6. As per Osaki et al. (1959)
Linear (Predicted Phi)
Linear (5. As per Wolf (1989))
Linear (6. As per Osaki et al. (1959))



55
50
45
40
35
30
25
20
15
10
Predict
20
25
30
35
40
45
50
Actual
Predicted Phi
7.Ranjan Kumar1 • Kapilesh Bhargava2,3 • Deepankar Choudhury4,5 2016
Hassan Mujtaba*, Khalid Farooq**, Nagaratnam Sivakugan***, and Braja M. Das 2017
Linear (Predicted Phi)
Linear (7.Ranjan Kumar1 • Kapilesh Bhargava2,3 • Deepankar Choudhury4,5 2016)
Linear (Hassan Mujtaba*, Khalid Farooq**, Nagaratnam Sivakugan***, and Braja M. Das 2017)

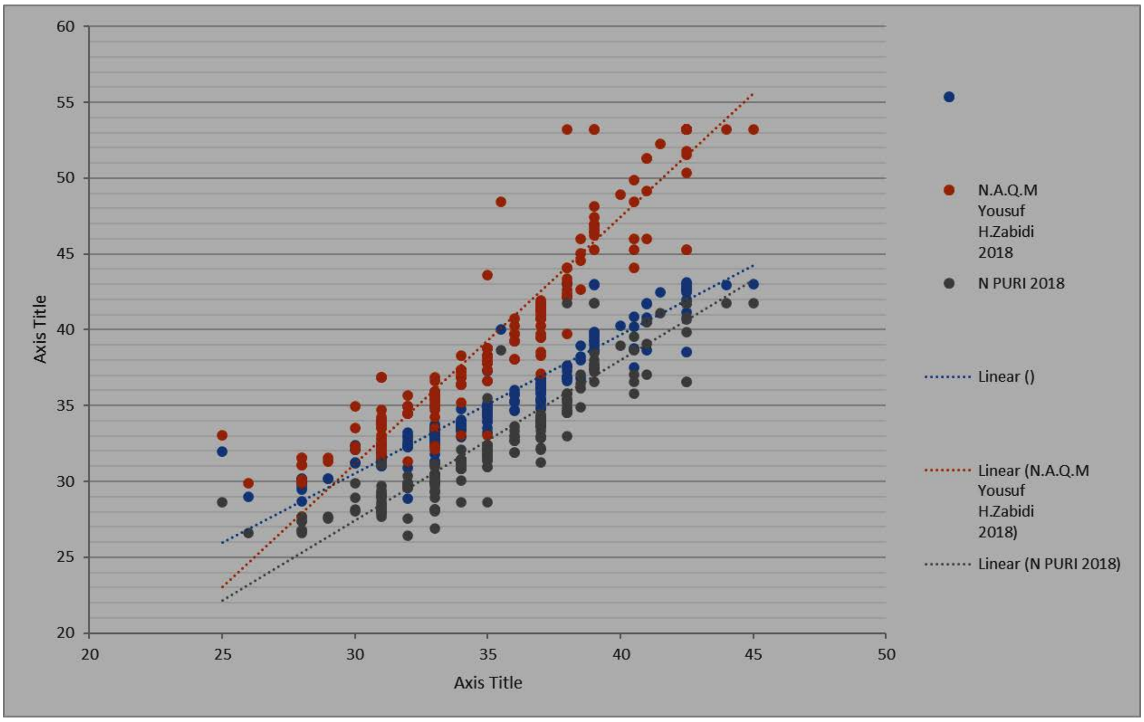


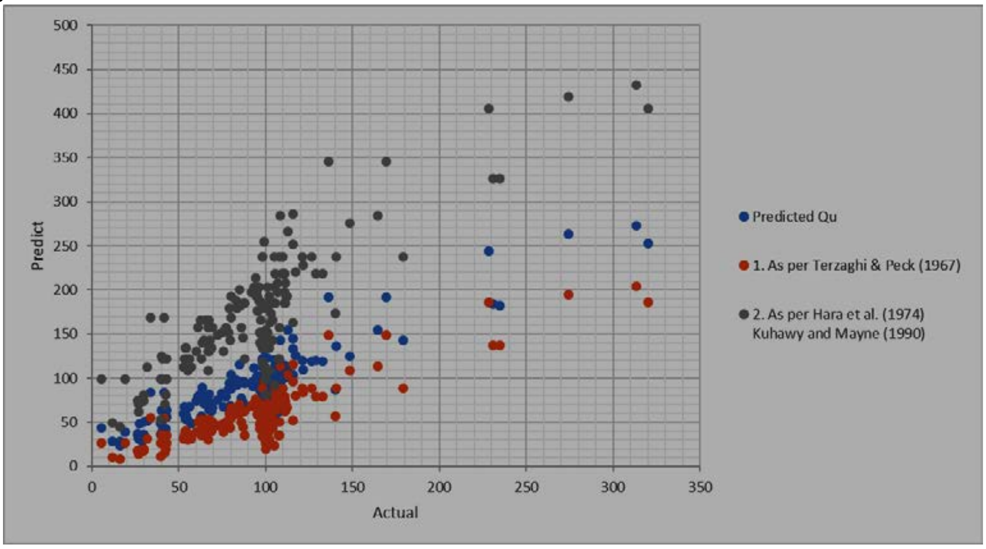
Figure 26 Relation between actual and predicted ϕ by SVR

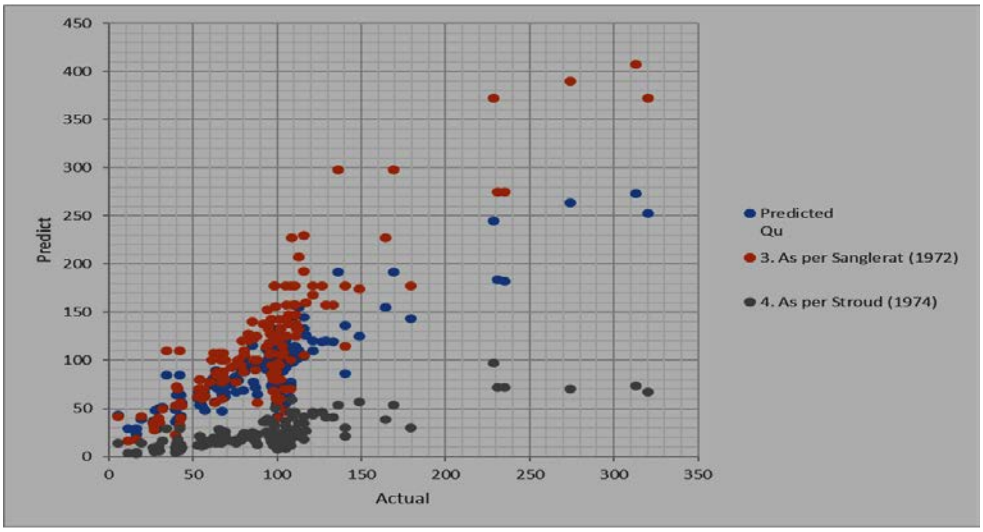
Table 16 Tabulation of the relation between actual and predicted

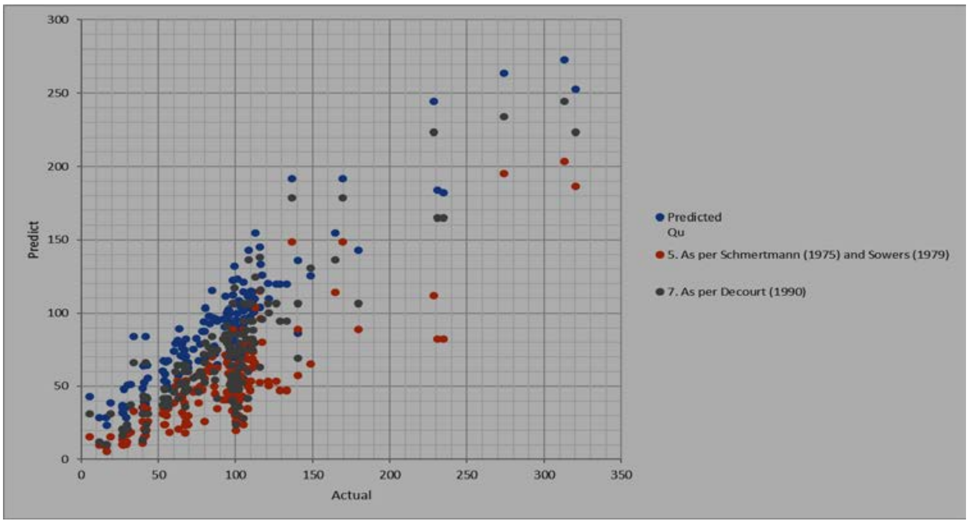
	Ø of Sylhet										
	Predicted	**1. As per Peck, Hanson, and Thornburn (1953)**	**2. As per Japan Building Design Standard**	**3. As per japan Road Bridge Specification**	**4. As per Japan " Design Standards for Structures" (Shioi and Fukui, 1982)**	**5. As per Wolf (1989)**	**6. As per Osaki et al. (1959)**	**7. As per Ranjan Kumar1 • Kapilesh Bhargava2,3 • Deepankar Choudhury4,5 2016**	**8. Hassan Mujtaba*, Khalid Farooq**, Nagaratnam Sivakugan***, and Braja M. Das 2017**	**9. N.A.Q.M Yousuf H.Zabidi 2018**	**10. N PURI 2018**
R^2	0.9071	0.9142	0.4688	0.4233	0.8105	0.8122	0.8018	0.9056	0.9064	0.9056	0.9056
RMSE	1.35	7.338	29.386	2.957	5.771	5.807	5.686	2.348	6.502	6.098	2.711

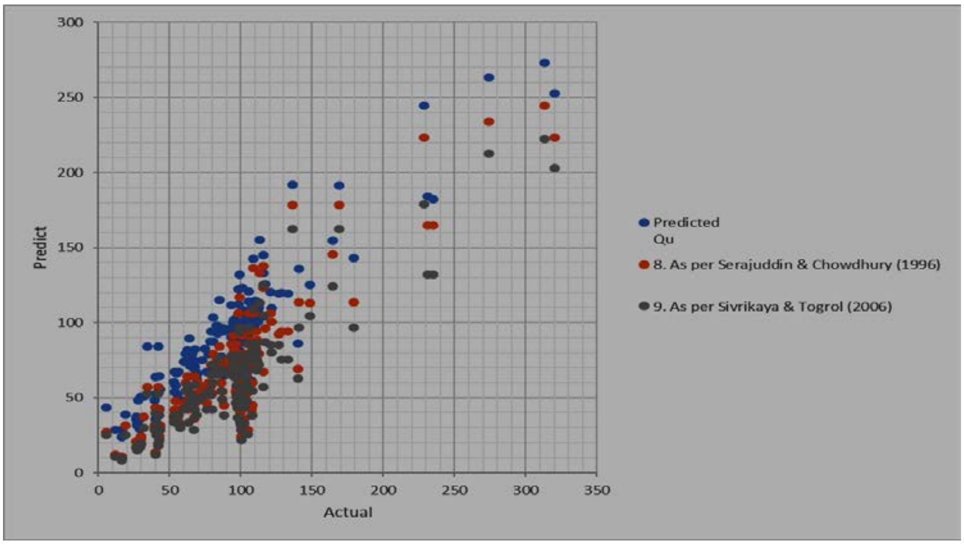
Here, we can see that in terms R^2, our model has the highest value compared to all other models except Peck, Hanson, and Thornburn (1953) had a higher score for R^2 and in terms of RMSE, our model has the smallest value so we can conclude that our model has the highest predictability rate for determining Ø of Dhaka.

q_u for Sylhet









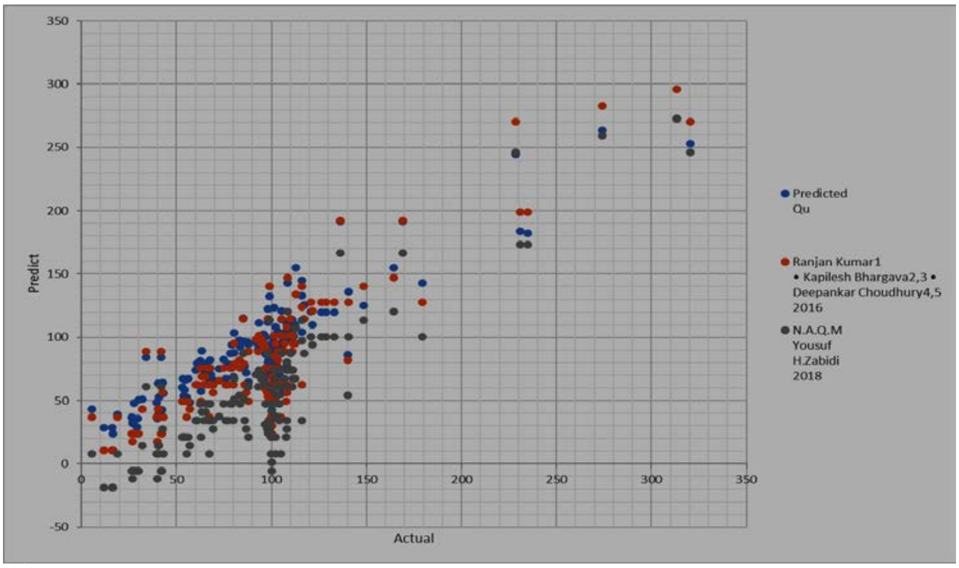


Figure 25 Relation between actual and predicted q_u by SVR

Table 17 Tabulation of the relation between actual and predicted

	q_u of Sylhet										
	Predicted	**1. As per Terzaghi & Peck (1967)**	**2. As per Hara et al. (1974) Kulhawy and Mayne (1990)**	**3. As per Sanglerat (1972)**	**4. As per Stroud (1974)**	**5. As per Schmertmann (1975) and Sowers (1979)**	**7. As per Decourt (1990)**	**8. As per Serajuddin & Chowdhury (1996)**	**9. As per Sivrikaya & Togrol (2006)**	**Ranjan Kumar1 • Kapilesh Bhargava2,3 • Deepankar Choudhury4,5 2016**	**N.A.Q.M Yousuf H. Zabidi 2018**
R^2	0.7874	0.745	0.7265	0.7483	0.6018	0.6959	0.745	0.7562	0.7551	0.7617	0.7617
RMSE	21.627	42.098	86.291	42.008	77.95	36.741	32.747	32.471	39.125	26.521	46.411

Here, we can see that in terms R^2, our model has the highest value and in terms of RMSE, our model has the smallest value so we can conclude that our model has the highest predictability rate for determining q_u of Sylhet.

CHAPTER 5 : CONCLUSION

General

In this chapter we have included the summary and the overview of our study. Here, we have elaborately discussed our findings, limitations of our established equations and recommendations of any future studies that can be done. We used about 700 data to develop these four models of the two different regions each with different types of soil to conduct this research.

Findings

- the F-stats value we have obtained for all the models is highly significant. So at least one independent variable has a significant relationship with the dependent variable.
- At least one in independent variable has a p-value less than 0.05 so they have a significant relationship with the independent variable at 5% level of significance.
- Our models have higher R^2 and RMSE values compared to other models so they have the highest predictability rate.

Limitations

- Some of our Datasets for models like Ø of Sylhet and q_u of Dhaka are moderately skewed which affects the regression model.
- We have found presence of multicollinearity in the model Ø of Sylhet which can lead to misleading results.
- The SVR model has no equation or graph thus it cannot be applied without a Web based application.
- The sample we used for developing the models in Dhaka region was small so the training and testing of data we have carried out maybe unreliable.
- We cannot guarantee the reliability of the data we have collected.

Recommendations

- Using a larger Sample Size will result in better holdout validation thus reliability will increase.
- Other soil parameters like D_{10}, D_{30}, D_{60}, E_s, v, V_s, C_c, C_s that affects q_u & Ø should be incorporated into the model.
- Interactive application can be created for using SVR so that field engineers can apply our model on the go without having to know any machine learning algorithms.

REFERENCES

[1] Md. Jahangir Alam, N. R. (2015). Relationship between standard penetration resistance and strength-compressibility parameters of clay. Journal of Civil Engineering (IEB), 115-131.

[2] KAMIMURA MAKOTO, T. T. (2013). Relationships between N value and parameters of ground strength in the South of Vietnam. Geotechnics for Sustainable Development.

[3] M. Serajuddin, M. A. (1998). Correlation Between Standard Penetration Resistance and Unconfined Compressive Strength of Bangladesh Cohessive Soil Deposits. Journal of Civil Engineering (IEB), CE-24(1), 69-83.

[4] Mahmoud, M. A. (2013). Reliability of using standard penetration test (SPT) in predicting properties of silty clay with sand soil. INTERNATIONAL JOURNAL OF CIVIL AND STRUCTURAL ENGINEERING, 3(3), 545-556.

[5] Pouya Salari, G. R. (2015). Presentation of Empirical Equations for Estimating Internal Friction Angle of GW and GC Soils in Mashhad, Iran Using Standard Penetration and Direct Shear Tests and Comparison with Previous Equations. Open Journal of Geology, 231-238.

[6] Ranjan Kumar, K. B. (2016). Estimation of Engineering Properties of Soils from Field SPT Using Random Number Generation. INEA Letters, 77-84.

[7] Tansir Zaman Asik, M. R. (2016). Correlation of Soil Parameters for the Proposed Dhaka - Chittagong Elevated Expressway. BUET-ANWAR ISPAT 1st Bangladesh Civil Engineering SUMMIT 2016. Dhaka: BUET.

[8] Terzaghi, K. (1943). Theoretical Soil Mechanics. Wiley, New York.

[9] O. Sivrikaya, E. T. (2006). DETERMINATION OF UNDRAINED STRENGTH OF FINE-GRAINED SOILS BY MEANS OF SPT AND ITS APPLICATION IN TURKEY. Engineering Geology, 52-69.

[10] Frazad Nassaji, B. K. (2011). SPT Capability to Estimate Undrained Shear Strength of Fine-Grained Soils of Tehran, Iran. Electronic Journal of Geotechnical Engineering, 1229-1237.

[11] Kanagaratnam Balachandran, J. L. (2017). Statistical Correlations between undrained shear strength (CU) and both SPT- N value and net limit pressure (PL) for cohesive glacial tills. GEO OTTAWA.

[12] N.Q.A.M. Yusof, H. (n.d.). Reliability of Using Standard Penetration Test (SPT) in Predicting Properties of Soil. Journal of Physics: Conference Series.

[13] Bowles, J. (1968). Foundation Analysis and Design, McGraw-Hill, New York.

[14] Gibbs, H. J. and Holtz, W. G. (1957). “Research on determining the density of sands by spoon penetration testing.” International Conference on Soil Mechanics and Foundation Eng., Vol. 4, No. 1, pp. 35-39.

[15] Hatanaka, M. and Uchida, A. (1996). “Empirical correlation between penetration resistance and internal friction angle of sandy soils.” Soils and Foundations, Vol. 36, No. 4, pp. 1-9, DOI: 10.3208/ sandf.36.4_1.

[16] Hettiarachchi, H. and Brown, T. (2009). “Use of SPT blow counts to estimate shear strength properties of soils: Energy balance approach.” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, pp. 25-32, DOI: 10.1061/(ASCE)GT.1943-5606.0000016.

[17] Japan Road Association (1990). Specification for Highway Bridges, Part IV.

[18] Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1974). Foundation Engineering, 2nd ed., Wiley, New York.

[19] Wolff, T. F. (1989). “Pile capacity prediction using parameter functions.” in Predicted and Observed Axial Behavior of Piles, Results of a Pile Prediction Symposium, sponsored by Geotechnical Engineering Division, ASCE, Evanston, Ill., June 1989, ASCE Geotechnical Special Publication No. 23, 96-106.

[20] Shioi, Y. and Fukui, J. (1982) Application of N-Value to Design of Foundation in Japan. 2nd ESOPT, Vol. 1, 40-93.

[21] Dunham, J. W. (1954): "Pile foundations for buildings," Proc.ASCE, Soil Mechanics and Foundation Division.

[22] Terzaghi, K., and Peck, R. B. 1967. Soil mechanics in engineering practice, 2nd Ed., Wiley, New York

[23] ASTM. (2008). “Standard test method for standard penetration test (SPT)

[24] Kulhawy, F.H. and Mayne, P.W. (1990). Manual on estimating soil properties for foundation design, Electric Power Research Institute, Palo Alto, CA.

[25] ASTM Committee D-18 on Soil and Rock, 2017. *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) 1*. ASTM International.

[26] A. Hossain, T. Alam, S. Barua and M. R. Rahman (2021). Estimation of shear strength parameter of silty sand from SPT-N60 using machine learning models. https://doi.org/10.1080/17486025.2021.1975048

[27] Puri, N., Prasad, H.D., and Jain, A., 2018. Prediction of geotechnical parameters using machine learning techniques. Procedia Computer Science, 125, 509–517. doi:10.1016/j. procs.2017.12.066

[28] Ranjan Kumar, Kapilesh Bhargava, Deepankar Choudhury (2017). Estimation of Engineering Properties of Soils from Field SPT Using Random Number Generation, DOI 10.1007/s41403-016-0012-6

[29] N.Q.A.M. Yusof, H.Zabidi (2018). Reliability of Using Standard Penetration Test (SPT) in Predicting Properties of Soil, doi:10.1088/1742-6596/1082/1/012094