EVALUATION OF SHANSEP PARAMETERS FOR SRI LANKAN COHESIVE SOILS

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DECLARATION

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Name: Prof. U.P. Nawagamuwa

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ABSTRACT

This thesis contains a study on SHANSEP parameters, evaluated for Sri Lankan cohesive soils. In Sri Lanka, various correlations are used for the evaluation of shear strength parameters and the settlement. Most of such correlations are developed in overseas countries, which may not accurately model the behavior of Sri Lankan soils as they are developed from other geological conditions.

Though the undrained shear strength is a function of both stress history and stress path, most of the time, they are not considered, which may lead to large errors. The SHANSEP model proposed by Prof. Charles C. Ladd, shows the normalize behavior of the cohesive soils which consider both stress path and stress history in determination of the Undrained Shear Strength of Soils. Therefore, this research is an effort to see the applicability of SHANSEP model for Sri Lankan cohesive soils, using the test data provided by major projects in Sri Lanka.

Since CK0U Triaxial testing facilities are not available in Sri Lanka, data from field vane shear test have been used for the estimation of Undrained Shear Strength in this study. Finally, a SHANSEP equation has been proposed in this thesis for selected alluvial clay soils along with two more conservative equations for the estimation of undrained shear strength and over consolidation ratio respectively.

***Key words**: SHANSEP, Normalized Undrained Shear Strength, Over Consolidation Ratio, Field Vane Shear, Alluvial Soil*

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01. INTRODUCTION

1.1 Background

It is very important to know the strength, settlement, permeability and physical parameters of the subsurface soil profile when carrying out a geotechnical design. One of the important parameters is shear strength of the soil. There are many methods introduced to evaluate the shear strengths of soils such as, in-situ tests, laboratory tests or use of empirical equations. However, most of such empirical equations have been developed using the data obtained for foreign soils with different geological formations than the Sri Lankan soils.

The undrained shear strength (S_u) of clay depends on soil type, moisture content, soil structure, stress history and stress path during undrained loading. The inconsideration of the stress history and stress path, may lead to large errors when estimating the undrained shear strength. Further, neglecting the stress history in the estimation of the compressibility of the soil under a certain stress increment is also in error. Hence, estimation of the consolidation settlement, a common problem in Sri Lanka when designing of domestic and other small to medium structures, may also be in serious error. Therefore, it is important to find a method to determine the undrained shear strength, considering the stress history and stress path.

The SHANSEP (Stress History and Normalized Soil Engineering Properties) is a method which considers stress history and stress path, developed by Professor Charles C. Ladd. This is based on the previous researches such as Skempton (1957) on the normalized undrained shear strength of clayey soils. It has been found that the undrained shear strength of most of the soils can be normalized by the effective vertical consolidation pressure.

Several researches are available to estimate normalized undrained shear strength of soils (Skempton (1957), Bjerrum and Simons (1960), Karlson and Viberg (1967)). However, all these relationships are only valid for normally consolidated soils.

The SHANSEP model is based on the relationship between the normalized undrained shear strength and the OCR. Therefore, the SHANSEP method can also be used for the settlement analysis by estimating the OCR of the soil of known undrained shear strength.

No research has been carried out on SHANSEP for Sri Lankan soils. Therefore, this research will be much helpful in future references for more accurate estimations of undrained shear strengths of Sri Lankan soils.

1.2 Objectives

1. Evaluation of applicability of SHANSEP model to Sri Lankan alluvial mineral cohesive soils through currently available information.
2. Calibration of SHANSEP model for the Sri Lankan alluvial mineral cohesive soils, if it is applicable to alluvial mineral cohesive soils in Sri Lanka.

02. LITERATURE REVIEW

2.1 Soil Formation

The soils are formed by the disintegration of rocks caused by weathering or disintegration of organic matter. There are three types of weathering (*Weathering*, n.d.).

i. Physical Weathering – The rock disintegration caused by the physical processes such as temperature fluctuations, freezing, wind, rain etc.

ii. Chemical Weathering – The rock disintegration caused by the reaction of water, oxygen, carbon dioxide, acids etc. with the minerals of the rocks.

iii. Biological Weathering – The rock disintegration due to the activities of living organisms.

2.2 Residual and Transported Soils

The soils can be remained at the same location after the weathering and formation. These soils are called residual soils. Soils which are transported from one place to another place after the formation are called transported soils. There are several types of transported soils which are categorized according to the mode of transportation (Neenu, 2019).

- Alluvial Soils – Soils which were transported by means of water are called alluvial soils. The soils are transported by either suspension or rolling in water.
- Aeolian Soils – Soils which were transported by means of wind are called as aeolian soils.
- Glacial Soils – Soils which were transported with the movement of glaciers are called as glacial soils.

- Colluvial Soils – Soils which were transported and deposited by means of their gravity are called as colluvial soils.

2.3 Alluvial Soils in Sri Lanka

Alluvial soils are deposited by the surface water by either suspension or rolling in water. They can be found along river beds, deltas, flood plains etc. (*What Are Alluvial Soils?*, 2020).

In Sri Lanka, the alluvial soils occur all over the country usually in narrow strips in the valleys and flood plains. The area of the alluvial soils in Sri Lanka is believed to be around 1.5 million acres (Moorman & Panabokke, 1961).

The alluvial soils are very fertile in general (Prathibha, 2020). A large proportion of alluvial soil areas are used for rice cultivation (Moorman & Panabokke, 1961). However, these soils are also utilized for the cultivation of sugarcane and cone.

2.4 Composition of Soils

The soils are composed of minerals, organic matter, water, air and organisms. However, according to the particle size and the plasticity properties, the inorganic soils, considered in this research can be classified mainly into following categories (Loganathan, n.d.).

- Gravel
- Sand
- Silt
- Clay

The soils which contain silt and clay as the major constituent are called as fine-grained soils. Similarly, the soils which contain gravel and sand as major constituents are called as coarse-grained soils. The fine-grained soils which shows cohesive strength and plasticity properties are called as cohesive soils (Gautam, 2018). The grouping of soils to above categories are done using soil classification systems and one such commonly used soil classification system called Unified Classification System (commonly known as USCS), is described in section 2.4.1.

2.4.1 Unified Soil Classification System

The Unified Soil Classification System (USCS) is a widely used soil classification system. This has been adopted from the ASTM D 2487 standard. In this system, two capital letters are used as symbols to describe a soil. However, in some cases, dual notations are used as symbols, such as SW-SC which is the notation for “Well-graded sand with clay”.

2.4.1.1 Identification of Fine-Grained Soils

As per USCS, if the material passing No. 200 sieve is 50% or more, the soil is considered as a fine-grained soil. However, in order to identify the soil as clay or silt, the plasticity chart has to be referred which requires Liquid Limit and Plasticity Index values. Basically, if the plasticity values come beneath the A-line of the plasticity chart, the soil is a silt. If the plasticity values are above the A-line, the soil is a clay. For a detailed classification of the fine-grained soils, refer Figure 2.1: Plasticity Chart of Unified Soil Classification Systemfor plasticity chart.

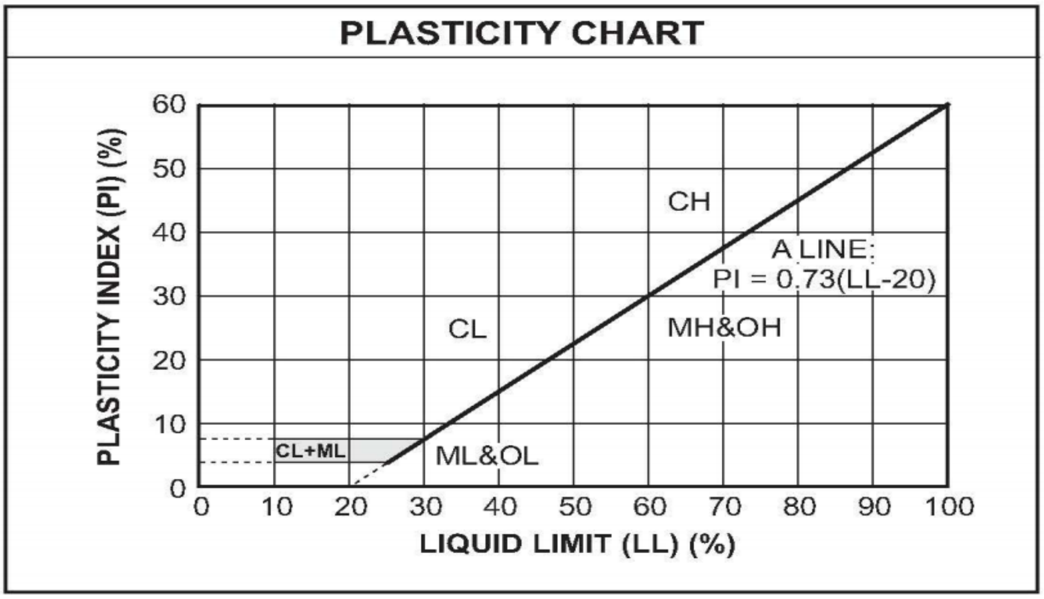

Figure 2.1: Plasticity Chart of Unified Soil Classification System

2.5 Activity and Sensitivity of Clays

The ratio between the plasticity index and the clay fraction is approximately constant for a certain clay soil. This ratio is defined as the activity of clay (Skempton, 1953). The strength ratio between undisturbed and remoulded clays is defined as sensitivity of clays. This ratio varies from 1 to over 100. For heavily overconsolidated soils, the sensitivity is 1.0 and for quick clays, the sensitivity is over 100 (Skempton & Northey, 1952).

2.6 Normalized Undrained Shear Strength

In between 1950 - 1970, it has been observed that the shear strength of most of the soils follows a characteristic pattern. It has been found that the shear strength of the soils, can be normalized with respect to the effective vertical consolidation pressure.

2.6.1 Shear Strength vs Atterberg Limits for Normally Consolidated Soils

Skempton (1957) proposed the following equation for the relationship between the normalized undrained shear strength and the plasticity index (PI) of soils which gives a linear relationship (Chaney, 1986) . The shear strengths obtained from field vane shear test have used for this study.

$$\frac{S_u}{\sigma'_{V_0}} = 0.11 + 0.0037 I_p$$

S_u - Undrained Shear Strength

σ'_{V_0} - In-situ effective Vertical Stress

I_p - Plasticity Index

However, Chaney (1986) states that this equation can only be applied for clay with following characteristics.

- Normally Consolidated
- Fairly uniform
- Low activity and sensitivity

Bjerrum and Simons (1960) proposed a similar equation for normally consolidated clays which shows a relationship between the normalized undrained shear strength and the plasticity index (for PI > 50%) (Obasi & Anyaegbunam, 2005).

$$\frac{S_u}{\sigma'_{V_0}} = 0.045\, {I_P}^{0.5}$$

Bjerrum and Simons (1960) proposed another equation for normally consolidated clays which gives a relationship between Liquidity Index (LI) and normalized undrained shear strength (LI > 50%) (Obasi & Anyaegbunam, 2005).

$$\frac{S_u}{\sigma'_{V_0}} = 0.18\, /\, LI^{0.5}$$

Where;

$$LI \;=\; \frac{W_n - W_P}{W_L - W_P}$$

W_L – Liquid Limit (%)

W_n – Natural Moisture Content (%)

W_P – Plastic Limit (%)

The data from the Field Vane Test (FVT) have been used for the both relationships proposed by Bjerrum and Simons (1960) (Ng et al., 2014).

Karlsson and Viberg (1967) developed the following formula for normally consolidated Swedish clays of LL > 20% (Karlsson & Viberg, 1967; Obasi & Anyaegbunam, 2005). The $\frac{S_u}{\sigma'_{V_0}}$ has been determined by using the data from Field Vane Test (Karlsson & Viberg, 1967).

$$\frac{S_u}{\sigma'_{V_0}} = 0.5\ \frac{LL}{100}$$

2.6.2 Normalized Shear Strength vs Over Consolidation Ratio

Since it has been observed that the OCR has much influence to the $\frac{S_u}{\sigma'_v}$ than Plasticity Index, Ladd and Foote (1974) developed a linear relationship between Over Consolidation Ratio and the normalized shear strength of the soils, which can be used for both normally consolidated and over consolidated soils. This model is called Stress History and Normalized Soil Engineering Properties (SHANSEP). The relationship is shown below (Bay et al., 2005).

$$\frac{S_u}{\sigma'_V} = S\ x\ (OCR)^m$$

Where;

S - Normally consolidated ratio of $\frac{S_u}{\sigma'_v}$

OCR - Over Consolidation Ratio

m - exponential (usually between 0.75 – 1.00)

The Figure 2.2: Normalized Behaviour of Soil illustrate the normalized behaviour of the soils (Bay et al., 2005).

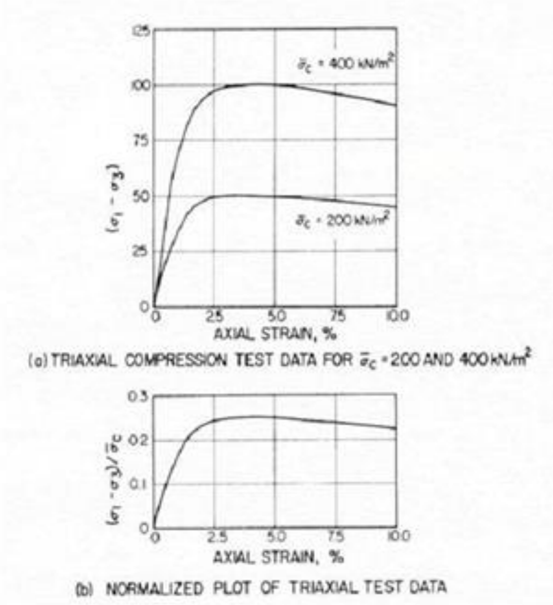

Figure 2.2: Normalized Behaviour of Soil

In the top curve, axial strain is plotted on x-axis and the deviator stress is potted on the y-axis. The same has been plotted in the lower curve, except that the deviator stress is normalized by the confining pressure (σ_c).

Bay et al. (2005) evaluated SHANSEP parameters for the soft Bonneville clays followed by the work by Ladd (1989) and Ng (1998) (Bay et al., 2005).

Bay et al. (2005) developed the following formula for the soft Bonneville Clays as shown in Figure 2.3: Normalized Plot of the Equation Proposed by Bay et al. (2005).

$$\frac{S_u}{\sigma'_V} = 0.32\ x\ (OCR)^{0.82}$$

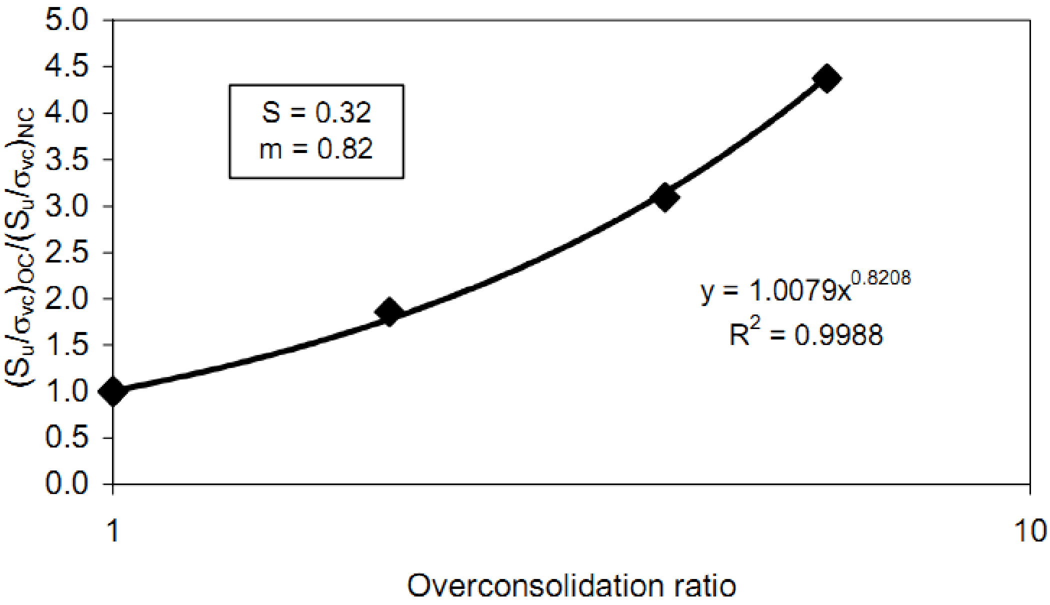

Figure 2.3: Normalized Plot of the Equation Proposed by Bay et al. (2005)

In this work Bay et al. (2005) mainly used CK_0U Tri axial Shear Test data and 1-D consolidation data for the development of the equation. The CK_0U Tri axial Shear tests have been performed for the OCRs ranging from 1 to 6.

Summary of the work by Bay et al. (2005) which followed Ladd and Foote (1974) is shown below.

- Selecting samples and carrying out 1-D consolidation test to determine the preconsolidation pressure (σ'_{vo}).
- Consolidate the specimen taken from the above samples to different preconsolidation pressures (σ'_c) of 1.5, 2.5 and 4.0 times the σ'_{vo}.
- Normally consolidated tests are run for the above consolidated specimens to evaluate the S_u/σ'_c.
- If S_u/σ'_c gives constant relationships, the SHANSEP is applicable.
- The lowest pressure which gives a constant relationship of S_u/σ'_c is selected as the laboratory consolidation pressure (σ'_{vm})

- Specimens from same samples are consolidated to the σ'_{vm} and allowed to swell for known OCRs between 1.5 to 6.0
- Shearing is carried out and evaluated the relationship between S_u/σ'_{vm} and OCRs.

2.7 K_0 and Isotropic Consolidation

2.7.1 K_0 Consolidation of In-situ Soils

Many of natural soils have been deposited without lateral strain. Therefore, horizontal and vertical stresses of these natural soils are different. These stresses are expressed by the coefficient of earth pressure at Rest, which is denoted by K_0.

$$K_0 = \frac{\sigma'_h}{\sigma'_v}$$

Where;

σ'_h - Effective Horizontal Stress

σ'_v - Effective Vertical Stress

The Figure 2.4: Behaviour of Soil Particles during Consolidation illustrates the behavior of the soils of an embankment construction, during consolidation with lateral displacements and without lateral displacements.

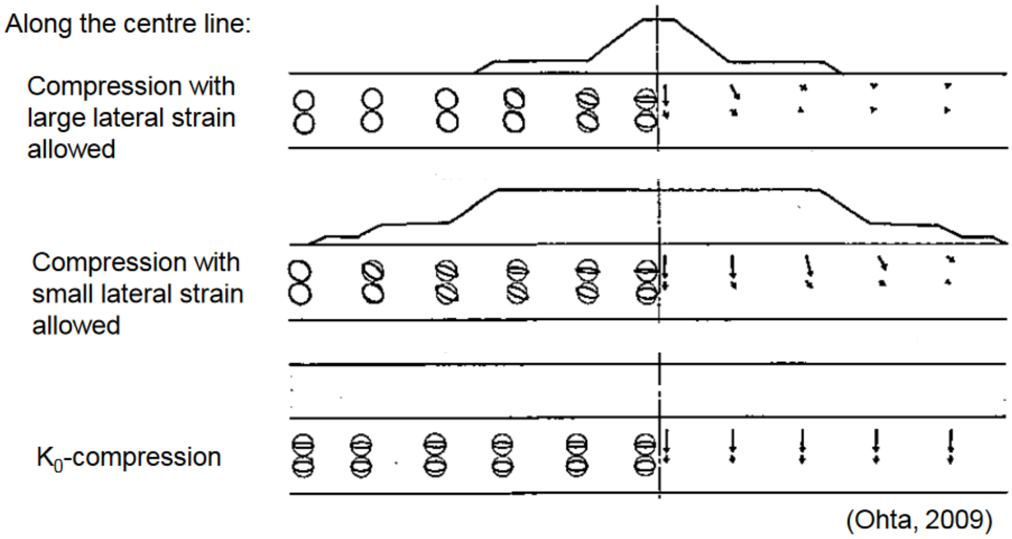

Figure 2.4: Behaviour of Soil Particles during Consolidation

2.7.2 K_0 Consolidation Test in the Laboratory

The common oedometer performs the consolidation of the sample, in K_0 condition. The soil sample is placed in a confining ring so that there will be no lateral strain. A porous disc is placed on top of the sample in order to allow the pore water dissipate from the sample. The load is applied from top of the sample as shown in Figure 2.5: Horizontal and Vertical Stresses of the Sample in Oedometer – Nishimura (n.d.).

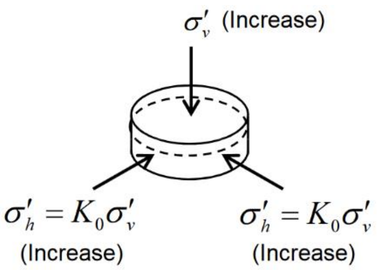

Figure 2.5: Horizontal and Vertical Stresses of the Sample in Oedometer – Nishimura (n.d.)

The loading of oedometer can be done by two methods.

I. Step loading

This is the conventional method of doing oedometer test, where the dead loads are added step by step in order to increase the vertical stress on to the sample.

II. Constant Rate of Strain

The load is increased continuously in order to apply the vertical stress on to the sample at a constant strain rate as shown in Figure 2.6: Vertical Stress vs Void Ratio - Nishimura (n.d.).

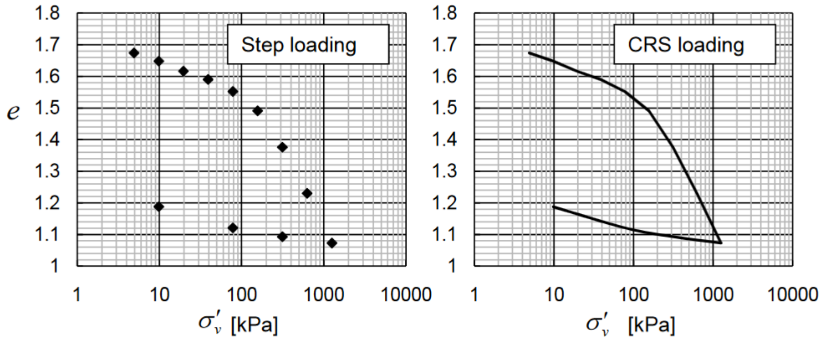

Figure 2.6: Vertical Stress vs Void Ratio - Nishimura (n.d.)

2.7.3 K_0 Consolidation in Stress Path Cell

In some cases, it is important to carry out the consolidation stage of triaxial shear tests, without allowing for lateral strain, keeping the cross-section area constant. This test is called as K_0 Consolidated Undrained Shear Test which is denoted by CK_0U. The CK_0U test has been introduced by Bishop and Henkel (1962) (Piriyakul & Haegeman, 2005).

During the consolidation stage of CK_0U, the horizontal deformation of the sample should be kept in the range of ±1 µm. Therefore, the volume change is almost equal to the axial deformation times the cross section. The test is also started without excess pore water pressure and increment of excess pore water pressure is not allowed during the consolidation stage. Both axial and radial deformations are measured using local strain gauges as shown in the Figure 2.7: Mounting of axial and radial sensors in the stress path cell - Piriyakul and Haegeman (2005).

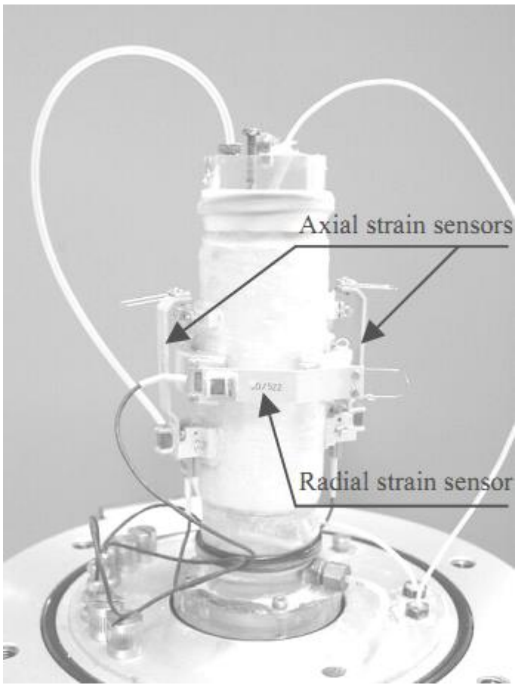

Figure 2.7: Mounting of axial and radial sensors in the stress path cell - Piriyakul and Haegeman (2005)

2.7.4 Isotropic Consolidation

The isotropic consolidation in the laboratory is easier in the triaxial cell than the K_0 consolidation. During the isotropic consolidation, the sample is given the same stresses in three dimensionally as shown in Figure 2.8: Stresses during Isotropic Consolidation (Nishimura, n.d.).

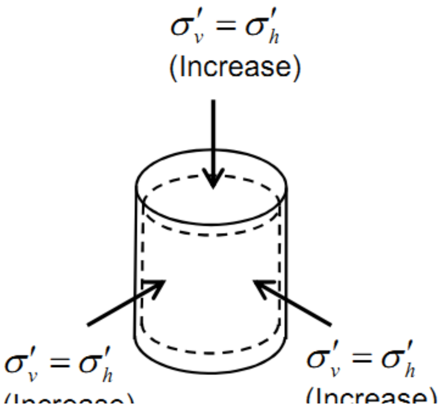

Figure 2.8: Stresses during Isotropic Consolidation (Nishimura, n.d.)

2.7.5 Comparison of UU, CIU and CK_0U triaxial shear tests

It has been found that the S_u obtained from the Isotropically Consolidated Undrained triaxial (CIU) test is higher than the S_u values obtained from CK_0U as shown in Figure 2.9: Normalized Undrained Shear Strength vs OCR (Almeida & Marques, 2013).

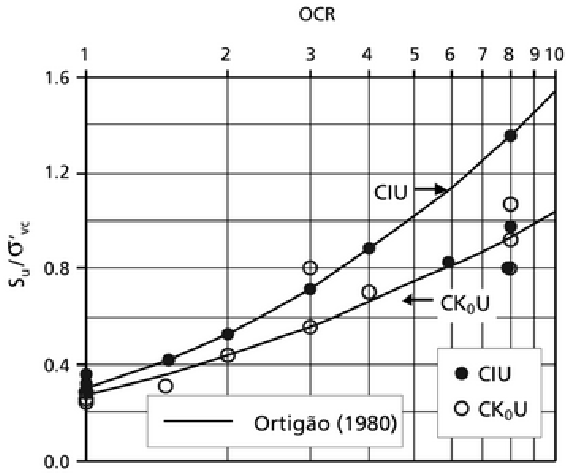

Figure 2.9: Normalized Undrained Shear Strength vs OCR

As per the 8, the normalized S_u obtained from both CIU and CK_0U are almost same at OCR = 1. However, the difference between the normalized S_u values obtained from CIU and CK_0U is getting higher, since the normalized S_u obtained from CK_0U give significantly lower values than CIU. Further, as per Chaney (1986), the CIU gives much higher strength values than the field vane shear test data. Therefore, as per both Almeida & Marques (2013) and Chaney (1986), the CIU gives higher values than that of the CK_0U and Vane Shear Test.

The conventional Unconsolidated Undrained (UU) shear tests give large errors about ± 25 – 50% for strength estimations (Ladd & DeGroot, 2003). The three reasons leading to this error are as follows.

i. The faster shearing speed increases the estimated S_u.

ii. Ignoring of anisotropy increases the estimated S_u.

iii. Sample disturbance decreases the estimated S_u.

Use of overestimated strength parameters in designs is unsafe while use of underestimated strength parameters is not cost effective. As per Ladd & DeGroot (2003), carrying out conventional UU and CIU shear tests is waste of time and money and they propose to adopt for more accurate estimation of shear strength by CK_0U shear tests.

2.8 Use of Vane Shear Test for SHANSEP

Several studies have been carried out for estimating SHANSEP parameters using field vane shear test data.

The studies carried out for following soils by Lacasse et al. (1978) and Ladd et al. (1983) have been showed that the field vane shear test data can be used to develop the SHANSEP equation proposed by Ladd et al. (1974) (Jamiolkowski et al., 1985).

The vane shear test uses a four-blade vane (as shown in Figure 2.10: Geometry of Field Vanes - ASTM D2573-01) to shear a cylindrical surface of an undisturbed cohesive soil by applying a torque from the surface. This torque is converted to the shear resistance of the soil (*ASTM D2573-01*, 2001). The shear resistance is plotted against the rotation (degrees) as shown in Figure 2.11: Plot of the Rotation (Degrees) vs Shear Stress in Field Vane Shear Test.

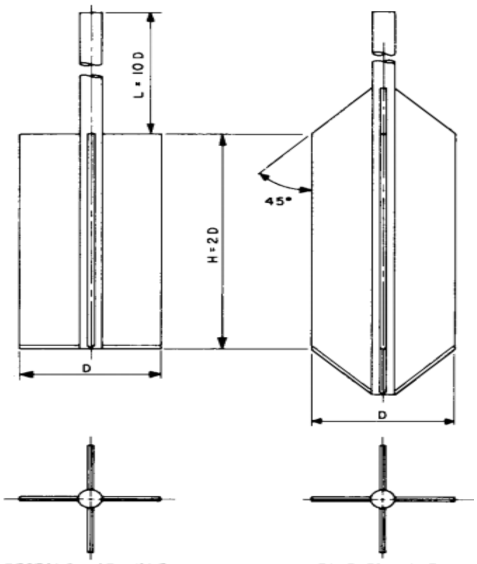

Figure 2.10: Geometry of Field Vanes - ASTM

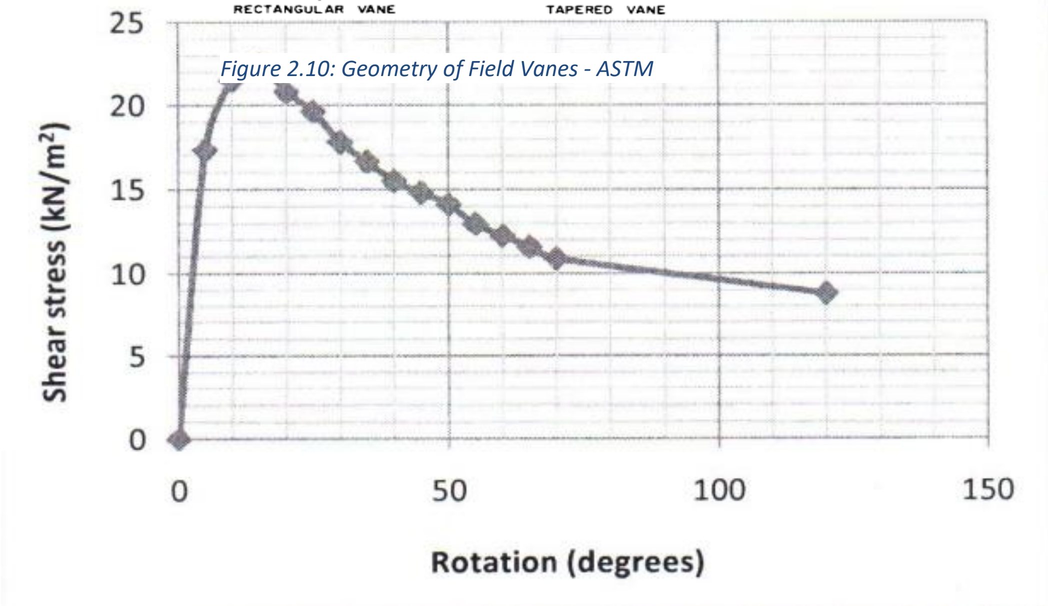

Figure 2.11: Plot of the Rotation (Degrees) vs Shear Stress in Field Vane Shear Test

The maximum shear strength is called as the Peak Shear Strength. The final part of the curve which is usually almost flat, is taken as the Residual Shear Strength. The Peak Shear Strength is taken as the Undrained Shear Strength (S_u).

The following assumptions are made for estimating shear strength by vane shear test (Trishna, 2018).

I. Shear strength is equal in both horizontal and vertical direction

II. The shear surface is cylindrical in shape with a diameter equal to the diameter of the vane.

III. At the peak of the torque, the shear strength at the end surface is equal to the center value.

After calculating the shear strength from the vane shear test in the field $(s_u)_{fv}$, it is important to correct the value before applying it in geotechnical analysis such as stability analyses and bearing capacity of soft soils. The mobilized shear strength ($\tau_{mobilized}$) can be obtained by the following equation.

$$\tau_{mobilized} = \mu_v (s_u)_{fv}$$

μ_v is a factor that depends on the plasticity properties of the soils. The ASTM D 2573-01 recommends to use the following equation provided that the PI > 5%.

$$\mu_v = 1.05 - b\,(PI)^{0.5}$$

Where b depends on the time to failure (t_f, in minutes) and is expressed by;

$$b = 0.015 + 0.0075 \log(t_f)$$

The correction factors proposed by the Bjerrum (1972) is also widely used. The Figure 2.12: Field Vane Correction Factor vs Plasticity Index - Ladd and Degroot (2003) shows the data used by Bjerrum (1972) and several other researchers to develop the correlation for the correction factor (μ) of the vane shear test.

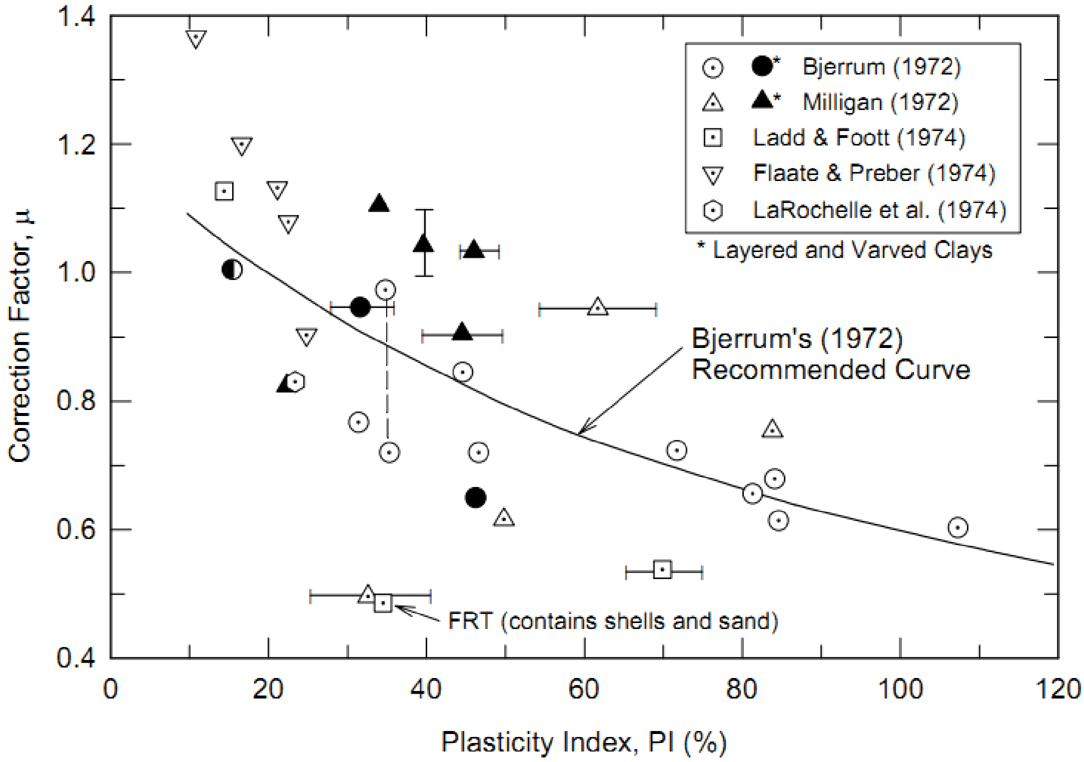

Figure 2.12: Field Vane Correction Factor vs Plasticity Index - Ladd and Degroot (2003)

03. METHODOLOGY

This research is based on the data on alluvial soils, obtained from the major projects in Western and North-Western parts of Sri Lanka. These alluvial soils are not pure clays and therefore, they consist of gravel, silt and/or sand too.

As per the literature, Ladd and Foote (1974), Ladd (1989), Ng (1998) and Bay et al. (2005) mainly used following laboratory tests for the analyses.

- 1-D Consolidation Test
- CK_0U Triaxial Shear Test

In Sri Lanka, it is not possible to find data on CK_0U Triaxial shear test. Even though it is possible to find data on CU (Isotropic) Triaxial shear data, the available data is not sufficient for the research. Further the UU or CIU does not give accurate estimations (Ladd & DeGroot, 2003).Therefore, the field vane shear test (FVT) data was used which is similar to the studies done by Lacasse et al. (1978) and Ladd et al. (1983) (Jamiolkowski et al., 1985).

The following data from major construction projects were used in the analyses.

- Description of Soil
- Sieve Analysis test
- Atterberg Limits test
- Hydrometer Analysis test
- 1-D Consolidation test
- Ground Water Level
- Vane shear test

The methodology followed in this research is as follows.

1. Obtain OCR's for each specimen

- Pre-consolidation pressure (σ'_c) is obtained using the 1-D consolidation data.

- Effective vertical pressure (σ'_v) is calculated using the ground water level, specimen depth and bulk densities (assumed).

- OCR is estimated as $\frac{\sigma'_c}{\sigma'_v}$.

2. Eliminate data which contains abnormal figures such as abnormally high OCR values.

3. Identify the clay soils by analyzing the data according to USCS classification system.

4. Obtain logarithm of both side of the SHANSEP equation.

$$\log\left(\frac{S_u}{\sigma'_V}\right) = \log(S\ x\ OCR^m)$$

$$\log\left(\frac{S_u}{\sigma'_V}\right) = \log S + \log(OCR^m)$$

$$\log\left(\frac{S_u}{\sigma'_V}\right) = m \text{ x} \log(OCR) + \log S$$

5. Plot " $\log\left(\frac{S_u}{\sigma'_V}\right)$" vs "$\log(OCR)$".

6. Further data elimination using hydrometer data after observing a linear pattern between "$\log\left(\frac{S_u}{\sigma'_V}\right)$" and "log (OCR)". Only the data of the clay soils,

containing majority of clay sized particles (smaller than 0.002 mm) were selected.

7. Carry out Regression Analyses in order to obtain ‘m’ and ‘log(S)’.

8. Define the range of “*S*” using the lower and upper boundaries of the plot.

04. ANALYSES AND RESULTS

The SHANSEP equation gives a relationship between the normalized undrained shear strength and the overconsolidation ratio. The following data can be used to obtain the undrained shear strength of the clay soils.

I. UU triaxial shear tests
II. CIU triaxial shear tests
III. CK_0U triaxial shear tests
IV. Field vane shear tests
V. Laboratory vane shear tests

Most of the SHANSEP researchers have used CK_0U triaxial tests for the determination of SHANSEP parameters (Bay et al., 2005). However, CK0U triaxial shear test data are not available in Sri Lanka. Further, UU and CIU triaxial shear test data, estimates the undrained shear strength with a considerably large error (Ladd & DeGroot, 2003).

The procedure which follows by CK_0U triaxial test, follows an almost similar condition to the in-situ stresses (Nishimura, n.d.). Hence, it gives an accurate estimation of the undrained shear strength. Also, Field Vane Shear test gives an estimation of in-situ undrained shear strength with a good accuracy since the test is carried out in-situ. Further, Lacasse et al. (1978) and Ladd et al. (1983) have also used the results of the vane shear tests in their studies on the normalized undrained shear strength (Jamiolkowski et al., 1985).

Similarly, results of field vane shear tests have been used for the estimation of normalized undrained shear strength of soils in this research.

Appendix - A shows the summary of the raw data extracted from the laboratory reports and the calculated Effective Vertical Stress (σ_v'), Over Consolidation Ratio (OCR) and

Normalized Shear Strength (S_u/σ'). The bulk density values are assumed for the soil layers based on the past experience, in order to calculate the σ_V'. Since it can be observed that several OCR values are abnormally high, the data with OCR ≥ 20 were omitted (Appendix - B).

Then, the data of the fine-grained soils were identified using the method given by Unified Soil Classification System (USCS). The data of the soils, which consist of 50% or more of material passing No. 200 sieve, were taken as fine-grained soils. Appendix – C shows the fine contents of the soils. After that the clay soils were selected as per the plasticity chart of the USCS. Basically, the plots above A-Line of the plasticity chart were taken as the clay soils. The Appendix – D shows the plasticity data (Atterberg limits) of the clay soils. For the soils which do not consist of Atterberg limits data, the descriptions of the soils were used for the categorization.

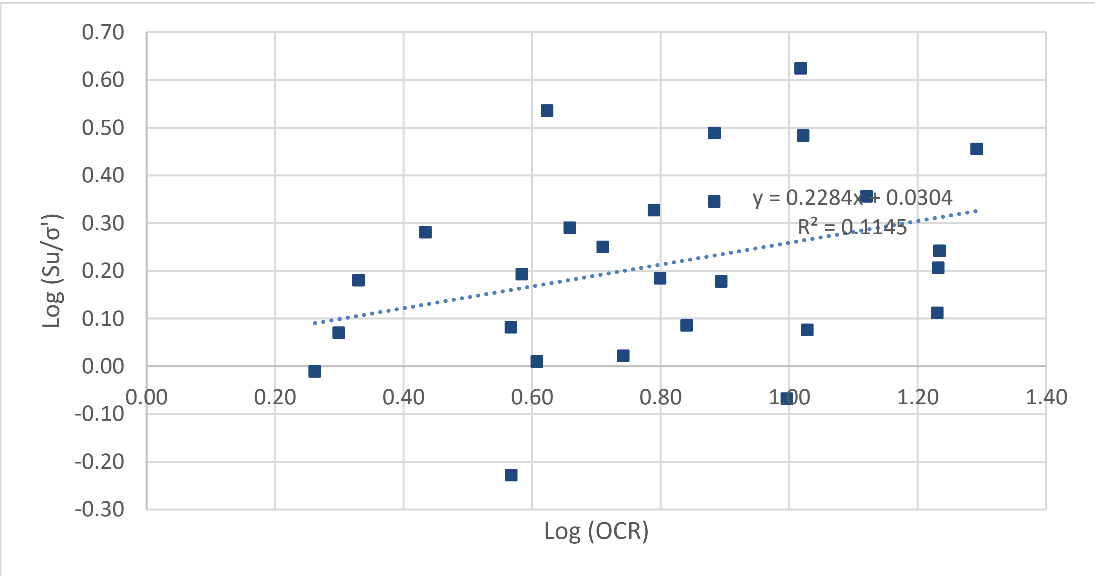

Figure 4.13: Log(Su/σv') vs Log(OCR)

As mentioned in the Step 5 of the Chapter 3: Methodology, " $\log\left(\frac{S_u}{\sigma'_V}\right)$" vs "$\log(OCR)$" were plotted (Figure 4.1). The linear regression of the above plot gives a R^2 of 0.1145, which is a very low value. However, a linear relationship can be observed even though the scatter is high.

After referring to more data of the soils, it could be observed in hydrometer analysis data, that the major constituent of the most of the clay soils is silt-sized particles (Refer Appendix – E for the hydrometer analysis data of the clay soils). Therefore, the data of the silt-sized soils were eliminated and " $\log\left(\frac{S_u}{\sigma'_V}\right)$" vs "$\log(OCR)$" were plotted as given in Figure 4.2.

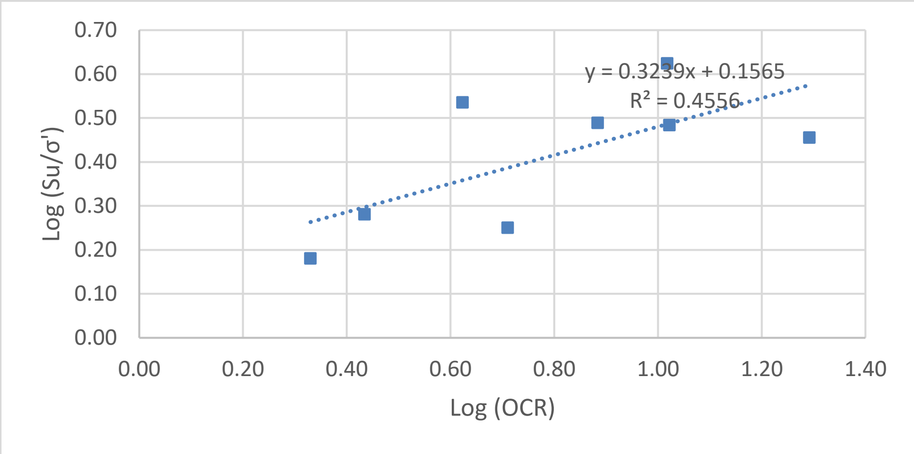

Figure 4.14: Log(S_u/σ_v') vs Log(OCR) only for the selected material

The linear regression of the clay-sized soils has given the R^2 of 0.4556. Finally, the analyses were carried out to determine the range of "*S*" as shown in Figure 4.3.

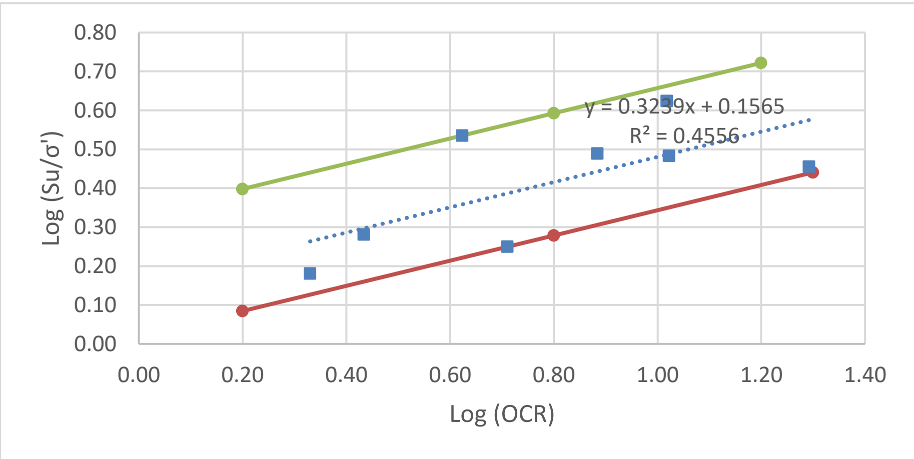

Figure 15: Range of "S"

It can be seen that using the lower boundary is always safe for estimating the undrained shear strength (S_u).

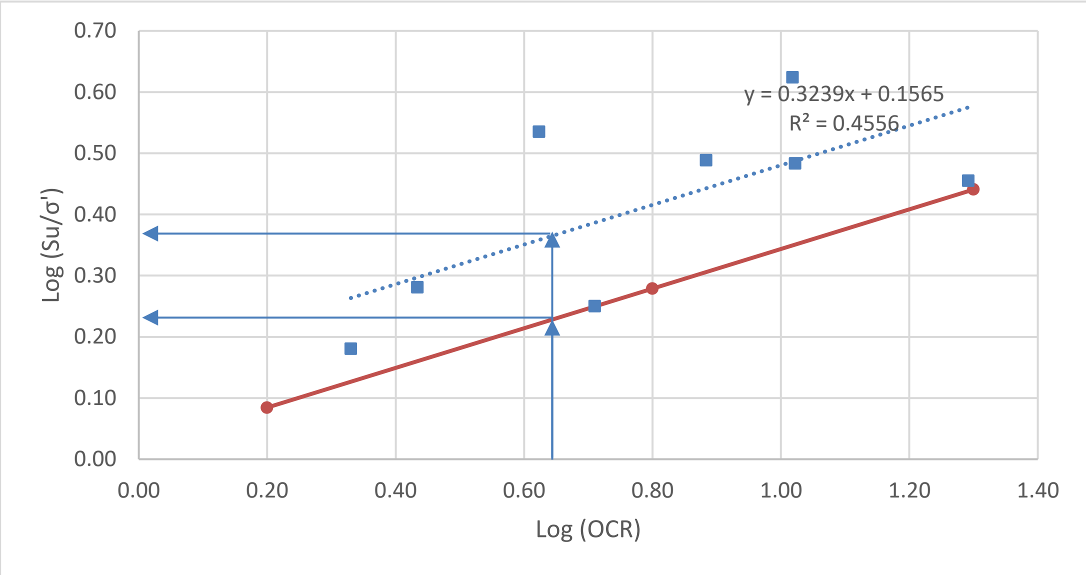

Figure 16: Lower Boundary of the Plot

As shown in Figure 4.4, the lower boundary always underestimates the normalized shear strength for a given overconsolidation ratio. However, the lower boundary must not be used for the estimation for the OCR using the normalized shear strength.

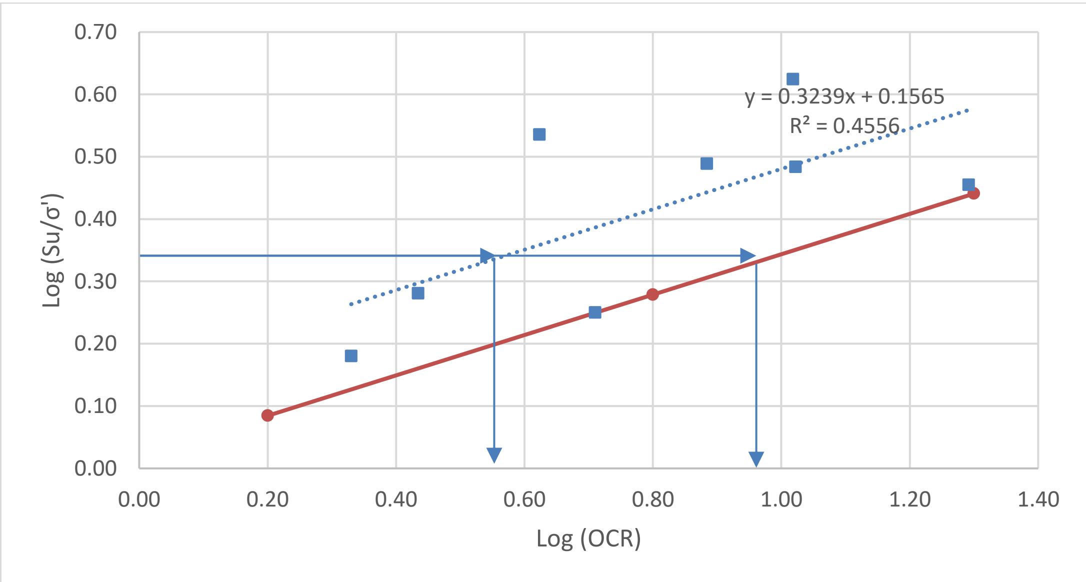

Figure 17: Lower Boundary for the estimation of OCR

As shown in Figure 4.5, the estimation of the OCR using the lower boundary, always over estimates the OCR value, which is unsafe for the geotechnical designs.

The upper boundary can be used for the estimation of OCR for a given normalized shear strength.

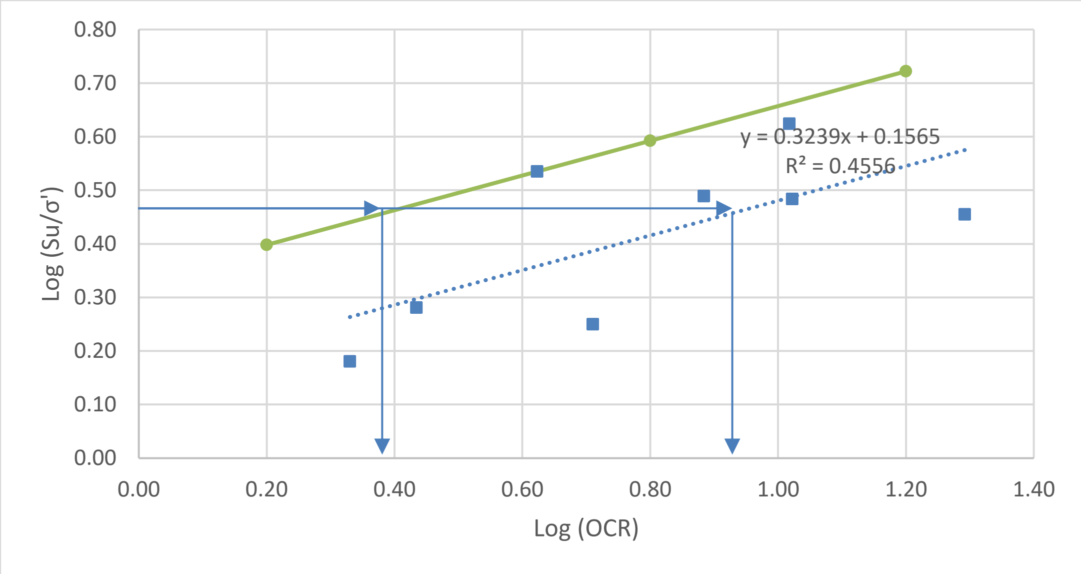

Figure 18: Upper Boundary for the Estimation of OCR

The upper boundary under estimates the OCR as shown in Figure 4.6. Therefore, the upper boundary is always safe for the estimation of the OCR for the given normalized shear strength. However, the upper boundary should not be used for the estimation of the undrained shear strength (S_u) since, the upper boundary over estimates the S_u.

4.1 Results of the Regression Analyses

The Figure 4.3 shows the linear regression line, lower boundary and upper boundary of the plot. The linear regression gives following data for ‘m’ and ‘S’.

$$m = 0.3239$$

$$\log(S) = 0.1565$$

Hence, $$S = 1.4338$$

$$R^2 = 0.46$$

The details of the lower boundary are shown below.

$$m = 0.3239$$

$$\log(S_L) = 0.01967$$

Hence, $$S_L = 1.0463$$

The details of the upper boundary are shown below.

$$m = 0.3239$$

$$\log(S_U) = 0.33322$$

Hence, $$S_U = 2.1539$$

05. DISCUSSION

The SHANSEP procedure has been introduced by Prof. Charles C. Ladd, after the Geotechnical engineers identified that the undrained shear strengths of many soils follow a characteristic pattern and the undrained shear strength can be normalized by the vertical overconsolidation pressure.

Several researchers have proposed relationships between normalized shear strength and the plasticity parameters before the introduction of SHANSEP (eg: Skempton 1957), Bjerrum and Simons (1960), Karlson and Viberg (1967)). However, all these relationships can only be used for normally consolidated clay soils.

Later, Ladd and Foott (1974) introduced SHANSEP procedure after identifying that the stress history and stress path have greater influence than the plasticity properties for the undrained shear strength of the soils. The SHANSEP is based on the following relationship.

$$\frac{S_u}{\sigma'_V} = S\ x\ (OCR)^m$$

This equation shows the relationship between the Normalized Shear Strength (normalized by the vertical overconsolidation pressure) and the Overconsolidation ratio. The SHANSEP equation can be used to estimate the undrained shear strength using the overconsolidation ratio or estimate the settlement by determining overconsolidation ratio using undrained shear strength.

Most of the times, the CK_0U triaxial shear tests have been used to determine the undrained shear strength for the researches on SHANSEP. The CK_0U triaxial shear test provides the undrained shear strength of the soil with a good accuracy to the in-situ condition than the conventional CIU triaxial shear tests. Similarly, the field vane shear test estimates the in-situ undrained shear strength with a good accuracy. Therefore, the field vane shear test has been used for the evaluation of SHANSEP parameters in this research similar to the researches carried out by Lacasse et al. (1978) and Ladd et al. (1983) (Jamiolkowski et al., 1985).

In previous researches, the SHANSEP parameters have been obtained for the Clays. Bay et al., (2005) evaluated the SHANSEP parameters for Soft Bonneville Clays. Lacasse has analyzed the $\frac{S_u}{\sigma'_{v0}}$ vs OCR for following clays (Jamiolkowski et al., 1985).

- Boston Blue Clay
- Connecticut Valley Varved Clay
- Organic Clay with shells, Fore river
- James Bay B-2 and B-6 Marine Clays

The purpose of this research is to evaluate the SHANSEP parameters for the alluvial clay soils in Sri Lanka. The test data from major projects were used in this research. However, no data were available for pure clays similar to the previous researches on SHANSEP. The following relationship was obtained after the analysis with a R^2 of 0.46.

$$\frac{S_u}{\sigma'_V} = 1.4338\ x\ (OCR)^{0.32}$$

The 'S' varies in between upper and lower bounds of 1.0463 and 2.1539 respectively. However, the exponent 'm' should usually fall between 0.75 – 1.00 (Bay et al., 2005). The studies done by Bay et al. (2005) has given the following formula for the soft Bonneville Clay.

$$\frac{S_u}{\sigma'_V} = 0.32\ x\ (OCR)^{0.82}$$

Therefore, it can be seen that there is a considerable deviation of 'm' from the given range in literature. Further, 'S' can be estimated using the equation proposed by Skempton (1957). Since the average plasticity index of the clay soils in this research is 27, the 'S' can be calculated as;

$$(\frac{S_u}{\sigma'_{V0}})_{nc} = S = 0.11 + 0.0037 I_p$$

$$S = 0.11 + (0.0037\ x\ 27)$$

$$S = 0.21$$

Therefore, it can be observed that the 'S' in this research has a considerably large deviation from the estimated 'S' using the equation proposed by Skempton (1957). Also, the R^2 of 0.46 is somewhat low.

The following could be the possible reasons for the deviations of the constants 'm' and 'S'.

- The behaviour and the properties of the Sri Lankan alluvial cohesive soils are different from the soils in previous researches.
- Field vane shear test results were used in this research, but the (Jamiolkowski et al., 1985) observed that 'm' obtained using field vane shear tests is usually higher than that of the CK_0U triaxial shear test.

The following could be the reasons for the low R^2 of the analyses.

- The soils used in the final analysis are not uniform
- The soils are not of low activity and sensitivity
- Disturbances to the samples taken for the oedometer test while extraction and sample preparation.

06. RECOMMENDATIONS

Though the R^2 is somewhat low in this analysis, a clear linear relationship between normalized shear strength and the OCR can be observed. Therefore, it is recommended to extend this research for more uniform alluvial clay soils of low activity and sensitivity. The major constituents of the soils should be clay-sized particles. Further, while using estimated S_u from this relationship, a suitable correction factor should be used before applying it for the geotechnical designs. This is due to fact that this research is based on the field vane shear test data, and the S_u values obtained from the field vane shear tests should be corrected before use them for the geotechnical designs (*ASTM D2573-01*, 2001).

As mentioned in the discussion, it is much safer use the lower boundary for the estimation of the undrained shear strength. The following equation gives the lower boundary of the plot.

$$\frac{S_u}{\sigma'_V} = 1.0463\ x\ (OCR)^{0.32}$$

Similarly, the following equation is safer for the estimation of the OCR for a given normalized S_u.

$$\frac{S_u}{\sigma'_V} = 2.1539\ x\ (OCR)^{0.32}$$

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Appendix – A: Summary of Raw Data Including Calculated OCR and S_u/σ_v'

Table 1: Raw Data

Location	Vane Shear			Consolidation		GWL	σ'	OCR	S_u/σ'
	Depth (m)	Visual Description	Peak Shear Strength (kN/m^2)	Depth (m)	P_c (kPa)				
8500L	1.00	Silty CLAY	28.0	0.50 - 1.00	83.5	0.50	8.80	9.5	3.1827
16800L	3.00	Firm clayey SILT	6.6	3.00 - 3.50	136.0	0.00	21.02	6.5	0.3140
6796 N-R	1.25	Soft sandy CLAY	6.0	0.50 - 1.00	79.0	0.00	4.64	17.0	1.2924
6640 N-C	1.00	loose silty SAND with some clay	24.4	1.00 - 1.50	94.0	0.90	18.39	5.1	1.3267
23034 N-R	1.00	firm silty CLAY with sand	49.5	1.00 - 1.50	123.0	0.90	16.07	7.7	3.0809
	1.75		65.7	1.50 - 2.00	80.5	0.90	19.16	4.2	3.4288
23074 N-R	1.40	firm clayey SILT with sand	51.3	1.00 - 1.50	63.5	1.50	20.40	3.1	2.5147
	1.90	soft silty CLAY	39.3	1.50 - 2.00	55.5	1.50	25.95	2.1	1.5146
23636 N-L	0.70	stiff clayey SILT with sand	33.5	0.50 - 1.00	83.0	0.30	7.96	10.4	4.2083
23656 N-R	0.50	firm sandy SILT with traces of clay & organic debris	50.6	0.50 - 1.00	55.5	0.80	12.00	4.6	4.2167
	1.00	firm silty CLAY with sand	27.7	1.00 - 1.50	80.0	0.80	15.59	5.1	1.7773
23797 N-L	1.10	firm sandy SILT with clay	26.8	1.00 - 1.50	66.0	0.90	17.19	3.8	1.5589
23817 N-R	1.00	firm sandy SILT with clay	28.9	1.00 - 1.50	84.0	0.50	13.62	6.2	2.1223
24214 N-L	0.50	Soft sandy SILT with clay	44.4	0.50 - 1.00	58.0	0.20	6.23	9.3	7.1274

24214 N-R	1.00	Soft sandy SILT with clay	19.9	1.00 - 1.50	82.0	0.50	13.02	6.3	1.5287
24274 N-L	0.50	Loose silty SAND with traces of clay	60.6	0.50 - 1.00	67.0	0.70	11.88	5.6	5.0991
24838 N-R	0.50	Soft sandy SILT with some clay	32.4	0.50 - 1.00	102.0	0.50	9.90	10.3	3.2736
24945 N-C	1.00	Firm sandy CLAY with silt	22.7	1.00 - 1.50	156.0	0.00	7.96	19.6	2.8509
12413 N-C	1.00	Loose silty GRAVEL with sand & some clay	30.3	1.00 - 1.50	52.0	1.10	19.38	2.7	1.5636
26426 N-C	1.10	Stiff sandy SILT with clay	44.4	1.00 - 1.50	87.0	1.50	20.63	4.2	2.1527
	1.70	Stiff sandy SILT with clay	27.0	1.50 - 2.00	107.0	1.50	26.42	4.0	1.0219
	2.20	Stiff sandy SILT with clay	17.6	2.00 - 2.50	110.0	1.50	29.77	3.7	0.5912
26868 N-C	1.00	Stiff sandy SILT with clay	16.0	1.00 - 1.50	49.0	0.50	13.27	3.7	1.2060
21495 N-C	0.50	Soft sandy SILT with clay	28.2	1.5	47.8	1.70	24.00	2.0	1.1750
21534 N-R	2.00	Very soft clayey SILT with sand	22.4	1.50 - 2.00	42.0	1.10	22.97	1.8	0.9751
3624 N-L	0.50	Very loose clayey SAND	22.9	0.50 - 1.00	61.5	0.18	6.81	9.0	3.3635
3914 N-L	1.00	Loose clayey SAND	30.3	0.50 - 1.00	103.5	0.15	6.49	16.0	4.6694
3944 N-L	0.50	Very loose silty SAND with clay & traces of gravel	21.9	0.50 - 1.00	72.5	0.10	5.50	13.2	3.9829
3944 N-R	0.50	Loose silty SAND with clay	11.8	0.50 - 1.00	56.0	0.10	6.00	9.3	1.9672
4995 N-R	1.50	Very loose clayey SAND	29.1	2.00 - 2.50	147.0	0.50	19.96	7.4	1.4581
5319 N-R	0.50	Soft sandy SILT with traces of clay	17.6	0.50 - 1.00	41.5	0.45	8.68	4.8	2.0272
	1.00	Firm sandy lean CLAY	25.9	1.00 - 1.50	89.5	0.45	11.70	7.6	2.2133

5339 N-R	0.50	Firm sandy lean CLAY	10.2	0.50 - 1.00	91.5	0.40	8.57	10.7	1.1907
26740 N-L	1.00	Loose clayey SAND	13.9	1.00 - 1.50	105.5	0.80	16.21	6.5	0.8575
26740 N-R	1.00	Loose clayey SAND	21.2	1.00 - 1.50	92.5	0.96	17.78	5.2	1.1923
	2.00	Firm sandy fat CLAY	46.0	2.00 - 2.50	65.5	0.96	24.10	2.7	1.9091
31881 N-R	1.00	Medium dense clayey SAND	14.3	1.00 - 1.50	93.0	1.36	21.18	4.4	0.6753
31881 N-L	0.50	Soft sandy SILT with clay	25.9	0.50 - 1.00	106.0	0.35	7.70	13.8	3.3632
	1.00	Firm fat CLAY	32.1	1.00 - 1.50	111.0	0.35	10.55	10.5	3.0438
32187 N-L	0.50	Soft sandy lean CLAY	12.0	0.50 - 1.00	166.0	0.00	3.89	42.6	3.0829
	1.00	Firm fat CLAY with sand	9.2	1.00 - 1.50	150.0	0.00	6.66	22.5	1.3809
	1.50		16.6	1.50 - 2.00	163.0	0.00	9.51	17.1	1.7460
32187 N-R	0.50	Soft sandy fat CLAY	7.2	0.50 - 1.00	191.5	0.00	3.89	49.2	1.8497
	1.00	Firm fat CLAY with sand	7.9	1.00 - 1.50	185.5	0.00	6.71	27.6	1.1769
16220 N-C	1.00	Soft sandy fat CLAY	21.7	1.00 - 1.50	114.0	1.20	20.63	5.5	1.0516
16320 N-C	1.00	Firm clayey SILT with sand and traces of clay	21.7	1.00 - 1.50	241.0	1.50	21.25	11.3	1.0212
16398 N-C	1.00	Soft elastic SILT	18.7	1.00 - 1.50	218.0	0.00	8.44	25.8	2.2163
19264 N-L	0.50	Very loose clayey SAND	14.1	0.50 - 1.00	115.5	0.00	5.77	20.0	2.4447
	1.00	Clayey SAND	8.1	1.00 - 1.50	175.0	0.00	9.61	18.2	0.8427
	2.00	Clayey SAND	39.3	1.50 - 2.00	66.0	0.00	13.53	4.9	2.9041
19264 N-R	0.50	Soft sandy SILT with CLAY	26.6	0.50 - 1.00	104.3	0.00	5.39	19.3	4.9328

19604 N-R	1.00	Loose clayey SAND	11.8	1.00 - 1.50	88.0	3.00	21.88	4.0	0.5394
32375 N-L	1.00	Soft lean CLAY with sand	20.1	1.00 - 1.50	117.0	0.00	8.86	13.2	2.2680
	2.00	Firm fat CLAY with sand	13.4	2.00 - 2.50	155.5	0.00	15.68	9.9	0.8547
32375 N-R	1.00	Firm lean CLAY with sand	21.7	1.00 - 1.50	123.5	1.00	17.82	6.9	1.2176
	2.00	Firm lean CLAY with sand	41.3	1.50 - 2.00	96.5	1.00	21.17	4.6	1.9511
32425 N-L	1.00	Very soft sandy lean CLAY	16.6	1.00 - 1.50	86.5	0.30	11.03	7.8	1.5049
32425 N-R	0.50	Very soft sandy lean CLAY	12.2	0.50 - 1.00	129.5	0.30	7.59	17.1	1.6083
	1.00	Very loose clayey SAND	21.5	1.50 - 2.00	157.0	0.30	14.63	10.7	1.4700

Appendix – B: Data after Deleting OCR ≥ 20

Table 2: Raw Data after Omitting High OCR values

Location	Vane Shear			Consolidation		GWL	σ'	OCR	S_u/σ'
	Depth (m)	Visual Description	Peak Shear Strength (kN/m^2)	Depth (m)	P_c (kPa)				
8500L	1.00	Silty Clay	28.0	0.50 - 1.00	83.5	0.50	8.80	9.5	3.1827
16800L	3.00	Firm clayey SILT	6.6	3.00 - 3.50	136.0	0.00	21.02	6.5	0.3140
6796 N-R	1.25	Soft sandy CLAY	6.0	0.50 - 1.00	79.0	0.00	4.64	17.0	1.2924
6640 N-C	1.00	Loose silty SAND with some clay	24.4	1.00 - 1.50	94.0	0.90	18.39	5.1	1.3267
23034 N-R	1.00	firm silty CLAY with sand	49.5	1.00 - 1.50	123.0	0.90	16.07	7.7	3.0809
	1.75		65.7	1.50 - 2.00	80.5	0.90	19.16	4.2	3.4288
23074 N-R	1.40	firm clayey SILT with sand	51.3	1.00 - 1.50	63.5	1.50	20.40	3.1	2.5147
	1.90	soft silty CLAY	39.3	1.50 - 2.00	55.5	1.50	25.95	2.1	1.5146
23636 N-L	0.70	stiff clayey SILT with sand	33.5	0.50 - 1.00	83.0	0.30	7.96	10.4	4.2083
23656 N-R	0.50	firm sandy SILT with traces of clay & organic debris	50.6	0.50 - 1.00	55.5	0.80	12.00	4.6	4.2167
	1.00	firm silty CLAY with sand	27.7	1.00 - 1.50	80.0	0.80	15.59	5.1	1.7773
23797 N-L	1.10	firm sandy SILT with clay	26.8	1.00 - 1.50	66.0	0.90	17.19	3.8	1.5589
23817 N-R	1.00	firm sandy SILT with clay	28.9	1.00 - 1.50	84.0	0.50	13.62	6.2	2.1223
24214 N-L	0.50	Soft sandy SILT with clay	44.4	0.50 - 1.00	58.0	0.20	6.23	9.3	7.1274
24214 N-R	1.00	Soft sandy SILT with clay	19.9	1.00 - 1.50	82.0	0.50	13.02	6.3	1.5287

24274 N-L	0.50	Loose silty SAND with traces of clay	60.6	0.50 - 1.00	67.0	0.70	11.88	5.6	5.0991
24838 N-R	0.50	Soft sandy SILT with some clay	32.4	0.50 - 1.00	102.0	0.50	9.90	10.3	3.2736
24945 N-C	1.00	Firm sandy CLAY with silt	22.7	1.00 - 1.50	156.0	0.00	7.96	19.6	2.8509
12413 N-C	1.00	Loose silty GRAVEL with sand & some clay	30.3	1.00 - 1.50	52.0	1.10	19.38	2.7	1.5636
26426 N-C	1.10	Stiff sandy SILT with clay	44.4	1.00 - 1.50	87.0	1.50	20.63	4.2	2.1527
	1.70	Stiff sandy SILT with clay	27.0	1.50 - 2.00	107.0	1.50	26.42	4.0	1.0219
	2.20	Stiff sandy SILT with clay	17.6	2.00 - 2.50	110.0	1.50	29.77	3.7	0.5912
26868 N-C	1.00	Stiff sandy SILT with clay	16.0	1.00 - 1.50	49.0	0.50	13.27	3.7	1.2060
21495 N-C	0.50	Soft sandy SILT with clay	28.2	1.5	47.8	1.70	24.00	2.0	1.1750
21534 N-R	2.00	Very soft clayey SILT with sand	22.4	1.50 - 2.00	42.0	1.10	22.97	1.8	0.9751
3624 N-L	0.50	Very loose clayey SAND	22.9	0.50 - 1.00	61.5	0.18	6.81	9.0	3.3635
3914 N-L	1.00	Loose clayey SAND	30.3	0.50 - 1.00	103.5	0.15	6.49	16.0	4.6694
3944 N-L	0.50	Very loose silty SAND with clay & traces of gravel	21.9	0.50 - 1.00	72.5	0.10	5.50	13.2	3.9829
3944 N-R	0.50	Loose silty SAND with clay	11.8	0.50 - 1.00	56.0	0.10	6.00	9.3	1.9672
4995 N-R	1.50	Very loose clayey SAND	29.1	2.00 - 2.50	147.0	0.50	19.96	7.4	1.4581
5319 N-R	0.50	Soft sandy SILT with traces of clay	17.6	0.50 - 1.00	41.5	0.45	8.68	4.8	2.0272
	1.00	Firm sandy lean CLAY	25.9	1.00 - 1.50	89.5	0.45	11.70	7.6	2.2133

5339 N-R	0.50	Firm sandy lean CLAY	10.2	0.50 - 1.00	91.5	0.40	8.57	10.7	1.1907
26740 N-L	1.00	Loose clayey SAND	13.9	1.00 - 1.50	105.5	0.80	16.21	6.5	0.8575
26740 N-R	1.00	Loose clayey SAND	21.2	1.00 - 1.50	92.5	0.96	17.78	5.2	1.1923
	2.00	Firm sandy fat CLAY	46.0	2.00 - 2.50	65.5	0.96	24.10	2.7	1.9091
31881 N-R	1.00	Medium dense clayey SAND	14.3	1.00 - 1.50	93.0	1.36	21.18	4.4	0.6753
31881 N-L	0.50	Soft sandy SILT with clay	25.9	0.50 - 1.00	106.0	0.35	7.70	13.8	3.3632
	1.00	Firm fat CLAY	32.1	1.00 - 1.50	111.0	0.35	10.55	10.5	3.0438
32187 N-L	1.50	Firm fat CLAY with sand	16.6	1.50 - 2.00	163.0	0.00	9.51	17.1	1.7460
16220 N-C	1.00	Soft sandy fat CLAY	21.7	1.00 - 1.50	114.0	1.20	20.63	5.5	1.0516
16320 N-C	1.00	Firm clayey SILT with sand and traces of clay	21.7	1.00 - 1.50	241.0	1.50	21.25	11.3	1.0212
19264 N-L	1.00	Clayey SAND	8.1	1.00 - 1.50	175.0	0.00	9.61	18.2	0.8427
	2.00	Clayey SAND	39.3	1.50 - 2.00	66.0	0.00	13.53	4.9	2.9041
19264 N-R	0.50	Soft sandy SILT with CLAY	26.6	0.50 - 1.00	104.3	0.00	5.39	19.3	4.9328
19604 N-R	1.00	Loose clayey SAND	11.8	1.00 - 1.50	88.0	3.00	21.88	4.0	0.5394
32375 N-L	1.00	Soft lean CLAY with sand	20.1	1.00 - 1.50	117.0	0.00	8.86	13.2	2.2680
	2.00	Firm fat CLAY with sand	13.4	2.00 - 2.50	155.5	0.00	15.68	9.9	0.8547
32375 N-R	1.00	Firm lean CLAY with sand	21.7	1.00 - 1.50	123.5	1.00	17.82	6.9	1.2176
	2.00	Firm lean CLAY with sand	41.3	1.50 - 2.00	96.5	1.00	21.17	4.6	1.9511
32425 N-L	1.00	Very soft sandy lean CLAY	16.6	1.00 - 1.50	86.5	0.30	11.03	7.8	1.5049

32425 N-R	0.50	Very soft sandy lean CLAY	12.2	0.50 - 1.00	129.5	0.30	7.59	17.1	1.6083
	1.00	Very loose clayey SAND	21.5	1.50 - 2.00	157.0	0.30	14.63	10.7	1.4700

Appendix – C: Data with Fine Content

Table 3: Data including Fines Content

Location	Vane Shear			Consolidation		σ'	OCR	S_u/σ'	Fine Content
	Depth (m)	Description	Peak Shear Strength (kN/m^2)	Depth (m)	P_c (kPa)				
8500L	1.00	Silty Clay	28.0	0.50 - 1.00	83.5	8.80	9.5	3.1827	47%
16800L	3.00	Firm clayey SILT	6.6	3.00 - 3.50	136.0	21.02	6.5	0.3140	96%
6796 N-R	1.25	Soft sandy CLAY	6.0	0.50 - 1.00	79.0	4.64	17.0	1.2924	81%
6640 N-C	1.00	loose silty SAND with some clay	24.4	1.00 - 1.50	94.0	18.39	5.1	1.3267	41%
23034 N-R	1.00	firm silty CLAY with sand	49.5	1.00 - 1.50	123.0	16.07	7.7	3.0809	82%
	1.75		65.7	1.50 - 2.00	80.5	19.16	4.2	3.4288	71%
23074 N-R	1.40	firm clayey SILT with sand	51.3	1.00 - 1.50	63.5	20.40	3.1	2.5147	72%
	1.90	soft silty CLAY	39.3	1.50 - 2.00	55.5	25.95	2.1	1.5146	96%
23636 N-L	0.70	stiff clayey SILT with sand	33.5	0.50 - 1.00	83.0	7.96	10.4	4.2083	75%
23656 N-R	0.50	firm sandy SILT with traces of clay & organic debris	50.6	0.50 - 1.00	55.5	12.00	4.6	4.2167	71%
	1.00	firm silty CLAY with sand	27.7	1.00 - 1.50	80.0	15.59	5.1	1.7773	77%
23797 N-L	1.10	firm sandy SILT with clay	26.8	1.00 - 1.50	66.0	17.19	3.8	1.5589	56%
23817 N-R	1.00	firm sandy SILT with clay	28.9	1.00 - 1.50	84.0	13.62	6.2	2.1223	54%
24214 N-L	0.50	Soft sandy SILT with clay	44.4	0.50 - 1.00	58.0	6.23	9.3	7.1274	63%
24214 N-R	1.00	Soft sandy SILT with clay	19.9	1.00 - 1.50	82.0	13.02	6.3	1.5287	59%

24274 N-L	0.50	Loose silty SAND with traces of clay	60.6	0.50 - 1.00	67.0	11.88	5.6	5.0991	39%
24838 N-R	0.50	Soft sandy SILT with some clay	32.4	0.50 - 1.00	102.0	9.90	10.3	3.2736	53%
24945 N-C	1.00	Firm sandy CLAY with silt	22.7	1.00 - 1.50	156.0	7.96	19.6	2.8509	66%
12413 N-C	1.00	Loose silty GRAVEL with sand & some clay	30.3	1.00 - 1.50	52.0	19.38	2.7	1.5636	40%
26426 N-C	1.10	Stiff sandy SILT with clay	44.4	1.00 - 1.50	87.0	20.63	4.2	2.1527	67%
	1.70	Stiff sandy SILT with clay	27.0	1.50 - 2.00	107.0	26.42	4.0	1.0219	57%
	2.20	Stiff sandy SILT with clay	17.6	2.00 - 2.50	110.0	29.77	3.7	0.5912	57%
26868 N-C	1.00	Stiff sandy SILT with clay	16.0	1.00 - 1.50	49.0	13.27	3.7	1.2060	70%
21495 N-C	0.50	Soft sandy SILT with clay	28.2	1.5	47.8	24.00	2.0	1.1750	69%
21534 N-R	2.00	Very soft clayey SILT with sand	22.4	1.50 - 2.00	42.0	22.97	1.8	0.9751	77%
3624 N-L	0.50	Very loose clayey SAND	22.9	0.50 - 1.00	61.5	6.81	9.0	3.3635	34%
3914 N-L	1.00	Loose clayey SAND	30.3	0.50 - 1.00	103.5	6.49	16.0	4.6694	26%
3944 N-L	0.50	Very loose silty SAND with clay & traces of gravel	21.9	0.50 - 1.00	72.5	5.50	13.2	3.9829	43%
3944 N-R	0.50	Loose silty SAND with clay	11.8	0.50 - 1.00	56.0	6.00	9.3	1.9672	46%
4995 N-R	1.50	Very loose clayey SAND	29.1	2.00 - 2.50	147.0	19.96	7.4	1.4581	45%

5319 N-R	0.50	Soft sandy SILT with traces of clay	17.6	0.50 - 1.00	41.5	8.68	4.8	2.0272	56%
	1.00	Firm sandy lean CLAY	25.9	1.00 - 1.50	89.5	11.70	7.6	2.2133	60%
5339 N-R	0.50	Firm sandy lean CLAY	10.2	0.50 - 1.00	91.5	8.57	10.7	1.1907	55%
26740 N-L	1.00	Loose clayey SAND	13.9	1.00 - 1.50	105.5	16.21	6.5	0.8575	43%
26740 N-R	1.00	Loose clayey SAND	21.2	1.00 - 1.50	92.5	17.78	5.2	1.1923	49%
	2.00	Firm sandy fat CLAY	46.0	2.00 - 2.50	65.5	24.10	2.7	1.9091	61%
31881 N-R	1.00	Medium dense clayey SAND	14.3	1.00 - 1.50	93.0	21.18	4.4	0.6753	40%
31881 N-L	0.50	Soft sandy SILT with clay	25.9	0.50 - 1.00	106.0	7.70	13.8	3.3632	57%
	1.00	Firm fat CLAY	32.1	1.00 - 1.50	111.0	10.55	10.5	3.0438	87%
32187 N-L	1.50	Firm fat CLAY with sand	16.6	1.50 - 2.00	163.0	9.51	17.1	1.7460	77%
16220 N-C	1.00	Soft sandy fat CLAY	21.7	1.00 - 1.50	114.0	20.63	5.5	1.0516	64%
16320 N-C	1.00	Firm clayey SILT with sand and traces of clay	21.7	1.00 - 1.50	241.0	21.25	11.3	1.0212	74%
19264 N-L	1.00	Clayey SAND	8.1	1.00 - 1.50	175.0	9.61	18.2	0.8427	45%
	2.00	Clayey SAND	39.3	1.50 - 2.00	66.0	13.53	4.9	2.9041	46%
19264 N-R	0.50	Soft sandy SILT with CLAY	26.6	0.50 - 1.00	104.3	5.39	19.3	4.9328	58%
19604 N-R	1.00	Loose clayey SAND	11.8	1.00 - 1.50	88.0	21.88	4.0	0.5394	30%
32375 N-L	1.00	Soft lean CLAY with sand	20.1	1.00 - 1.50	117.0	8.86	13.2	2.2680	72%
	2.00	Firm fat CLAY with sand	13.4	2.00 - 2.50	155.5	15.68	9.9	0.8547	83%
32375 N-R	1.00	Firm lean CLAY with sand	21.7	1.00 - 1.50	123.5	17.82	6.9	1.2176	84%

	2.00	Firm lean CLAY with sand	41.3	1.50 - 2.00	96.5	21.17	4.6	1.9511	61%
32425 N-L	1.00	Very soft sandy lean CLAY	16.6	1.00 - 1.50	86.5	11.03	7.8	1.5049	67%
32425 N-R	0.50	Very soft sandy lean CLAY	12.2	0.50 - 1.00	129.5	7.59	17.1	1.6083	62%
	1.00	Very loose clayey SAND	21.5	1.50 - 2.00	157.0	14.63	10.7	1.4700	44%

Note:

1. The soil descriptions shown herein are only visual descriptions given in the respective borehole logs and, do not give the correct classification according to the USCS.

Appendix – D: Data of Clay Soils

Table 4: Data of Clay Soils

Location	Vane Shear		Log (OCR)	Log (S_u/σ')	Fines Content	Atterberg Limits		A Line Value
	Depth (m)	Description				LL (%)	PI (%)	
6796 N-R	1.25	Soft sandy CLAY	1.23	0.11	81%	49	28	21.2
23034 N-R	1.00	firm silty CLAY with sand	0.88	0.49	82%	57	27	27.0
	1.75		0.62	0.54	71%	55	28	25.6
23074 N-R	1.90	soft silty CLAY	0.33	0.18	96%	57	29	27.0
23636 N-L	0.70	stiff clayey SILT with sand	1.02	0.62	75%	54	26	24.8
23656 N-R	1.00	firm silty CLAY with sand	0.71	0.25	77%	50	30	21.9
23797 N-L	1.10	firm sandy SILT with clay	0.58	0.19	56%	43	23	16.8
23817 N-R	1.00	firm sandy SILT with clay	0.79	0.33	54%	46	27	19.0
24214 N-R	1.00	Soft sandy SILT with clay	0.80	0.18	59%	42	21	16.1
24945 N-C	1.00	Firm sandy CLAY with silt	1.29	0.45	66%	50	34	21.9
26426 N-C	1.70	Stiff sandy SILT with clay	0.61	0.01	57%	41	19	15.3
	2.20	Stiff sandy SILT with clay	0.57	-0.23	57%	41	19	15.3
26868 N-C	1.00	Stiff sandy SILT with clay	0.57	0.08	70%	55	27	25.6
21495 N-C	0.50	Soft sandy SILT with clay	0.30	0.07	69%	45	25	18.3
21534 N-R	2.00	Very soft clayey SILT with sand	0.26	-0.01	77%	70	41	36.5
5319 N-R	1.00	Firm sandy lean CLAY	0.88	0.35	60%	40	27	14.6

5339 N-R	0.50	Firm sandy lean CLAY	1.03	0.08	55%	35	16	11.0
26740 N-R	2.00	Firm sandy fat CLAY	0.43	0.28	61%	63	35	31.4
31881 N-L	1.00	Firm fat CLAY	1.02	0.48	87%	61	35	29.9
32187 N-L	1.50	Firm fat CLAY with sand	1.23	0.24	77%	55	30	25.6
16220 N-C	1.00	Soft sandy fat CLAY	0.74	0.02	64%	56	30	26.3
32375 N-L	1.00	Soft lean CLAY with sand	1.12	0.36	72%	47	24	19.7
	2.00	Firm fat CLAY with sand	1.00	-0.07	83%	57	32	27.0
32375 N-R	1.00	Firm lean CLAY with sand	0.84	0.09	84%	40	23	14.6
	2.00	Firm lean CLAY with sand	0.66	0.29	61%	Not Available		-
32425 N-L	1.00	Very soft sandy lean CLAY	0.89	0.18	67%	49	25	21.2
32425 N-R	0.50	Very soft sandy lean CLAY	1.23	0.21	62%	35	17	11.0

Note:

1. The soil descriptions shown herein are only visual descriptions given in the respective borehole logs and, do not give the correct classification according to the USCS.

Appendix – E: Hydrometer Data of Clay Soils

Table 5: Hydrometer Data of Clay Soils

Location	Depth (m)	Description	Log (OCR)	Log (S_u/σ')	Fines Content	Clay Size (%)	Silt Size (%)
6796 N-R	1.25	Soft sandy CLAY	1.23	0.11	81%	5%	76%
23034 N-R	1.00	firm silty CLAY with sand	0.88	0.49	82%	46%	36%
	1.75		0.62	0.54	71%	39%	32%
23074 N-R	1.90	soft silty CLAY	0.33	0.18	96%	54%	42%
23636 N-L	0.70	stiff clayey SILT with sand	1.02	0.62	75%	38%	37%
23656 N-R	1.00	firm silty CLAY with sand	0.71	0.25	77%	42%	35%
23797 N-L	1.10	firm sandy SILT with clay	0.58	0.19	56%	23%	33%
23817 N-R	1.00	firm sandy SILT with clay	0.79	0.33	54%	24%	30%
24214 N-R	1.00	Soft sandy SILT with clay	0.80	0.18	59%	24%	35%
24945 N-C	1.00	Firm sandy CLAY with silt	1.29	0.45	66%	37%	29%
26426 N-C	1.70	Stiff sandy SILT with clay	0.61	0.01	57%	17%	40%
	2.20	Stiff sandy SILT with clay	0.57	-0.23	57%	17%	40%
26868 N-C	1.00	Stiff sandy SILT with clay	0.57	0.08	70%	27%	43%
21495 N-C	0.50	Soft sandy SILT with clay	0.30	0.07	69%	15%	54%

21534 N-R	2.00	Very soft clayey SILT with sand	0.26	-0.01	77%	27%	50%
5319 N-R	1.00	Firm sandy lean CLAY	0.88	0.35	60%	24%	36%
5339 N-R	0.50	Firm sandy lean CLAY	1.03	0.08	55%	16%	39%
26740 N-R	2.00	Firm sandy fat CLAY	0.43	0.28	61%	32%	29%
31881 N-L	1.00	Firm fat CLAY	1.02	0.48	87%	46%	41%
32187 N-L	1.50	Firm fat CLAY with sand	1.23	0.24	77%	33%	44%
16220 N-C	1.00	Soft sandy fat CLAY	0.74	0.02	64%	31%	33%
32375 N-L	1.00	Soft lean CLAY with sand	1.12	0.36	72%	31%	41%
	2.00	Firm fat CLAY with sand	1.00	-0.07	83%	41%	42%
32375 N-R	1.00	Firm lean CLAY with sand	0.84	0.09	84%	41%	43%
	2.00	Firm lean CLAY with sand	0.66	0.29	61%	29%	32%
32425 N-L	1.00	Very soft sandy lean CLAY	0.89	0.18	67%	28%	39%
32425 N-R	0.50	Very soft sandy lean CLAY	1.23	0.21	62%	24%	38%

Note:

1. The soil descriptions shown herein are only visual descriptions given in the respective borehole logs and, do not give the correct classification according to the USCS.