Determination of undrained shear strength and modulus of elasticity of fine grained soils from SPT in Islamabad, Pakistan

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This is to certify that

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THESIS ACCEPTANCE CERTIFICATE

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Date: ___________________________

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Date____________________________

Signature (Dean/Principal) _________________________

Date: ___________________________

Dedicated to

My Parents, Wife and Children

For their support, love and encouragement

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Abstract

Many researchers report that the Standard Penetration Test (SPT) is not standard as the name implies. However, the test is relatively simple to conduct, produces an indicator of soil resistance (N-value) and provides a soil sample for visual identification and laboratory testing therefore, maintains its popularity with geotechnical engineers worldwide. Due to unavailability of equipment, financial limitations, time limitations and especially when limited data is available use of correlations to determine different parameters prove to be helpful. A number of correlations of N-value, with parameters as relative density, friction angle, allowable bearing capacity, pile skin friction and many others, have been developed. This study is carried out to develop correlation of SPT with undrained shear strength (S_u) and soil modulus of elasticity of fine grained soil in Islamabad, Pakistan. It is tried to conclude how corrected and uncorrected values of SPT influence the parameters to be determined. This study answers the question of SPT applicability in fine grained soils. This work also addresses the correlation between plasticity index and undrained shear strength (S_u). All the correlations are developed using Microsoft Excel 2013 with a focus on the statistics.

Contents

List of Figures

List of Tables

Chapter 01

1. Introduction

1.1. General

Every Civil Engineering structure regardless of its size requires soil and site investigations to varying extent. Mega or sensitive structures would require more detailed investigation as compared to a normal building. The design engineer would be constrained if he does not have sufficient information of soil properties (both index as well as engineering) and behavior. The stability of a structure besides other factors depends on the response of the foundation soil. A better soil investigation program results in better understanding of the soil and reliable design parameter.

Soil investigation includes both field as well as laboratory tests. Standard Penetration Test (SPT) is one of the widely used field test for subsurface soil exploration. SPT's extensive use lies in the reason it is easy to perform and inexpensive. Certain number of developed correlations between soil properties and SPT blow counts (N value) are being used due to following reasons.

1. Unavailability of equipment
2. Financial Limitations
3. Time Limitations
4. Cross checking of the lab results

Comprehensive characterization of a site at a location is well beyond the scope of most of the project's budget. Instead, a design engineer need to rely on limited information that is when correlation becomes important.

1.2. Need for Research

Although, several researchers have developed the correlation under study however, caution must be taken when using the broad and generalized correlations. The source, extent and limitations of the correlation must be examined carefully before using it. Whenever available local calibration to be preferred over the broad and generalized correlations.

In Pakistan sufficient work is not done in this regard. In an environment with less resources (unavailability of equipment and financial limitations) this study will provide a better way to determine engineering properties of soil. This study will be a helpful tool for engineers working in the field of soil investigation.

Moreover Frazad and Behzad (2011) recommends specific relationship regarding that particular area to be used when predicting values for s_u.

1.3. Objectives of Research

Development of correlation between

1. SPT blow counts (uncorrected) and undrained shear strength
2. SPT blow counts (corrected) and undrained shear strength
3. Plasticity Index (PI) and undrained shear strength
4. SPT blow counts (uncorrected) and Soil Modulus of Elasticity
5. SPT blow counts (corrected) and Soil Modulus of Elasticity

1.4. Thesis Outline

This study is carried in the following way

Chapter 01

Provides the problem background, need for research, objectives of this research and brief outline of thesis.

Chapter 02

This chapter is literature review regarding fine grained soils, undrained shear strength, soil modulus of elasticity and standard penetration test (SPT).

Chapter 03

This chapter is about the methodology of the study.

Chapter 04

This chapter presents results and discussion.

Chapter 05

In this chapter conclusions and further recommendation are covered.

Chapter 02

2. Literature Review

2.1. Undrained Shear Strength

Deriving conditions in field or lab results in either drained or undrained shear strength (s_u). Strength of the soil is called undrained strength when failure is because of loading under undrained conditions. In the field, undrained conditions prevail when the rate of loading is such that soil cannot drain. In the laboratory, undrained conditions are achieved by two ways one is loading test specimens so rapidly that it cannot get time to drain, other way is to seal the sample in impermeable membranes. Shear induced excess pore pressure does not dissipate at the same rate.

2.2. Determination of Undrained Shear Strength

2.2.1. Laboratory Methods

2.2.1.1. Unconsolidated Undrained Test (UU Test)

In this test, drainage valves of the triaxial cell are closed when specimen is placed. So consolidation cannot occur when a confining pressure is applied. After that hundred percent saturated soil sample is sheared undrained. The sample is loaded to failure in about 10 to 20 min. This test is total stress test and it results strength in terms of total stresses.

When cell pressure with closed drainage valves is applied, a positive pore pressure is developed in the specimen, which is same as the applied cell pressure. All the increase in hydrostatic stress is carried by the pore water because of 100 % saturation of sample.

All test specimens for hundred percent saturated clays are assumed to have equal water content, as a result they will have the equal strength because there is no drainage allowed.

As far as Mohr circles are concerned all circle will have the same diameter at failure and the Mohr failure envelop will be a horizontal line.

2.2.1.2. Unconfined Compressive Strength Test (UCS)

UCS test is a special case of the U-U test where cell pressure is equal to atmospheric pressure. Unconfined compression test can be conducted to obtain the UU total stress strength. The effective stress conditions are similar for both the cases and if the effective stress conditions are the same in both tests, then strength will be same.

Below are assumptions for UCS test to yield same strength as the UU test

1. Soil sample must be 100 % saturated
2. Soil sample must be intact, homogenous clay
3. The soil must be fine grained
4. Soil sample must be sheared rapidly

2.2.2. Field Methods

The major disadvantage of field test is that they do not provide direct measurement of shear strength. Shear strength is measured indirectly from the field test by using their correlations with laboratory test. The s_u mobilized in the field may be very much different from the strength measured by laboratory tests, primarily due to the reasons; (1) Modes of shear along a slip surface in the field are different than in laboratory and field (2) s_u in the field takes much longer time as compared to shear tests (3) failure progression is more significant in the field (4) disturbance of soil is a factor only with the soil samples for tests not for the field tests.

Thus before using the s_u measured in shear tests, either it must be corrected for the above mentioned effects, or it must be calibrated against the strength mobilized in full-scale failures. The most practical and reasonable approach is to calibrate laboratory test values against s_u back-calculated from actual failures.

2.2.2.1. Standard Penetration Test (SPT)

This field test is abundantly used for determining undrained shear strength by using its empirical correlation. This is also the subject of this study. Already developed correlations are presented at the end of this chapter under previous research work.

2.2.2.2. Cone Penetration Test (CPT)

Correlations exist to estimate s_u from CPT values. The formula used is based on the bearing capacity theory proposed by Terzaghi (1943) rewritten as

$$q_c = N_k . s_u + \sigma_{vo} \qquad \text{Eq (2.1)}$$

$$s_u = (q_c - \sigma_{vo}) / N_k \qquad \text{Eq (2.2)}$$

Where

q_c is tip resistance of cone

N_k is empirical cone factor which is different for different soils

s_u is undrained shear strength

σ_{vo} is the total vertical stress

Table: 2.1 N_k for different theoretical model from Manual on estimating soil properties for foundation design (1990)

Theory	**Suggested N_k**	**Remarks**
Classical Plasticity	Order of 9	For general shear model
Cavity Expansion	7-13	Increasing value of rigidity index (Ir)*
Steady Penetration	14-18	For wide range of rigidity index

Rigidity Index (Ir) = G/s_u ; where G is shear modulus

With the various uncertainties in choosing appropriate theoretical models, N_k usually is determined empirically. Following factors can change the N_k

1. Inconsistent reference strengths
2. Mixing of different type of cones
3. Need for correction of q_c for pore water stress effects

The value of N_k ideally should be determined experimentally by comparison with a consistent reference strength. Often, the field VST is used as the reference. In this regard, it is important to recall that the VST requires a correction factor for s_u in itself. Early correlations; Battaglio et.al (1986) for N_k using uncorrected VST data suggested a trend for N_k in terms of plasticity Index (PI). However, upon re-analysis of the same data using the corrected VST strength, N_k apparently was suggested to be independent of PI. Subsequent studies by Keaveny and Mitchell (1986) and Konard and Law (1987) have demonstrated that Vesic's cavity expansion theory (1973) provides a reasonable estimate for Nk, as given below

$$N_k = 2.57 + 1.33 (\ln I_r + 1) \qquad \text{Eq (2.3)}$$

Keaveny and Mitchell (1986) suggest using triaxial compression tests to evaluate rigidity index (Ir) while Konard and Law (1987) recommend using the self-boring pressuremeter test.

Table 2.2: N_k suggested by different researcher for different soil type from Zsolt (2012)

N_k	Soil Type	Researcher
11-19	Normally consolidated Scandinavian marine clays	Lunne and Kleven (1981)
20 15	Marine Clays Boulder Clays	Jorss (1998)
15.5	Soft to stiff saturated Chicago clays	Eid and Stark (1998)

2.2.2.3. Cone Penetration Test (CPTU)

CPTU is an equipment that gives measurement of pore water pressure along with tip resistance.

$$s_u = (q_t - \sigma_{vo}) / N_{kt} \qquad \text{Eq (2.4)}$$

Where

q_t tip resistance values corrected for pore pressure effects

N_{kt} is empirical cone factor (for expression using q_t)

Table 2.3: N_{kt} value suggested by different researchers after Zsolt (2012)

N_{kt} value	**Comments**	**Researcher**
8-16	For clays (3% < I_p < 50%) N_{kt} increases with I_p	Aas et al. (1986)
11-18	No correlation was found between N_{kt} and I_p	La Rochelle et al. (1988)
8-29	N_{kt} varies with over consolidation ratio	Rad and Lunne (1988)
10-20		Powell and Quarterman (1988)
6-15	N_{kt} decreases with pore pressure ratio (B_q)*	Karlsrud (1996)
7-20	Busan clay, Korea 25% < I_p < 40%	Hong et al. (2010)
4-16	soft clay with high plasticity, 42% < I_p < 400%	Almeida et al. (2010)

Pore pressure ratio (B_q) = $u_2 - u_o / q_t - \sigma_{vo}$

Where

u_2 = Shoulder pore water pressure measured behind tip

u_o = In-situ pore water pressure

Another method to calculate s_u from CPT tip resistance is the use of effective cone resistance.

$$s_u = (q_t - u_2) / N_{ke} \qquad \text{Eq (2.5)}$$

Where

$(q_t – u_2)$ is effective cone resistance q_E

N_{ke} cone factor (for expression using q_E)

Table 2.4: N_{ke} suggested by different authors from Szolt (2012)

N_{ke} value	**Comments**	**Researcher**
6 to 12	Clays with (3% < Ip < 50%)	Senneset et al. (1982)
1 to 13	N_{ke} is dependent on B_q	Lunne et al. (1985)
2 to 10	N_{ke} decreases with B_q	Karlsrud (1996)
3 to 18	Busan clay (Korea) having 25% < Ip < 40%	Hong et al. (2010)

For very soft clays the use of excess pore water pressure may be better to find a reliable correlation because measured q_c values are relatively small, so even minor errors can influence the measured values significantly.

$$s_u = \Delta u / N_{\Delta u} = u_2 – u_o / N_{\Delta u} \qquad \text{Eq (2.6)}$$

Where

Change in pore water pressure $(\Delta u) = u_2 – u_o$

u_2 is shoulder pore water pressure measured behind the tip of CPT

u_o is In-situ pore water pressure

$N_{\Delta u}$ is the empirical cone factor for the expression using Δu

Table 2.5: values $N_{\Delta u}$ suggested from Zsolt (2012)

$N_{\Delta u}$	**Researcher**
4-10	Lunne et al.(1985)
6-8	Karlsrud et al.(1996)
4-9	Hong et al. (2010)

2.2.2.4. Pressuremeter Test (PMT)

The pressuremeter test (PMT) ideally provides a measurement of S_u at the PMT limit stress. Based on cavity expansion theory Baguelin et al. (1978) Su can be evaluated from:

$$s_u = (p_L - p_o)/N_p \qquad \text{Eq (2.7)}$$

Where

p_L = PMT limit stress; pressure required to double the cavity

p_o = PMT total horizontal stress; approximate start of cavity expansion

Np = 1 + ln ($E_{PMT}/3s_u$) and E_{PMT} = PMT modulus

Mair and Wood (1987) have suggested values of Np ranging from 2 to 20, but typical values usually range from 5 to 12, with an average of 8.5. Difficulties in choosing the correct value of Np are compounded by possible measurement errors in both P_L and P_o.

Undrained shear strength determined by using different types of pressuremeters is different, Wroth (1984). When self-boring pressuremeter (SBPM) is used post peak strength is determined from the latter part of the loading curve considering soil perfectly plastic, Windle and Wroth (1977).

$$p = p_L + s_u \ln\{\Delta V/V - (1 - \Delta V/V)\, \sigma_h/G\} \qquad \text{Eq (2.8)}$$

Where

p = Pressure applied on membrane

p_L = Limiting pressure

σ_h = Total in-situ lateral stress also referred as p_o

ΔV = Volume change in membrane

V = Volume of membrane at applied pressure p

Table 2.6: Empirical correlation between undrained shear strength and net limit pressure (After Clarke, 1995)

<table>
<tr><th>s_u</th><th>Soil Type</th><th>Reference</th></tr>
<tr><td>$(p_L - \sigma_h)/k$</td><td>Where k = 2 to 5</td><td>Menard (1957)</td></tr>
<tr><td>$(p_L - \sigma_h)/5.5$</td><td>Soft to firm clays</td><td rowspan="3">Cassan (1972), Amar and Jezequel (1972)</td></tr>
<tr><td>$(p_L - \sigma_h)/8$</td><td>Firm to stiff Clays</td></tr>
<tr><td>$(p_L - \sigma_h)/15$</td><td>For stiff to very stiff Clays</td></tr>
<tr><td>$(p_L - \sigma_h)/5.1$</td><td>All Clays</td><td>Lukas and LeClerc de Bussy (1972)</td></tr>
<tr><td>$(p_L - \sigma_h)/10+25$</td><td></td><td>Amar and Jezequel (1972)</td></tr>
<tr><td>$(p_L - \sigma_h)/6.8$</td><td>Stiff Clays</td><td>Marsland and Randolph (1977)</td></tr>
</table>

$(p_L - \sigma_h)/10$	Stiff Clays	Martin and Drahos 1986
$p_L/10+25$	Soft to Stiff Clays	Johnson (1986)

For full displacement pressuremeter (FDPM) test Houlsby et al. (1988) mentioned technique to determine s_u from the unloading part of the FDPM curve

$$p = p_L - 2s_u\,[1+\ln\{\sinh(\varepsilon_{max} - \varepsilon)/\sinh(s_u/G)\}] \qquad \text{Eq (2.9)}$$

Where

ε_{max} = Maximum strain reached in the test

ε = Cavity strain at any pressure

2.2.2.5. Dilatometer Test (DMT)

In dilatometer test (DMT) horizontal stress index has been correlated with s_u by Marchetti (1980)

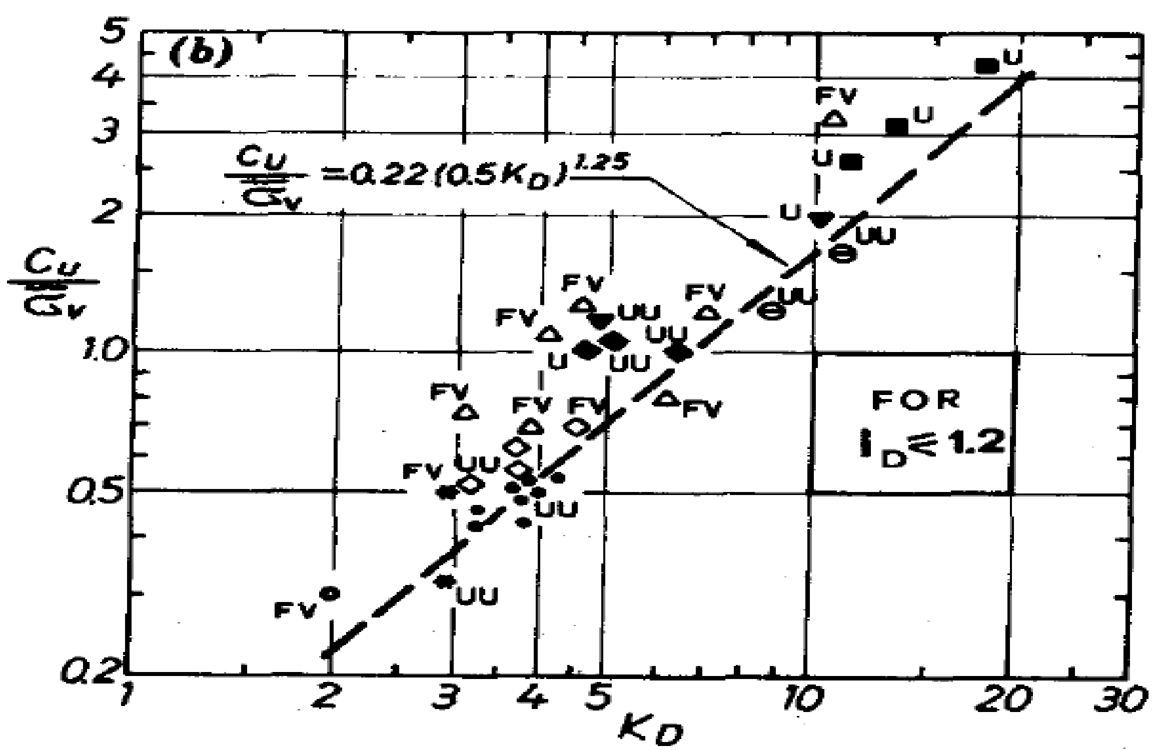

Fig 2.1 correlation between s_u/σ'_{vo} and K_D Marchetti (1980)

$$(s_u / \sigma'_v)_{DMT} = 0.22\ (0.5\ K_D)^{1.25} \qquad \text{Eq (2.10)}$$

$K_D = p_o - u_o / \sigma'_{v;}$ Horizontal stress index

$I_D = p_1 - p_o / p_o - u_{o;}$ Material index

p_o = corrected A-pressure when the membrane move 0.05 mm

p_1 = corrected B-pressure when membrane expand 1.1 mm

u_o = in-situ pore water pressure

σ'_v = effective vertical stress

This equation originally was based on clays with a material index, I_D, less than or equal to 1.2. Schmertmann (1988) limited this relationship to clays with $I_D \leq 0.6$. The strength data initially was taken from unconfined compression tests (UCS), UU triaxial compression tests UU and field vane shear tests (VST). Subsequent work by Lacasse and Lunne (1988) suggests that the 0.22 coefficient in equation should vary with test type as follows: 0.14 for direct simpler shear, 0.20 for triaxial compression, and 0.17 to 0.21 for field VST. Other data by Powell and Uglow (1986) indicate different factors for fissured clays and glacial tills if the reference s_u is determined from plate load tests or the self-boring pressuremeter test.

2.2.2.6. Vane Shear Test (VST)

Vane shear is one of the oldest in-situ tests for the evaluation of the mobilized undrained shear strength in clays. The value of s_u is determined from the torque required to rotate a four bladed vane in clay. Both the peak and remolded s_u can be determined, and therefore the sensitivity (S_t) of the clay can be computed.

$$s_{uo}\,(FV) = 6T/7\pi D^3 \qquad \text{Eq (2.11)}$$

It should be avoided to use directly the value of s_u calculated from the VST because it needs to be corrected for the strain rate during testing and the soil anisotropy. Bjerrum (1972, 1973) analyzed footing, embankment and excavation failures in terms of in situ vane undrained shear strength. He plotted a graph of factors of safety with the plasticity indices of soils. His best fit straight line through the data points is shown in Fig 2.2.

Using this line Bjerrum suggested the in-situ vane correction factor μ= 1/FS need to be applied to s_u (VST).

$$s_{uo}\,(mob) = \mu s_{uo}\,(FV) \qquad \text{Eq (2.12)}$$

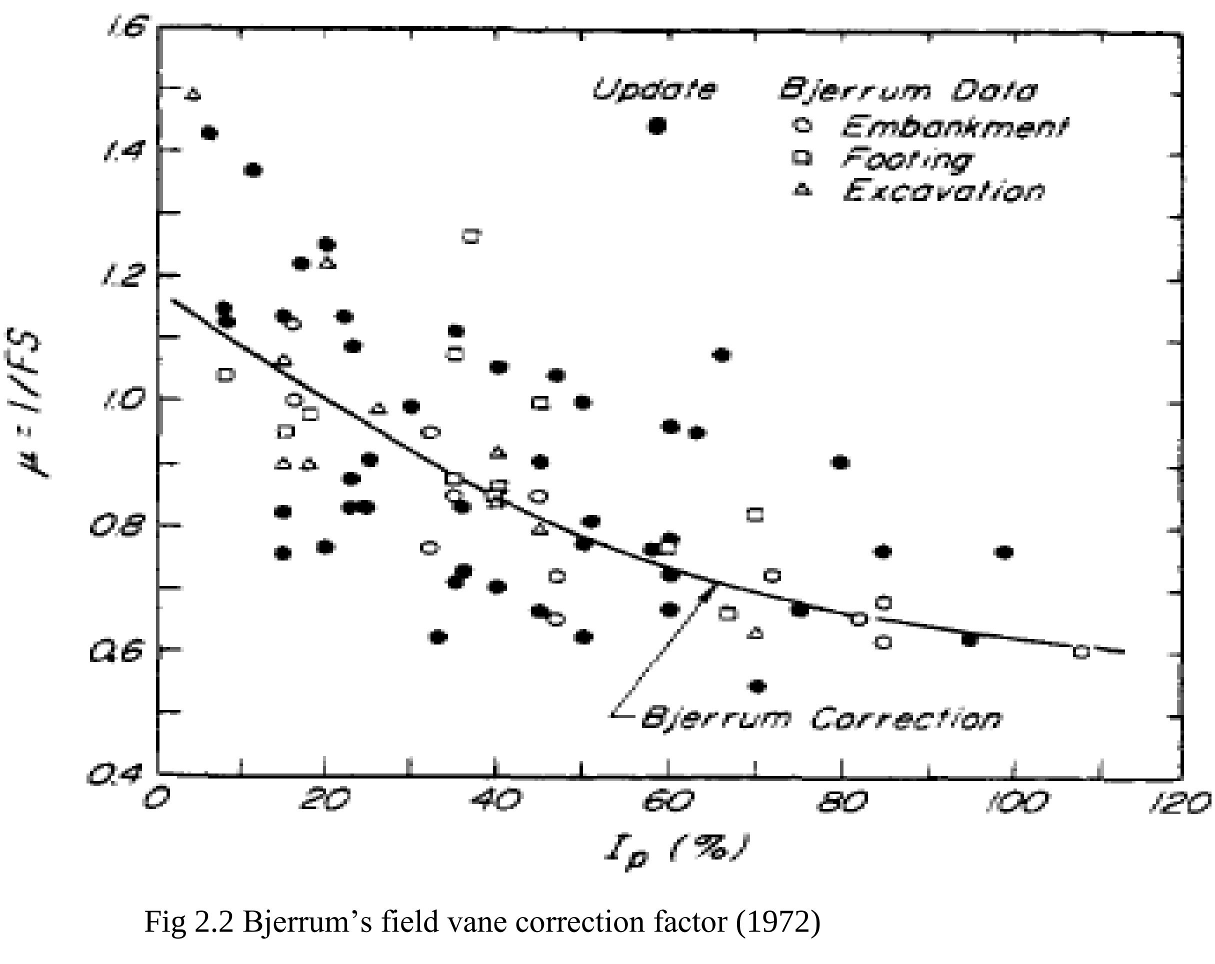

Fig 2.2 Bjerrum's field vane correction factor (1972)

A comprehensive compilation of s_{uo} (FV)/σ'_p data, by Tavenas and Leroueil (1987) is shown in Fig 2.3 together with the relationship based on Bjerrum's curves. Equipment and procedure other than the standard for the measurements of vane strength, pre-consolidation pressure, and plasticity index partly effect the measured values.

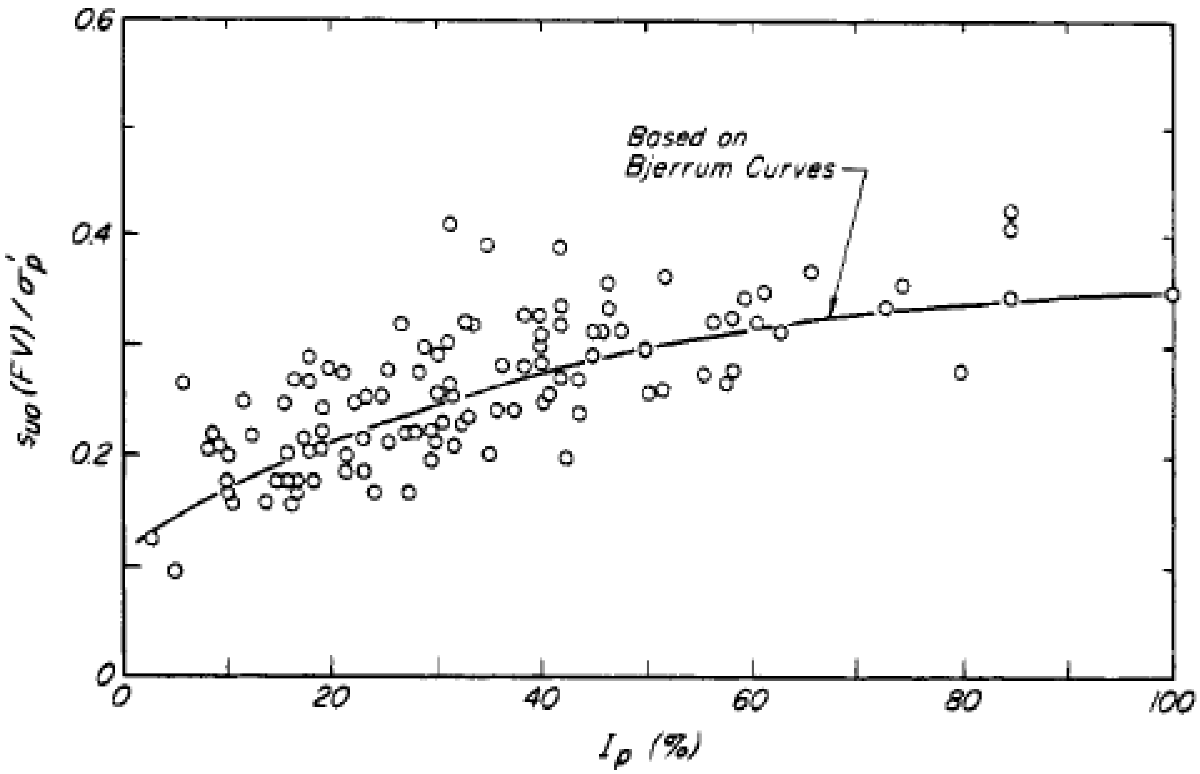

Fig 2.3 Tavenas and Leroueil (1987)

2.3. Soil Modulus of Elasticity

2.3.1. General

Elastic soil parameters Young's modulus Es, shear modulus Gs and Poisson's ratio are used to evaluate soil elastic distortion or immediate settlement. Truly elastic materials obey Hooke's law in which increment of applied uniaxial stress causes a proportionate increase in strain in other words stress is proportional to strain.

$$\varepsilon_z = {}^{\sigma_z}/_{E} \qquad \text{Eq (2.13)}$$

Where E is soil modulus of elasticity. Hooke's law originally developed from elastic behavior of metal bars in tension is also applicable to homogeneous and isotropic materials. Soil under relatively small loads is sometimes assumed to behave linearly elastic. During partially elastic material obeys Hooke's law, but this material on removal of applied stress will not gain its initial shape and dimensions. These materials are nonlinear and most of soils are included in this category.

Cohesive soils exhibit time-dependent response to loading. For initial quick loading conditions, the response is undrained. With time, the excess pore water stresses developed during undrained loading will dissipate, leading to consolidation. For undrained loading, the modulus of cohesive soils can be described by either the undrained Young's modulus E_u.

2.3.2. Factors Affecting Soil Modulus

State Factors

Packing of the particles: when particles are closely packed then modulus comes out to be high. Particles organization: This refer the structure of the soil for coarse grained soil it maybe loose or dense for fine grained soil it maybe flocculated or dispersed. Water content: the effect of this parameter is high on modulus. For fine grained soils low water content modulus tend to be high and for coarse grained soil reverse is true. Stress history: referring to the stress history the soil may be over consolidated and normally consolidated. Over consolidated soils will generally have more moduli than the same normally consolidated soil.

Loading Factor

Level of stress in the soil: at one point or at any given time there are three principal stresses in soil the mean of these stresses have impact on the modulus. This is also called confinement effect. Konder (1963) developed a model to study the effect of confinement. His model suggests that modulus is proportional to power law of confinement. Strain level in the soil: secant modulus depends upon mean strain level in the zone of influence. Strain rate in the soil: the faster the soil is loaded stiffer the soil will be so higher is modulus. Number of cycles the soil has experienced: the more the number of cycles the smaller the modulus will be. Drainage conditions: depending upon the conditions of drainage the modulus may be drained or undrained modulus.

2.3.3. Empirical Methods of Determining Modulus of fine grained soils

Apparently few studies have been carried to correlate modulus with SPT, CPT and PMT.

The pressuremeter test PMT provides a measurement of the horizontal modulus in soils. In clays it is assumed $E_{PMT} = E_S$. Attempts have been made to correlate E_{PMT} with SPT N values.

The elastic undrained modulus Es for clay may be estimated from the undrained shear strength (USACE EM 1110-1-1904).

$$E_s = K_c C_u \quad \text{Eq (2.14)}$$

E_s = Young’s soil modulus in tsf

K_c = correlation factor

$C_u = s_u$ in tsf

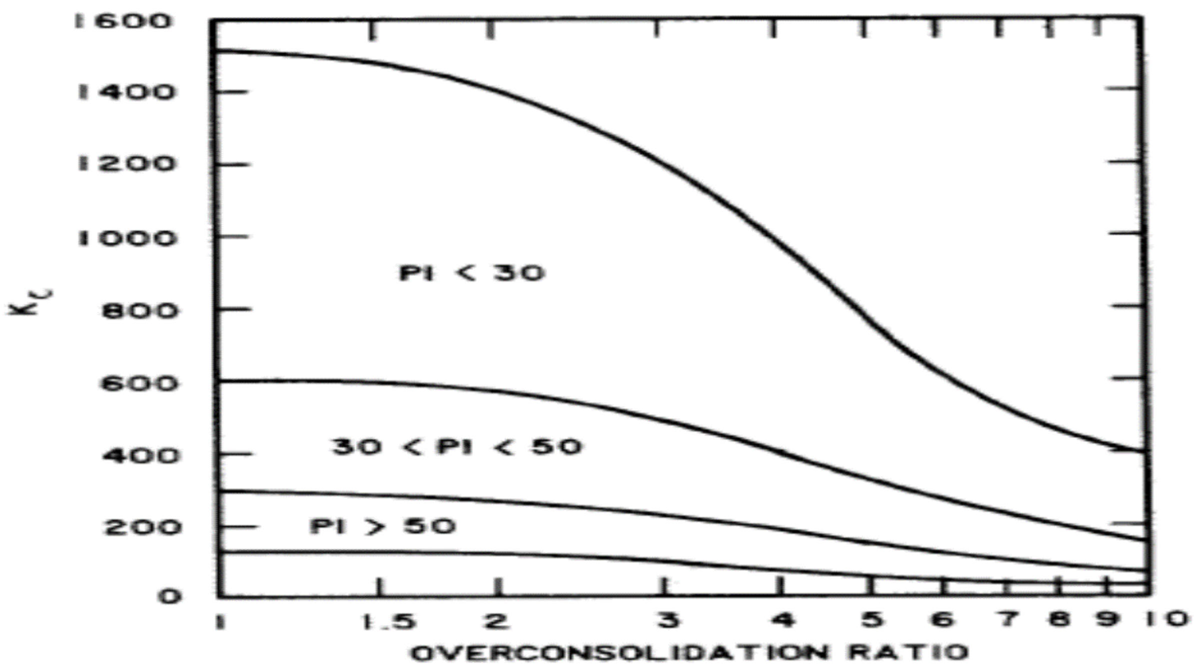

Fig 2.4 chart for determining value of Kc (USACE EM 1110-1-1904)

2.4. Fine Grained Soils

2.4.1. General

Different soils with similar properties may be classified into different groups and sub-groups according to their engineering behavior. Classification systems give general characteristics of soils without detailed descriptions. Based on particle-size distribution and Atterberg limits following two classification systems are commonly used by engineers.

1. American Association of State Highway and Transportation Officials (AASHTO) classification system
2. Unified Soil Classification System (USCS)

Both systems categorize the soils into two broader groups, coarse grained and fine grained. In USCS, soil falls within one of three major categories coarse-grained soils, fine-grained soils and highly organic soils. Coarse grained soil if passing No.200 sieve is less than 50%

and fine grained soil if passing No. 200 is more than or equal to 50%. AASHTO system classify a soil as fine grained when more than 35% passes through the No. 200 sieve otherwise coarse grained. AASHTO system assign a group classification and group index. The group classification ranges from A-1 to A-8 and Group index value from 0 to 20.

2.4.2. USCS Classification

This classification is covered in ASTM D 2487. Only the flow charts for classification of fine grained soil from the standard is included.

1. Flow chart for classification of fine grained soil in Fig 2.5
2. Flow chart for classification of organic fine grained soil in Fig 2.6

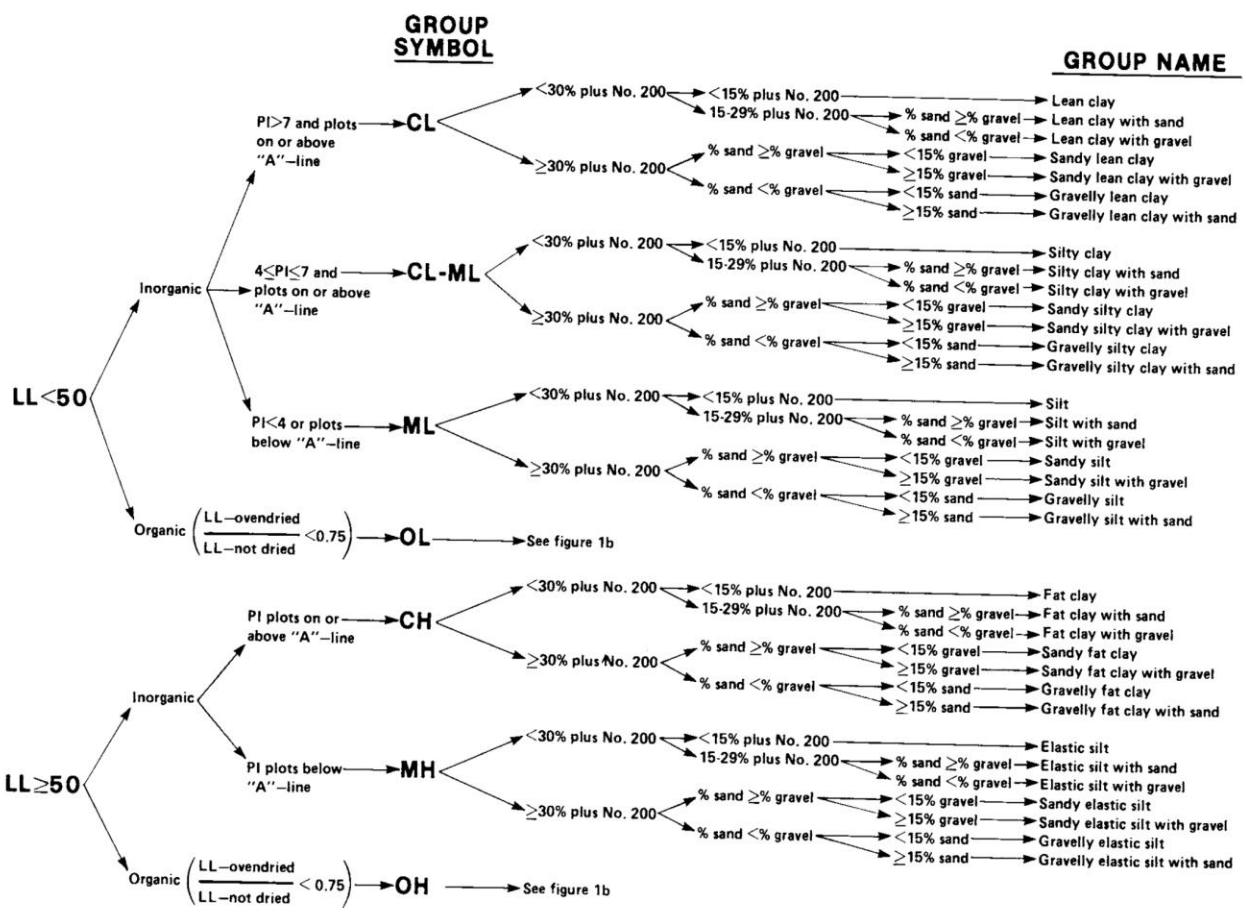

Fig 2.5 Flow chart for classification of fine grained soil (ASTM D2487)

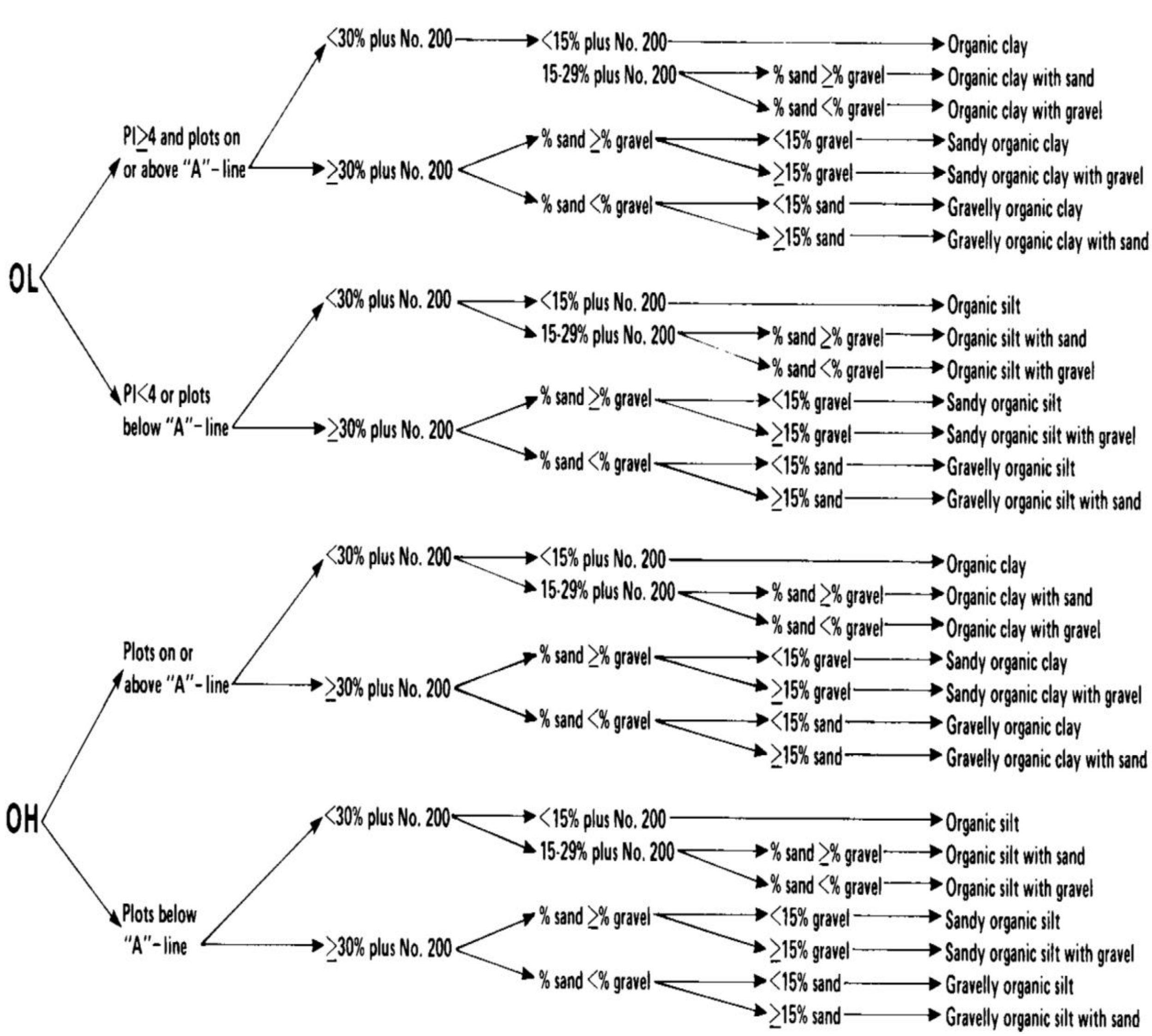

Fig 2.6 Flow chart for classification of organic fine grained soil (ASTM D2487)

Plasticity chart for classification of fine grained soil extracted from the same standard is shown in Fig 2.7

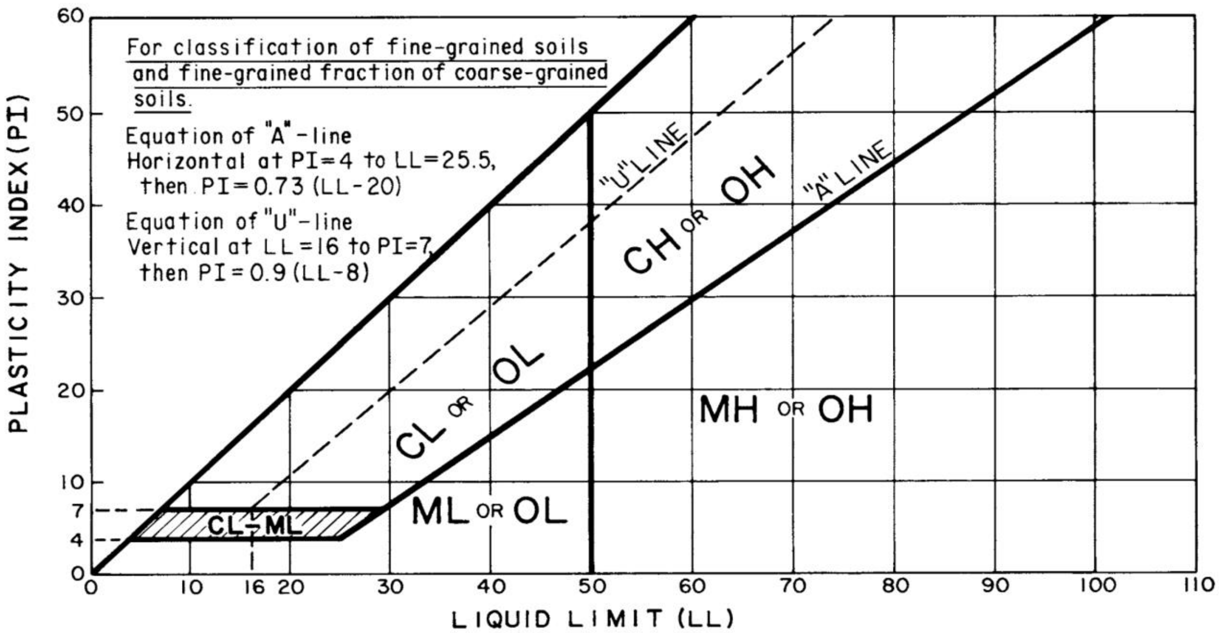

Fig 2.7 Plasticity chart (ASTM D 2487)

2.4.3. AASHTO Classification

This classification system is covered in ASTM D 3282. Classification chart obtained from the standard for fine grained is included and shown in Fig 2.8

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	A-4	A-5	A-7 A-7-5[a] A-6	A-7-6[b]
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a]For A-7-5, $PI \leq LL - 30$
[b]For A-7-6, $PI > LL - 30$

Fig 2.8 Classification chart (ASTM D 3282)

2.5. Standard Penetration Test (SPT)

2.5.1. General

Despite the variability of the test, SPT is the most common in-situ test used for foundation design and other geotechnical applications, Kovacs and Salomone (1982). Many researchers report that the SPT is not standard as the name implies. However, because the test is relatively simple to conduct, produces an indicator of soil resistance (N-Value), and gives a soil sample for visual identification and laboratory testing, the test maintain its popularity with geotechnical engineers worldwide. A number of correlations relate the N-value to parameters as relative density, friction angle, allowable bearing capacity, and pile skin friction. Because of the frequent use of such correlations, it is important for the performance of the SPT, it allows for a variety of equipment to be used. As a result there are a variety of hammer types in use, ranging from donut and safety hammers using a cathead and rope system to the more recently developed automatic trip hammers. These factors, as well as numerous others, result in different SPT systems introducing different amounts of energy per blow into drill rods. As a result, for SPTs performed in identical soil conditions, different driller –rig hammer systems will likely record different N-Values.

Most of the correlations developed from the results of SPT originated before use of the automatic hammer and are based on the cathead and safety hammer system. These safety hammer systems are generally found to have efficiencies of about 60 % of the theoretically available energy is transferred from the hammer to drill rods. The N-values from systems operating at 60% efficiency are referred to as N_{60}. If an SPT hammer-rig driller system is not operating at 60 % efficiency, then the correlations will not hold. The newer automatic hammers, for instance, typically have much higher efficiencies, N-values taken from these hammers may not be appropriate for use with N_{60} correlations.

2.5.2. Brief History

In March 1988, the First International Symposium on Penetration (ISOP-I) Testing held in Orlando Florida reports, the origin of the test and development of the test method, that initially Lt. Col Charles R. Gow in 1922 drove one-inch diameter pipes into the ground for soil sampling. Riggs (1986) as well as Broms and Flodin (1988) reports that the actual work by Charles Gow dates as far back as 1902 and that the one inch pipe served both as sampler and the sampling rod column. Riggs states that Gow used this method to estimate pile-driving resistance.

In 1927, the two inch outside diameter sampling spoon was developed based on fieldwork by G.F.A. Fletcher and Mohr. Documentation of early development of the SPT is contained in a paper by Mohr (1937) and a paper by Fletcher (1965). According to Fletcher, "the original purpose was to measure the density of soil formation by a standard procedure, thereby providing a correlation with experience in the design and installation of caisson foundations."

However, (ISOP-I) documents that Terzaghi, in 1947 paper presented to the 7^{th} Texas conference on Soil Mechanics and Foundation Engineering, was the first to call the method the "Standard Penetration Test." The first textbook reference to the SPT occurred in the 1948 edition of Terzaghi and Peck Soil Mechanics in Engineering Practice.

2.5.3. Apparatus

1. Drilling Equipment

For drilling equipment to be acceptable two basic requirements are that it provides a suitable borehole before inserting the sampler and ensure that the test is performed on undisturbed soil. The following equipment is considered necessary to be a part of equipment to ensure the above mentioned condition for most of the subsurface conditions

   a) Drag, Chopping, and Fishtail Bits
   b) Roller-Cone Bits
   c) Hollow-Stem Continuous Flight Augers
   d) Solid, Continuous Flight, Bucket and Hand Augers

2. Sampling Rods

For connecting split-barrel sampler to the assembly flush-joint steel drill rods shall be used. The stiffness of the sampling rod shall be equal to or more than that of parallel wall. Normally a steel rod with an outer diameter of 41.3 mm and an inner diameter of 28.5 mm is considered suitable.

3. Split Barrel Sampler

The sampler has an outside diameter of 50.8 mm and the inside diameter can be either 38.1 mm or 34.9 mm. A 16-gauge liner can be used inside 38.1 mm split barrel sampler. The driving shoe shall be of hardened steel. Driving shoe whenever becomes dented after using shall be repaired. The end of the drive shoe may be slightly rounded.

2.5.4. Drive Weight Assembly

The hammer shall be rigid metallic mass with a weight of 63.5 kg. The hammer that strike the anvil should make perfect contact when it is dropped. To have perfect striking hammer fall guide shall be used. Hammers used with the cathead and rope method shall have an unimpeded over lift capacity of at least 100 mm. To ensure safety the use of a hammer assembly with an internal anvil is recommended. The total mass of the hammer assembly on the drill rods should not be more than 113.65 kg.

To enable the operator or inspector to judge the hammer drop height Hammer fall guide be permanently marked.

2.5.5. Testing Procedure

SPT is covered in ASTM D 1586. However, steps involved as below

1) Bore hole drilling to the desired depth
2) After drilling the borehole drilling equipment is removed for lowering the sampler to the bottom of hole
3) Sampler is penetrated into the soil by using drop hammer weighing 63.5kg with fall height of 750 mm
4) Sampler is penetrated by 450 mm and the number of hammer blows (N) required to drive each 150 mm are recorded
5) The standard penetration number (N) is obtained by neglecting the number of blows (N) for the first 150 mm whereas the number of blows recorded for last two 150 mm penetration are summed.

2.5.6. SPT Correction Factors

There are many factors other than hammer type mentioned in ASTM D 1586-99 that affect the N value. Different authors have proposed different correction factor for rod length, the type of anvil, the blow rate, the use of liners or borehole fluid and the type of hammer. Correction factors suggested by different authors are shown below

Table 2.7: Correction factors for rod length

Rod Length	McGregor and Duncan 1998	Bowles 1996	Skempton 1986	Robertson and Wride 1997
Length over 30 m	1	1	1	> 1
10-30 m	1	1	1	1
6-10 m	1	0.95	0.95	0.95
4-6 m	1	0.85	0.85	0.85
3-4 m	1	0.75	0.75	0.75
0-3 m	0.75	0.75	0.75	-

Table 2.8: correction factors for Blow Rate

Correction for Blow Rate	McGregor and Duncan 1998	
	Bdf	Frequency of Blows
< 20	0.95	10-20 blows/min
> 20	1.05	10-20 blows/min

Table 2.9: Correction Factor for Anvil

Anvil	Skempton 1986	Mcgregor and Duncan 1998
Small	0.7-0.8	0.85
Large	0.6-0.7	0.7
Safety	0.7-0.8	0.9

Table 2.10: Correction factor for Borehole dia

Borehole diameter (mm)	Bowles 1996	Robertson and Wride 1997	Skempton
60-120	1	1	1
150	1.05	1.05	1.05
200	1.15	1.15	1.15

Table 2.11: Correction factor for sampler liner

Sampler	Bowles 1996	Robertson and Wride 1997	Skempton
without Linear	1	1.1 - 1.3	1.2
With Linear loose sand	0.9	1	1
With liner dense sand	0.8	1	1

Table 2.12: Correction factor for hammer type

Hammer Type	McGregor and Duncan 1998	Robertson and Wride 1997
Automatic	1.67	0.8 - 1.5
Pulley Safety	1	0. 7 - 1.2
Donut	0.75	0.5 - 1.0

2.6. SPT Applicability in Fine Grained Soils

Even though the SPT was originally developed for coarse grained Soils, it has been applied to fine grained soils to estimate the engineering properties. However, its applicability for fine grained soils is still not confirmed Brooms (1986) and Decourt (1990) as referred in Sivrikaya and Togrol (2006). L.Behpoor and A.Ghahramani (1989) state that pessimistic view of applicability of SPT to cohesive soils is not warranted.

2.7. Previous Research Work

2.7.1. General

First ever proposed correlation of N-value with s_u is by Terzaghi and Peck (1967). Then Sangleart (1972) suggested correlation equations of SPT with s_u for clay and silty clay. Stroud developed the different equations against different plasticity index values. Similarly many other researchers worked in this regard and suggested their own equations. The earlier research work does not describe that either the N-value being used is corrected or not. However, Decourt (1990) and Sivrikaya and Togrol (2002) have developed different equations for corrected and uncorrected N-value. Sivrikaya and Togrol (2009) suggested equations based on s_u values from different test. Frazad and Behzad (2011) developed equations for fine grained soil less than or equal to 20. They considered moisture content, liquid limit and plasticity index as independent variables along with N-value. Present work considers only N-value as independent variable for the sake of comparison with the previous work especially with Terzaghi and Peck (1967). Previous research work is given in table 2.13 and 2.14.

2.7.2. SPT N-value with undrained shear strength

Table 2.13: Previous research work with only N-value as independent

Author	**Type of soil**	**s_u (kpa)**
Terzaghi and Peck (1967)	Fine grained soil	6.25 N
Sangleart (1972)	Clay Silty Clay	12.5 N 10 N
Hara et al (1974)	Fine grained Soil	$29\ N^{0.72}$
Stroud (1974)	PI < 20 20 < PI < 30 PI > 30	6-7 N 4-5 N 4.2 N
Sower (1979)	High Plastic Soil Medium Plastic Clay Low Plastic Soil	12.5 N 7.5 N 3.75 N
Nixon (1982)	Clay	12 N
Ajayi and Balogun (1988)	Fine-Grained Soil	1.39N + 74.2
Decourt (1990)	Clay	12.5 N $15\ N_{60}$
Sivrikaya and Togrol (2002)	Highly Plastic Clay Low Plastic Clay Fine-Grained Soil	$4.85\ N_{field}$ $6.82\ N_{60}$ $3.35\ N_{field}$ $4.93\ N_{60}$ $4.32\ N_{field}$ $6.18\ N_{60}$
Hettiarachchi and Brown (2009)	Fine-Grained Soil	$4.1\ N_{60}$

Table 2.14: Previous research work with more than one independent variable

Sivrikaya (2009)	
UU Test	s_u= 3.33N - .75w_n + 0.20LL + 1.67PI
UU Test	s_u= 4.43N_{60} – 1.29w_n + 1.06LL + 1.02PI
UCS Test	s_u= 2.41N - 0.82w_n + 0.14LL + 1.44PI
UCS Test	s_u= 3.24N_{60} - .53w_n - 0.43LL + 2.1PI
Frazad and Behzad (2011)	
Fine-Grained Soil with PI ≤ 20	s_u = 1.6 N_{field} + 15.4, (r=0.72) s_u = 2.1 N_{60} + 17.6, (r=0.73) s_u= 1.5N_{field} - 0.1w_n - 0.99LL + 2.4PI + 21.1, (r=.8) s_u= 2N_{60} - .4w_n – 1.1LL + 2.4PI + 33.3, (r=.81)

2.7.3. Plasticity Index and Undrained Shear Strength

Stroud (1974) suggested that s_u decreases with increasing plasticity index in a constant N value. Sower (1979) stated that s_u increases with increasing plasticity index. Sivrikaya and Togrol (2006) it is very difficult to find any relation and make any comment between undrained shear strength and plasticity index. However, it seems as if undrained shear strength decreases with the increase in plasticity index.

2.7.4. SPT- Soil Modulus of Elasticity

In-situ tests SPT and cone penetration test (CPT) are used in empirical correlations to obtain Es. Other in situ tests such as the pressuremeter and flat dilatometer gives more direct measurements of Es. The value of Es obtained from these tests is generally the horizontal value. Because the laboratory values of Es are expensive to obtain so standard penetration test (SPT) and cone penetration test (CPT) have been widely used to obtain the Es resulting from empirical equations and correlations. Most of these correlations are for cohesionless soils. L.Behpoor and A. Ghahraman (1989) suggested Eq (2.15) for clayey and silty clay soils with N-value less than 25.

$$E\ (\mathrm{MPa}) = 0.17\ N \qquad \text{Eq (2.15)}$$

Chapter 03

3. Research Methodology

3.1. General

O. Sivrikaya and E. Togrol (2006) mentions the following four uncertainties for the correlations in use.

a) SPT values for the developed correlations is either corrected or uncorrected. If corrected, then which corrections are applied

b) Statistics of the

c) Which test results are used

d) For which type of soil correlation is valid

These uncertainties have considerable effect on correlation equation. Above mentioned uncertainties are addressed and well defined for the purpose of this study herein this chapter.

3.2. Data Collection

To determine s_u of soil undisturbed samples were required. Retrieving of undisturbed sample is costly and time consuming to obtain sufficient number of data pairs Soil and Site investigation reports with required data were collected form Pacific Engineering, Geo Deep Rock and Geotech Engineering Services, AJK enterprises, Geo Project consultants, Advance soil Lab, Royal Material Testing Lab.

3.3. Location

Data of different projects located in Rawalpindi and Islamabad was used in this study. Collected data is of projects Hassanabdal-Havelian Section of E35 Package-1, Hassanabdal-Havelian Section of E35 Package-2, Model Prison Sector H16, Al-Huda International School Sector H-11/4, Zarkon Heights G15, Development of NUST H-12 and Construction of C-Type Flats at E-10. Map showing the location of above mentioned projects is shown below.

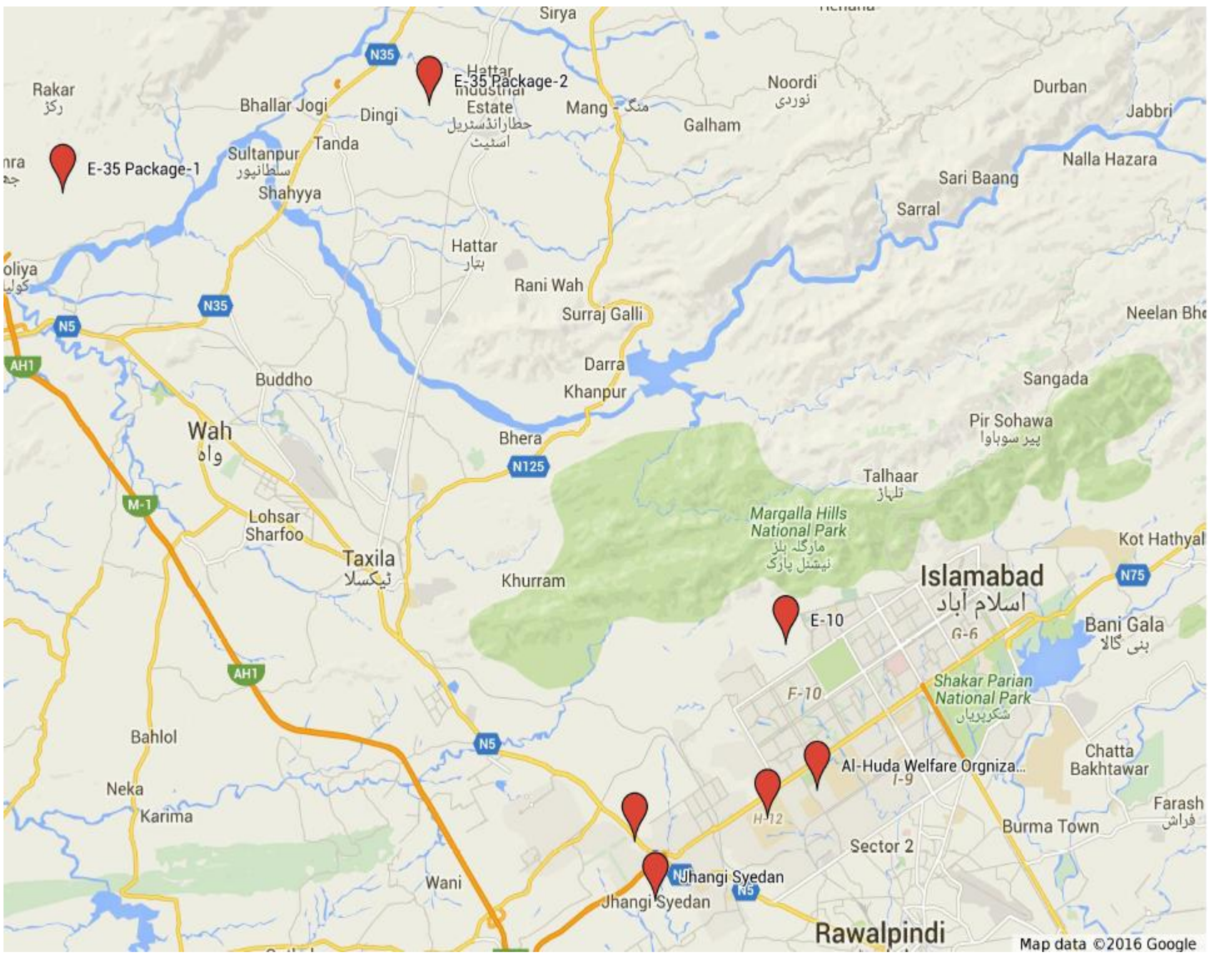

Fig 3.1 Map showing the site locations for data collection

3.4. Data Acquired

The scope of the study is limited to fine grained soil only. Following parameters for the fine grained soil were acquired from the reports.

1. Undrained Shear strength

   Undrained shear strength, a parameter for correlation between SPT N value and undrained shear strength, was obtained from test results of unconfined compression strength test contained in collected investigation reports. The value of undrained shear strength reported in different units of measure is reported in kpa for this study. With highest value 169.97 kpa, lowest value 12.5 kpa and average value 65.49 kpa.

2. Undrained shear strength/UCS Test graph

   UCS test graph to determine soil modulus of elasticity. Soil modulus of elasticity is calculated as

   a) slope of the line joining point of zero stress strain with peak point on the UCS graph
   b) slope of the line joining point of zero stress strain with 50% of peak strength on the UCS graph
   c) slope of the line joining point of zero stress strain and point of 1% strain on the UCS graph

3. Plasticity Index of Soil

   Plasticity Index (PI) of soil was taken to develop a correlation between PI and undrained shear strength. PI obtained has highest value 20, lowest value 3 and average value 8.77

   Sowers, 1979 (as in Coduto, 1999 Geotechnical Engineering Principals and Practices) provides the following limits.

Table 3.1: soil classification based on plasticity

Classification	**Plasticity Index (PI)**
Non plastic	0 to 3
Slight Plastic	3 to 15
Medium-Plastic	15 to 30
Highly-Plastic	>30

3.5. SPT Blow Counts/ N value

SPT blow counts were taken from bore logs in reports. N-value reported almost at the same depth from which undisturbed sample for UCS test was extracted was considered. Both uncorrected and corrected N-values were needed. N-values were corrected using correction factors for different working standards.

McGregor and Duncan (1998) reports SPT-N correction equation as below

$$N_{60} = (C_B C_C C_R C_{BF} C_S C_A C_E)\, N_{field} \qquad \text{Eq (3.1)}$$

$N_{1,60} = C_N\, N_{60}$

C_B = Borehole diameter

C_C = Hammer cushion

C_R = Rod Length

C_{BF} = Blow count frequency

C_S = Liner

C_A = anvil

C_E = Energy

All above mentioned corrections are made in coarse grained soils. However, O. Sivrikaya and Togrol (2006) as per (Saran, 1996; McGregor and Duncan, 1998) suggests not to apply blow count frequency and overburden corrections in fine-grained soils. The following general equation for SPT correction in fine grained soil is recommend by O. Sivrikaya and Togrol (2006)

$$N_{60} = (C_B C_C C_R C_S C_A C_E)\, N_{field} \qquad \text{Eq (3.2)}$$

After consulting the firms from which data was collected it is assumed $C_B = 1.05$ (Skempton, 1986) for 150 mm diameter normally in practice herein Pakistan, $C_C = 1$ due to not using hammer cushion (Decourt, 1990), $C_R = 0.85$ for rod length 5 m being used, $C_S = 1.2$ for not using any liner (Skempton, 1986), $C_A = .85$ for small anvil being used (Tokimatsu, 1988) and $C_E = .75$ for Donut type hammer (Seed, 1984).

3.6. Statistical Analysis

In many situations the objective of study is to check whether the two variables are related instead of using one to predict the value of other. Correlation analysis is used to measure the strength of relationship between two variables. Correlation coefficient r is a measure how strongly related two variables x and y are in a sample.

Regression analysis is used to predict the value of a dependent variable basing on the value of independent variable and it tells the impact of change in independent variable to dependent variable.

For the purpose of this study linear regression analysis with one independent variable is performed by using Microsoft Excel 2013. Regression statistics of the model are taken from the analysis.

Chapter 04

4. Results and Discussion

4.1. General

Evaluation of regression model is important to find out how good the model is. To check the significance of model values of R^2, significance of F, t stat and p-value are considered. If these values are in provided limits then model is appropriate.

4.2. SPT-UCS Correlation

For this correlation selected variables are SPT N value as independent variable and undrained Shear Strength as dependent variable.. Keeping in view R^2, significance of F, t stat and p-value the model is statistically appropriate and the selected variables are statistically significant.

Table 4.1: Regression statistics of SPT-UCS

Type of soil	**R^2**	**Significance F**	**t stat**	**p-value**	**Status**
Fine grained	0.85	6.08E-65	29.34	6.08E-65	Ok
CL	0.71	1.4E-28	2.5	0.013	Ok
CL-ML	0.87	5.9E-18	2.95	0.0055	Ok

Regression Statistics						
Multiple R	0.92					
R Square	0.85					
Adjusted R Square	0.85					
Standard Error	15.60					
Observations	156.00					
ANOVA						
	df	*SS*	*MS*	*F*	*Significance F*	
Regression	1.00	209397.07	209397.07	860.99	6.08366E-65	
Residual	154.00	37453.46	243.20			
Total	155.00	246850.53				
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	9.71	2.27	4.27	3.42E-05	5.22	14.20
X Variable 1	3.90	0.13	29.34	6.08E-65	3.63	4.16

Fig 4.1 Regression Statistics excel output of SPT (uncorrected)-s_u model for fine grained soils

Regression Statistics						
Multiple R	0.84					
R Square	0.71					
Adjusted R Square	0.70					
Standard Error	13.92					
Observations	103.00					
ANOVA						
	df	*SS*	*MS*	*F*	*Significance F*	
Regression	1.00	46959.30	46959.30	242.42	1.35387E-28	
Residual	101.00	19564.69	193.71			
Total	102.00	66523.99				
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	7.50	2.97	2.53	0.01	1.61	13.38
X Variable 1	3.91	0.25	15.57	1.3539E-28	3.41	4.40

Fig.4.2 Regression Statistics excel output of SPT (uncorrected)-s_u model for CL soils

Regression Statistics						
Multiple R	0.93					
R Square	0.87					
Adjusted R Square	0.87					
Standard Error	15.80					
Observations	39.00					
ANOVA						
	df	*SS*	*MS*	*F*	*Significance F*	
Regression	1	61679.93	61679.93	247.00	5.87456E-18	
Residual	37	9239.60	249.72			
Total	38	70919.52				
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	15.62	5.30	2.95	0.01	4.88	26.37
X Variable 1	3.85	0.24	15.72	5.875E-18	3.35	4.35

Fig.4.3 Regression Statistics excel output of SPT (uncorrected)-s_u model for CL-ML soils

4.2.1. SPT (uncorrected)-UCS Correlation

Correlation graphs for fine grained, CL and CL-ML type soils are shown in Fig.4.4, Fig.4.5, and Fig.4.6 respectively. SPT values used are uncorrected.

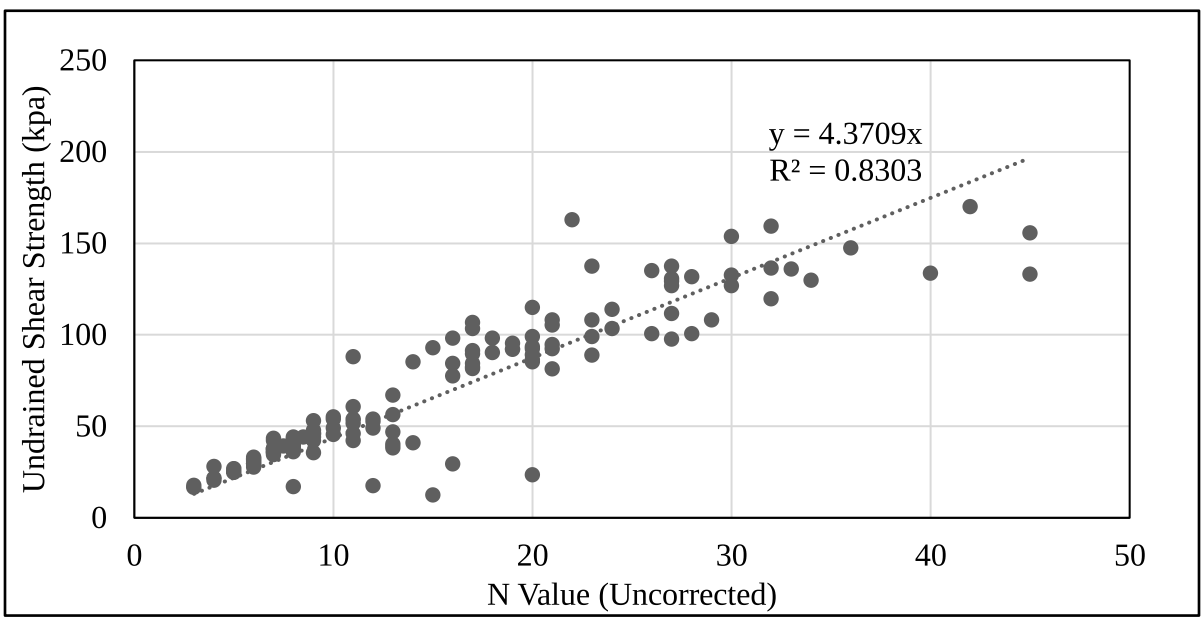

Fig 4.4 Correlation graph of SPT (uncorrected)-s_u for fine grained soils

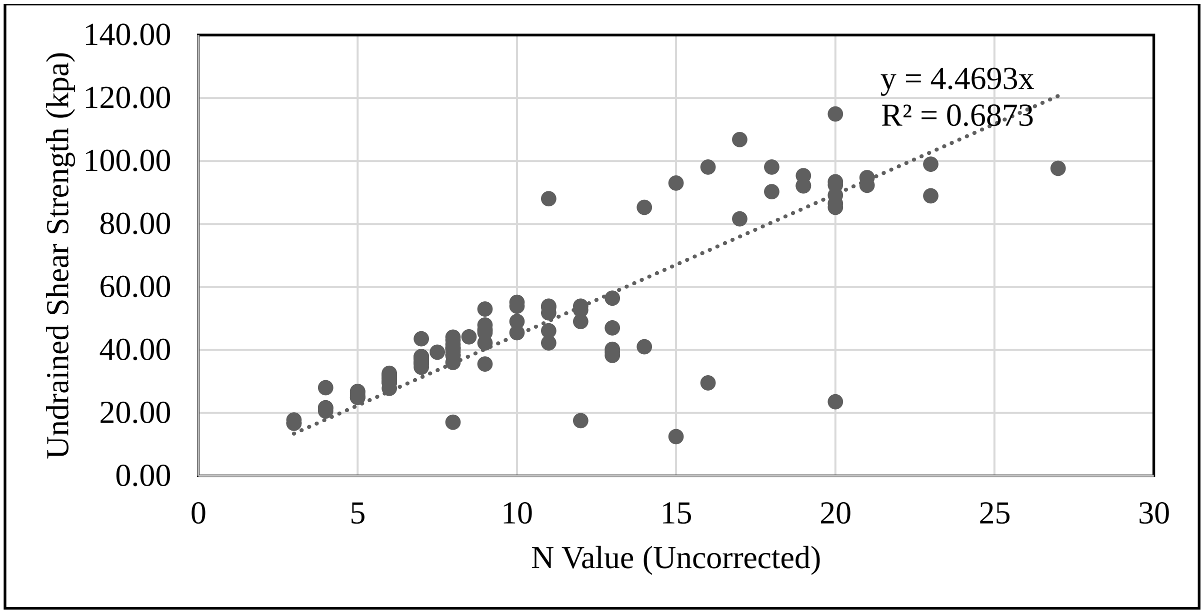

Fig.4.5 Correlation graph of SPT (uncorrected)-s_u for CL soils

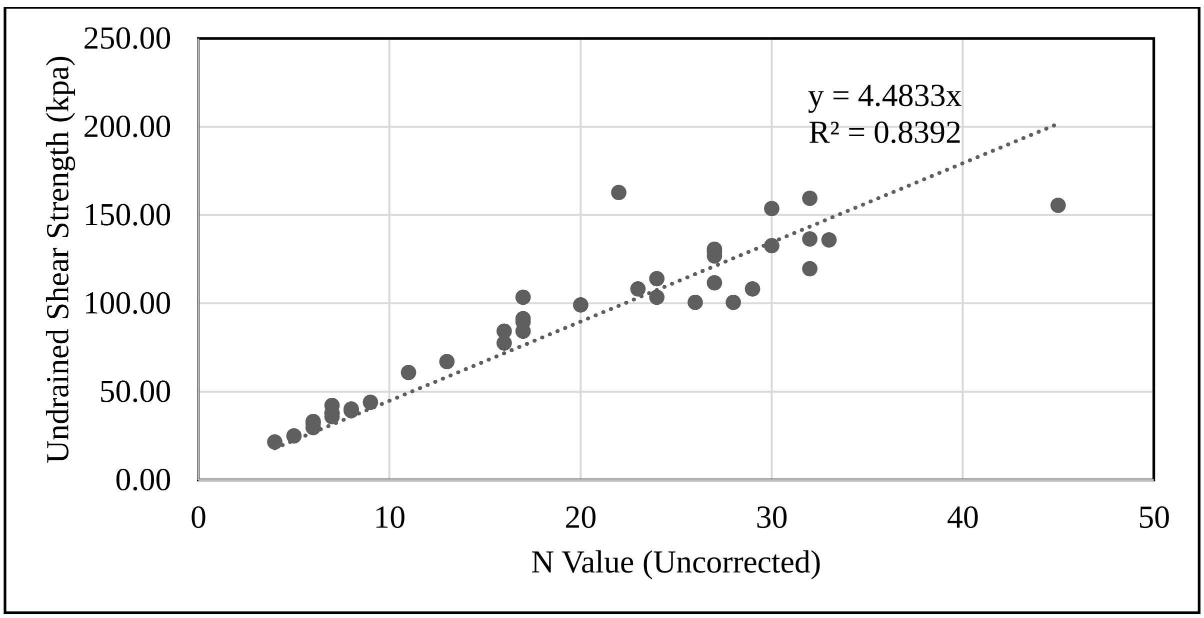

Fig.4.6 Correlation graph of SPT (uncorrected)-s_u for CL-ML soils

Correlation equations between these two variables suggested for fine grained, CL and CL-ML soils are Eq (4.1), Eq (4.2) and Eq (4.3) respectively

$$s_u \text{ (kpa)} = 4.37\ N_{field} \qquad \text{Eq (4.1)}$$

$$s_u \text{ (kpa)} = 4.47\ N_{field} \qquad \text{Eq (4.2)}$$

$$s_u \text{ (kpa)} = 4.48\ N_{field} \qquad \text{Eq (4.3)}$$

4.2.2. SPT (corrected)-UCS Correlation

In this correlation independent variable is corrected SPT N-value and undrained shear strength is dependent variable. N-values are corrected for borehole diameter, hammer cushion, rod length, liner, anvil and energy. Correlation graphs are shown in Fig.4.7, Fig.4.8 and Fig.4.9 for fine grained, CL and CL-ML type soils.

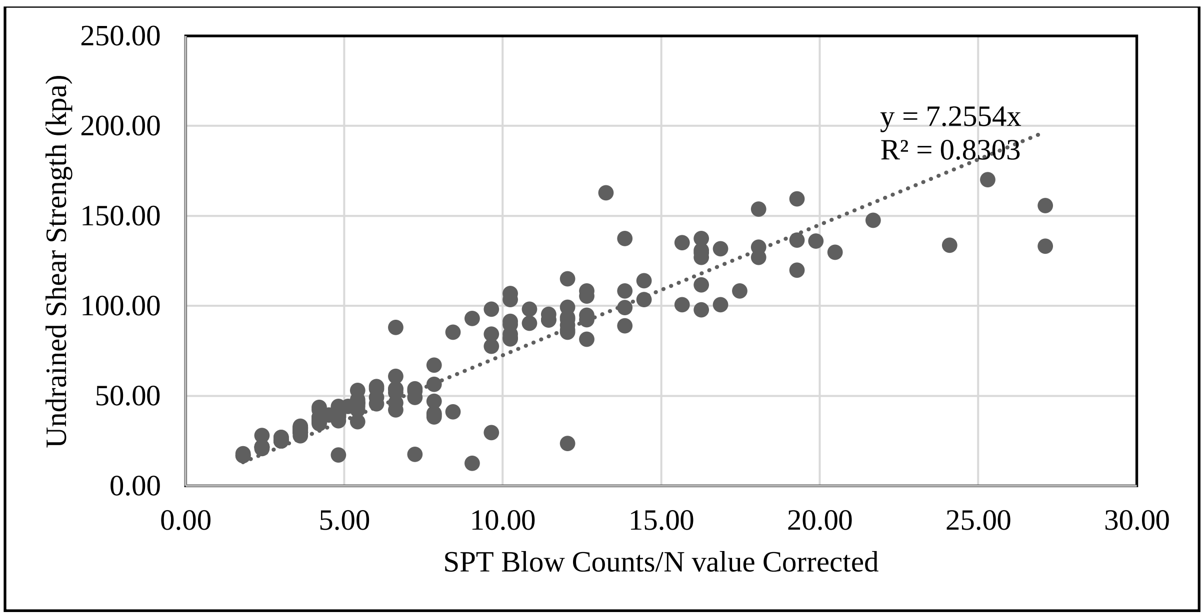

Fig.4.7 Correlation graph of SPT (corrected)-s_u for fine grained soils

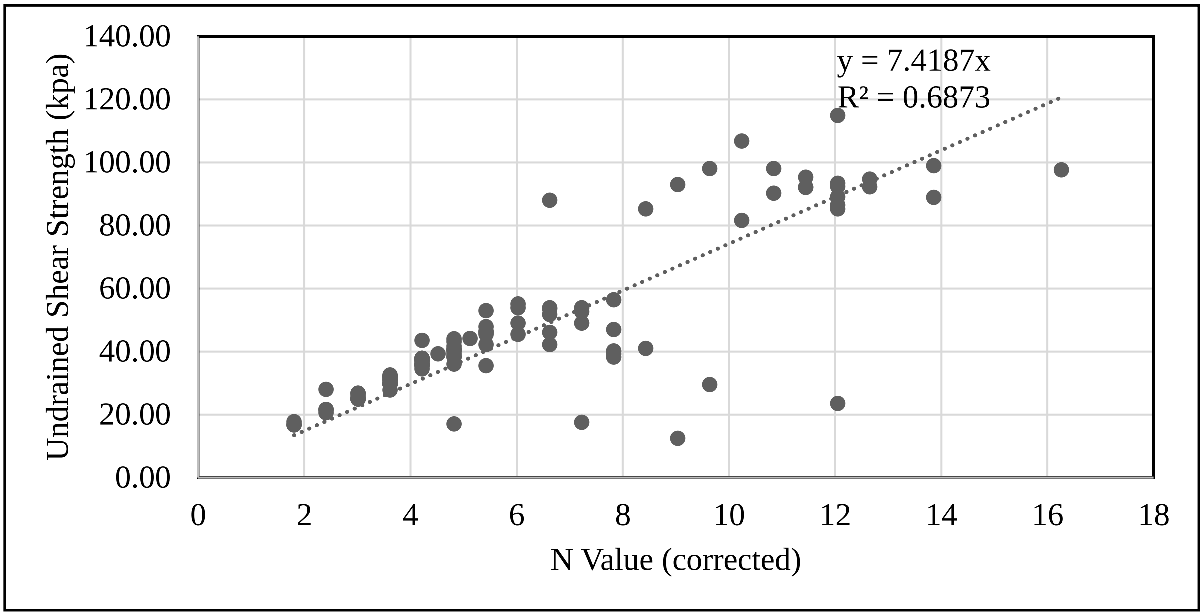

Fig.4.8 Correlation graph of SPT (corrected)-s_u for CL soils

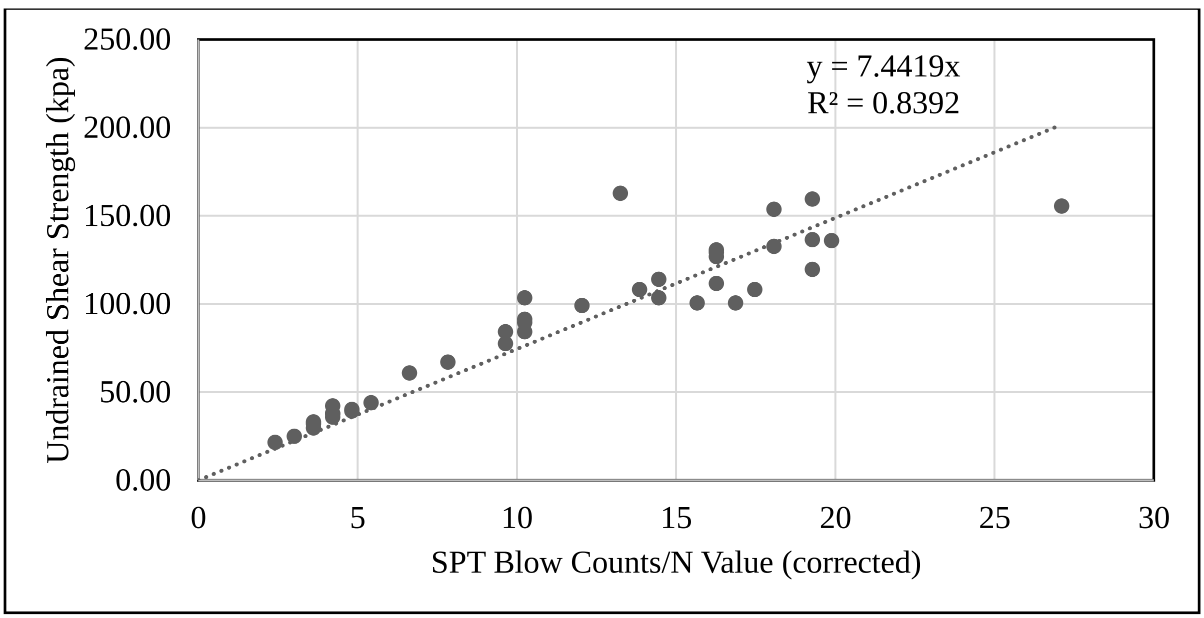

Fig.4.9 Correlation graph of SPT (corrected)-s_u for CL-ML soils

Suggested correlation Eq (4.4) is for fine grained, Eq (4.5) for CL and Eq (4.6) for CL-ML soils.

$$s_u \text{ (kpa)} = 7.25\ N_{60} \qquad \text{Eq (4.4)}$$

$$s_u \text{ (kpa)} = 7.42\ N_{60} \qquad \text{Eq (4.5)}$$

$$s_u \text{ (kpa)} = 7.44\ N_{60} \qquad \text{Eq (4.6)}$$

4.3. PI-UCS Correlation

For developing this correlation Plasticity Index of soil (PI) is considered as independent variable and undrained shear strength is dependent variable. Value of R^2 for this model is very low so correlation can't be found between these two variables. Regression statistics for the model is shown in Fig.4.10. By plotting the graph with different combinations of arithmetic and log scale no significant improvement in correlation was found as it is clear from Fig.4.11 when both the variables are on arithmetic scale, Fig.4.12 when PI is on log scale and undrained shear strength on arithmetic scale, in Fig.4.13 PI is on arithmetic scale and undrained shear strength on log scale and Fig.4.14 when both the variable are on log scale. Value of R^2 in all the cases remain same and even equation is also same.

Regression Statistics	
Multiple R	0.37
R Square	0.14
Adjusted R Square	0.13
Standard Error	36.05
Observations	142.00

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1.00	29234.28	29234.28	22.49	5.1354E-06
Residual	140.00	181953.09	1299.66		
Total	141.00	211187.37			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	104.22	9.23	11.29	2.02E-21	85.97	122.47
X Variable 1	-4.84	1.02	-4.74	5.14E-06	-6.86	-2.82

Fig.4.10 Regression Statistics excel output of PI-UCS model

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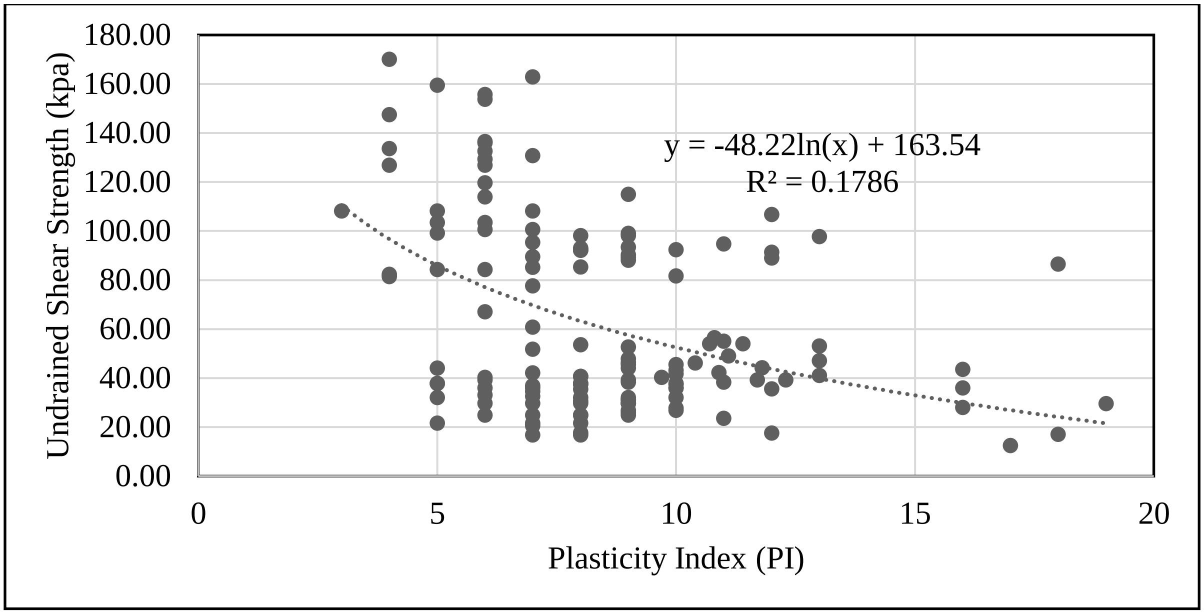

Fig.4.11 Correlation graph PI-UCS

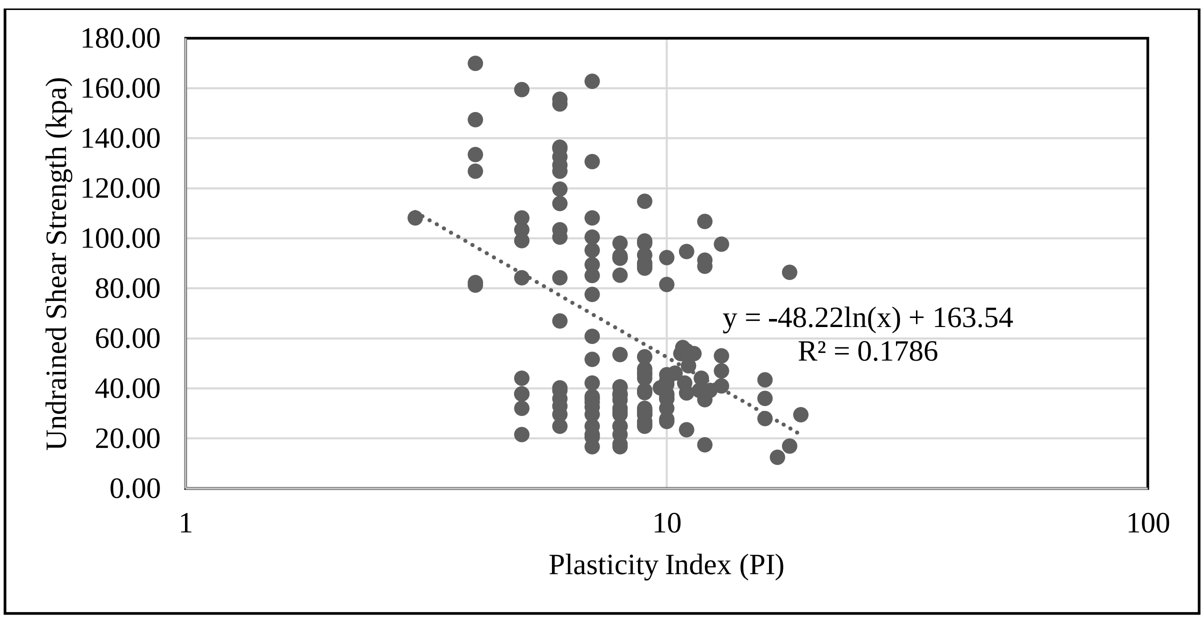

Fig.4.12 Correlation graph PI (log scale)-UCS (arithmetic scale)

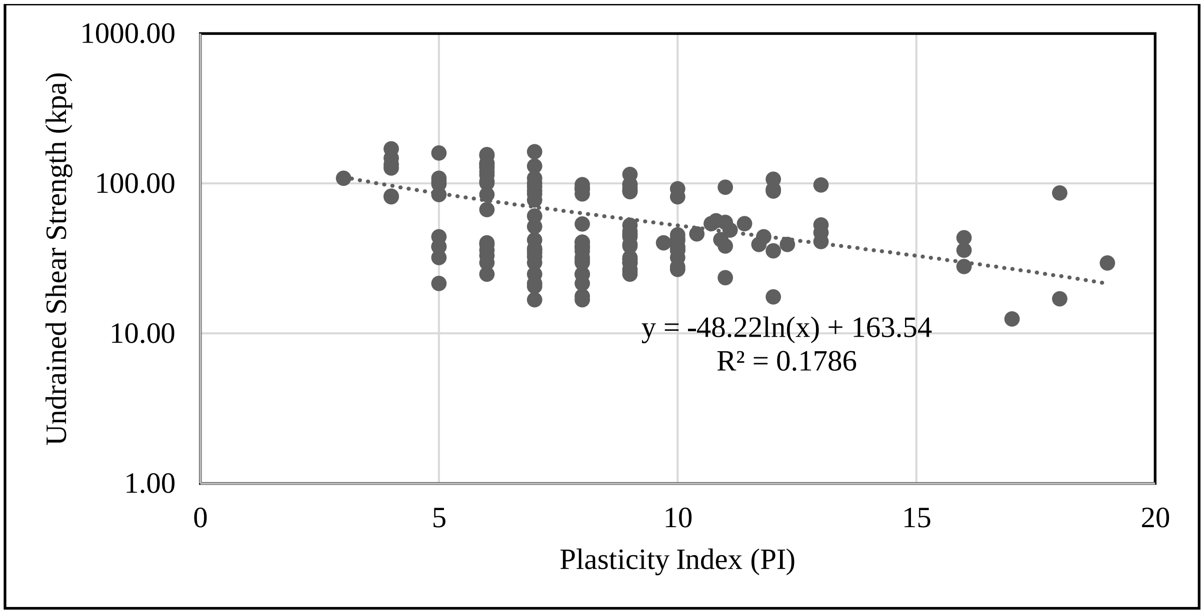

Fig.4.13 Correlation graph PI (arithmetic scale)-UCS (log scale)

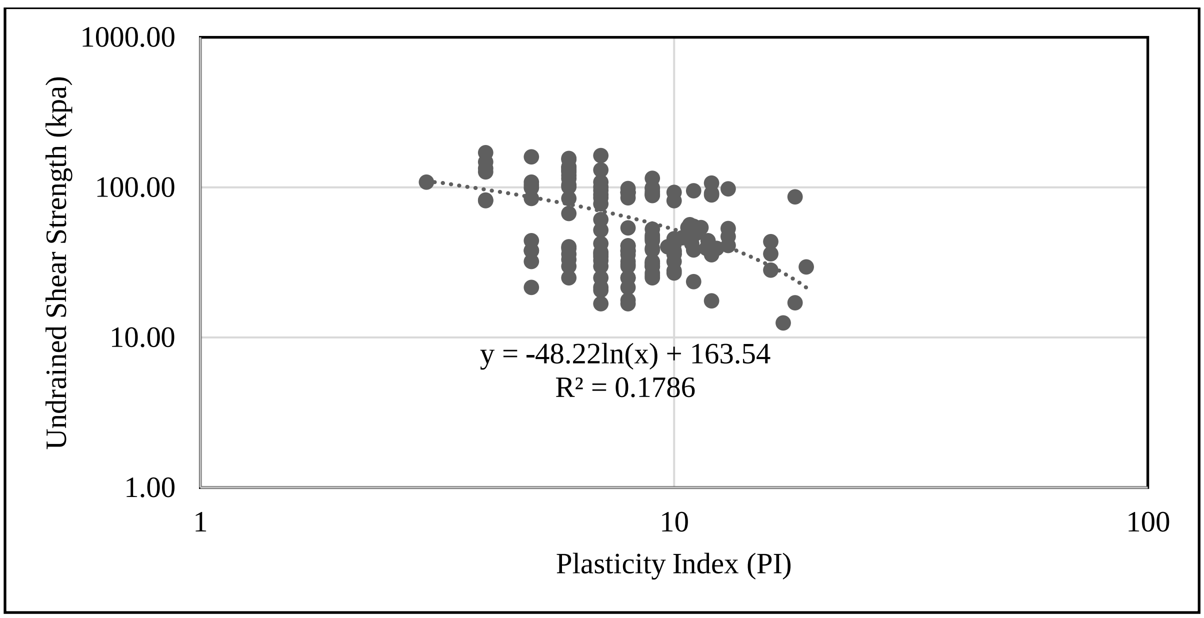

Fig.4.14 Correlation graph PI (log scale)-UCS (log scale)

4.4. SPT-Soil Modulus of Elasticity

Regression statistics for soil modulus at peak strength, 50 % of peak strength and 1 % of strain are shown in Fig.4.15, Fig.4.16, and Fig.4.17. Values of R square, significance of F, t stat and P-value are separately shown in table 4.2. Variables in this correlation correlates moderately when for independent variable; soil modulus at evaluated at peak strength and 50 % of peak strength. However, correlation is weak when modulus is evaluated at 1 % strain. As far as significance of all three models is concerned, for values of Significance F, t stat, p-value first two models are significant and third model is not significant so equation for correlation for modulus at 1 % strain is not suggested.

Table 4.2: Regression statistics of SPT-Soil modulus of elasticity

Modulus at	**R^2**	**Significance F**	**t stat**	**p-value**	**Status**
Peak Strength	0.76	1.15E-15	-3.3	0.00185	Significant
50 % of Peak Strength	0.68	4.13E-11	-2.07	0.04	Significant
1% Strain	0.58	6.53E-09	-0.94	0.35	Not significant

Regression Statistics						
Multiple R	0.87					
R Square	0.76					
Adjusted R Square	0.76					
Standard Error	678.27					
Observations	47.00					
ANOVA						
	df	*SS*	*MS*	*F*	*Significance F*	
Regression	1.00	66633284.69	66633284.69	144.84	1.15449E-15	
Residual	45.00	20702112.30	460046.94			
Total	46.00	87335397.00				
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	-989.39	299.04	-3.31	0.00185106	-1591.69	-387.09
X Variable 1	238.60	19.83	12.03	1.1545E-15	198.67	278.53

Fig.4.15 Regression Statistics excel output of SPT-soil modulus of elasticity at peak strength

Regression Statistics						
Multiple R	0.82					
R Square	0.68					
Adjusted R Square	0.67					
Standard Error	938.27					
Observations	45.00					
ANOVA						
	df	*SS*	*MS*	*F*	*Significance F*	
Regression	1.00	79275601.40	79275601.40	90.05	4.13196E-12	
Residual	43.00	37854825.47	880344.78			
Total	44.00	117130426.87				
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	-888.83	428.37	-2.07	0.04	-1752.72	-24.95
X Variable 1	266.41	28.07	9.49	4.13196E-12	209.79	323.02

Fig.4.16 Regression Statistics excel output of SPT-soil modulus of elasticity at 50% of peak strength

Regression Statistics						
Multiple R	0.76					
R Square	0.58					
Adjusted R Square	0.57					
Standard Error	1115.26					
Observations	41.00					
ANOVA						
	df	*SS*	*MS*	*F*	*Significance F*	
Regression	1.00	67694654.92	67694654.92	54.43	6.53619E-09	
Residual	39.00	48508203.40	1243800.09			
Total	40.00	116202858.33				
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	-484.98	513.20	-0.95	0.35	-1523.03	553.06
X Variable 1	247.90	33.60	7.38	6.536E-09	179.93	315.87

Fig.4.17 Regression Statistics excel output of SPT-soil modulus of elasticity at 1% of peak strain

4.2.3. SPT (Uncorrected)-Soil Modulus of Elasticity

For this correlation SPT value (uncorrected) is independent variable and soil modulus of elasticity is dependent variable. Correlation graph for dependent variable calculated at peak strength and 50 % of peak strength is shown in Fig.4.18 and Fig.4.19 and correlation equations for both the cases are Eq (4.8) and Eq (4.9).

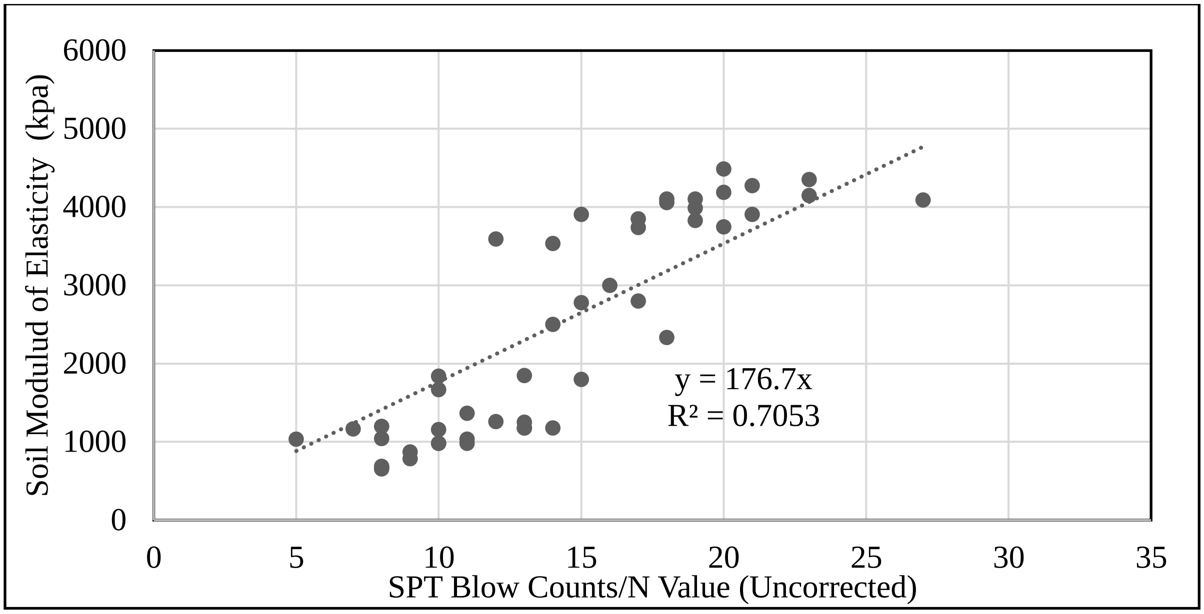

Fig.4.18 Correlation graph of SPT (uncorrected)-soil modulus of elasticity at peak strength

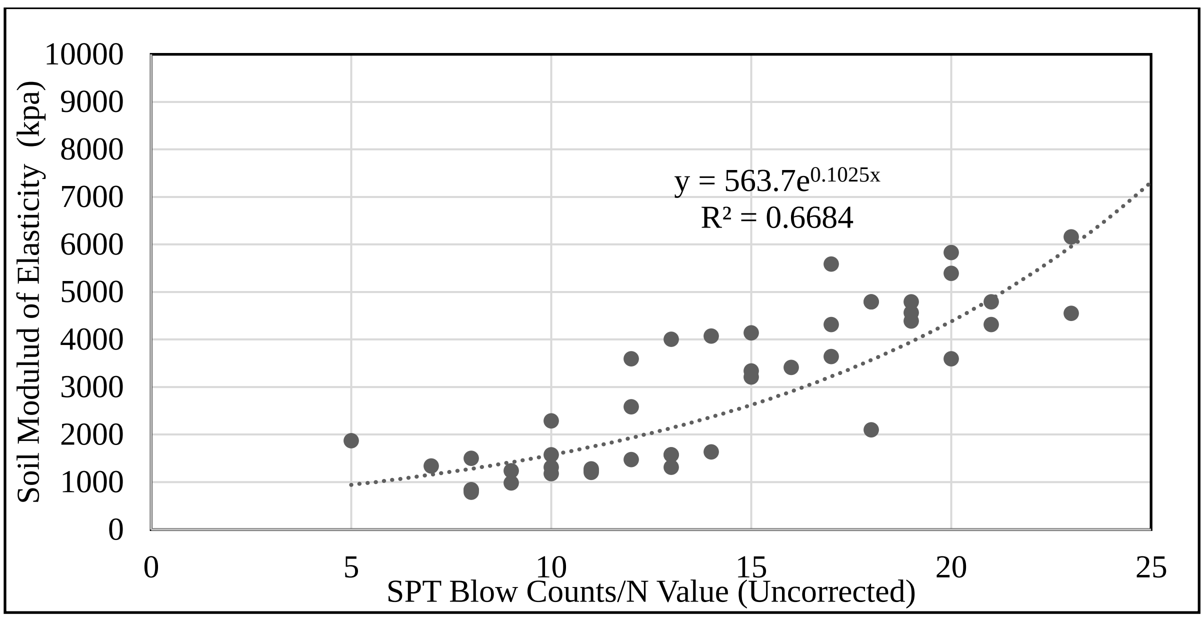

Fig.4.19 Correlation graph of SPT (uncorrected)-soil modulus of elasticity at 50 % strength

$$E_S\,(kpa) = 176.7\ N_{field} \qquad \text{Eq (4.8)}$$

$$E_S\,(kpa) = 563.7e^{0.1025\ N_{field}} \qquad \text{Eq (4.9)}$$

4.2.4. SPT (Corrected)-Soil Modulus of Elasticity

In this correlation SPT value (corrected) is independent variable and soil modulus of elasticity dependent variable. Correlation graphs for modulus at peak strength and 50 % of peak strength are shown in Fig.4.20 and Fig.4.21.

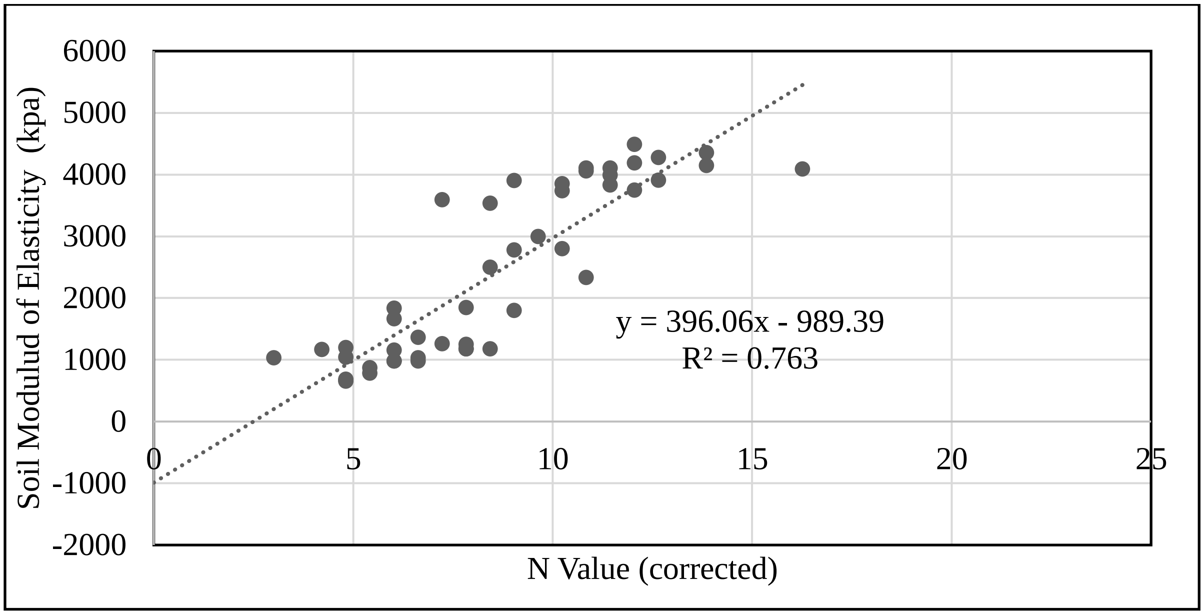

Fig.4.20 Correlation graph of SPT (corrected)-soil modulus of elasticity at peak strength

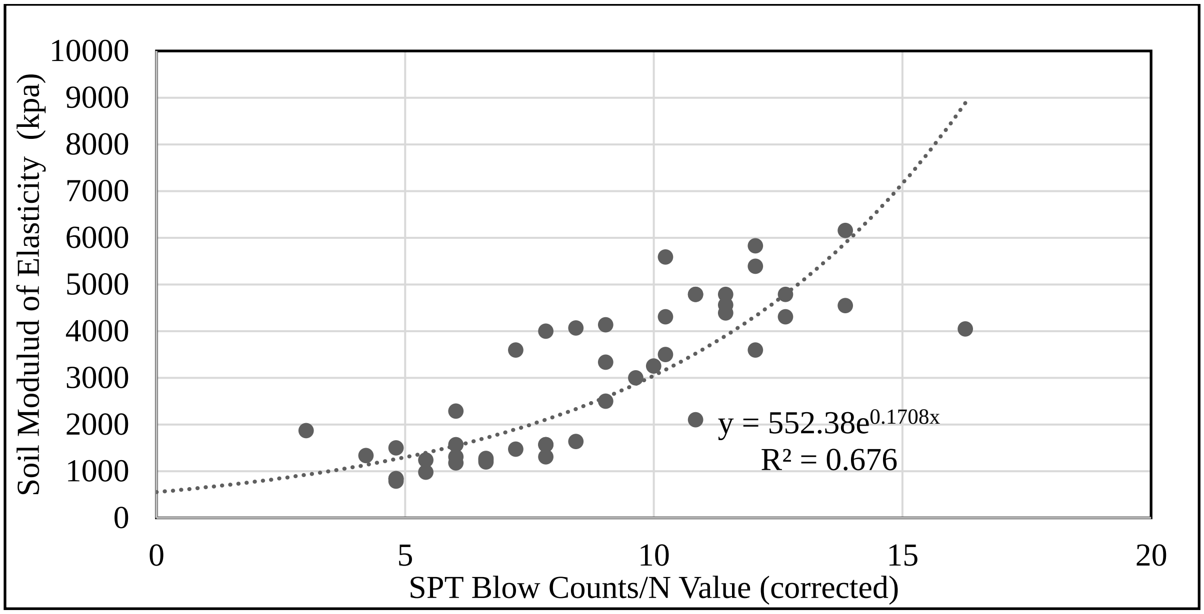

Fig.4.21 Correlation graph of SPT (corrected)-soil modulus of elasticity at 50 % strength

Correlation equations are Eq (4.10) for modulus at peak strength and Eq (4.11) for modulus at 50 % of peak strength and corrected N-value are as below

$$E_S\,(kpa) = 396.06\ N_{60} - 989.39 \qquad \text{Eq (4.10)}$$

$$E_S\,(kpa) = 552.7e^{0.1708\ N_{60}} \qquad \text{Eq (4.11)}$$

Chapter 05

5. Conclusions and Recommendations

5.1. Conclusions

From previous research work it comes to knowledge that some authors have observations regarding the validity of SPT in fine grained soils however, from present study it is clear that SPT is valid for fine grained soils and it holds a strong relationship with s_u of fine grained soils. Equations (4.1) to (4.3) and (4.4) to (4.6) depict that results for both SPT N-value (corrected) and SPT N-value (uncorrected) are very much different from each other.

For fine grained soils

(1) Tarzaghi's equation is close to correlation equation (4.4) after correction of N-values.

(2) The result is same in case of uncorrected N-values and is close for corrected N-values as suggested by Sivrikaya and Togrol (2002).

(3) Proposed correlation equation (4.4) is different from Hettiarachchi and Brown (2009) when N-value is corrected.

For clay proposed correlation Eq (4.2) and (4.5) are different form Decourt (1990). However in other studies for clay and silty clay by Sangleart (1972) and Nixon (1982) it not known that N-values are either uncorrected or corrected.

From the statistics of model for correlation between plasticity index and s_u it is obvious that there is hardly any correlation between plasticity index and s_u.

Soil modulus of elasticity though depends upon many factors which influence it. But it has correlation with SPT equations (4.8) to (4.10) are suggested in present study. Suggested equation (4.8) for uncorrected N-values is same as suggested by L.Behpoor and A. Ghahraman (1989).

5.2. Recommendation

1. Keeping in view the previous research and present study it is recommended to develop and use correlation for the specific area.
2. Tarzaghi equation to be used after correcting N-value because it is very much close to correlation equation after correction of N-values
3. While using the correlation it is important to know whether it is for corrected N-values or uncorrected N-values. So there should be clear distinction while developing and using correlation either N-value is corrected or uncorrected.
4. SPT corrections for the developed equation must be known

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