VERIFICATION OF CPTU SOIL TYPES BY USCS

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ABSTRAK

Pengelasan tanah adalah sangat penting dalam kejuruteraan geoteknik. Pengelasan tanah membolehkan jurutera geoteknikal di seluruh dunia untuk menyampaikan ciri-ciri umum tanah lebih berkesan. Pengelasan tanah di tapak diberikan oleh kaedah CPTU tanpa perlu membawa sampel ke makmal. Keputusan yang diperolehi daripada kaedah CPTU kemudiannya dikaitkan dengan klasifikasi yang sedia ada tekstur berdasarkan yang Sistem Pengelasan Tanah Bersatu (USCS). CPT berasaskan SBT mungkin tidak sentiasa bersetuju dengan jenis tanah berasaskan USCS konvensional dan perbezaan yang paling besar mungkin berlaku di kawasan tanah bercampur seperti campuran pasir dan campuran kelodak. Kajian yang dilaporkan dalam tesis ini mengkaji korelasi antara kaedah USCS dan kaedah CPTU untuk membandingkan nama yang diberi oleh kedua-dua kaedah. Kajian ini melibatkan beberapa ujian seperti “Standard Proctor”, Graviti Tentu dan ujian CPTU. Ia telah mendapati bahawa nama-nama CPTU dan USCS tidak akan menjadi sama kerana mereka datang dari dua sistem yang berbeza. Walau bagaimanapun, kedua-dua sistem hadir menggambarkan perubahan nama dalam sistem mereka kerana pengubahan mudah pada kandungan campuran.

ABSTRACT

Soil classification is very important in geotechnical engineering. Soil classification enables worldwide geotechnical engineer to convey the general characteristics of soil effectively. The soil classification at site is given by CPTU method without having to bring the sample to the laboratory. The result obtained from CPTU method is then correlated with the existing texture based classification that is Unified Soil Classification System (USCS). However, CPT-based SBT may not always agree with conventional USCS-based soil types and the biggest difference is likely to occur in the mixed soil region such as sand mixtures and silt mixtures. The study reported in this thesis examine the correlation between USCS method and CPTU method in order to compare the names given by these two method. The study involved several tests such as Standard Proctor, Specific Gravity and CPTU test. It was found out that The CPTU names and USCS names would never be the similar since they have come from two distinct systems. Nevertheless, both systems appeared to illustrate name change in their system due to simple alteration on the mix content.

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LIST OF ABBREVIATIONS

CPT	**C**one **P**enetration **T**est
LL	**L**iquid **L**imit
PI	**P**lasticity **I**ndex
PL	**P**lastic **L**imit
SBT	**S**oil **B**ehavior **T**ype
USCS	**U**nified **S**oil **C**lassification **S**ystem

NOMENCLATURES

a Cone area ratio

A_c Projected area of the cone

A_n Cross-sectional area of the load cell or shaft

B_q Pore pressure ratio

C_c Coefficient of curvature

C_u Uniformity coefficient

D_{10} Size at 10% finer by weight

D_{30} Size at 30% finer by weight

D_{60} Size at 60% finer by weight

f_s Sleeve friction

G_s Specific gravity

ρ_b The wet or moist density of a compaction sample

ρ_d The dry density of a compaction sample

q_c Cone resistance

R_f Friction ratio

u_2 The pore pressure acting behind the cone

w Moisture content

CHAPTER 1

INTRODUCTION

1.1 Background Study

Different soils with common properties may be classified into groups and subgroups according to their shared engineering behavior. The soil classifications provide a common language to convey the general characteristics of soil and these information is very useful for the geotechnical engineer. There are several existing soil classification systems around the world such as the American Association of State Highway and Transportation Official (AASHTO), Unified Soil Classification System (USCS), and the United States Department of Agriculture (USDA). A site based soil classification method can also be said to have been given by the Piezocone Penetration Test (CPTU) method which avoids having to bring the soil samples to the laboratory. The CPTU is similar to Cone Penetration Test (CPT) but giving additional measurement of the pore water pressure at one or more locations on the penetrometer surface. Cone penetration testing, with pore water pressure measurement – the CPTU - gives a more reliable determination of stratification and soil type rather than a standard CPT would do. Measurements made by the CPT produce four principle components; cone resistance (qc), sleeve friction (fs), pore pressure (u2), and the friction ratio (Rf). A CPT soil classification chart is technically referred to as a Soil Behavior Type (SBT) chart as it indicates the soil physical and mechanical properties or how it behaves rather than simply name. Tip resistance or normalized tip resistance (Qt) and friction ratio which is sleeve friction divided by tip resistance ((fs/q_t) data are used to derive most CPT soil charts.

1.2 Problem Statement

Even with the existing classification systems already in use today, none is totally absolute for any soil because the other methods would probably give a different name to the same soil. One such naming system is given by the CPTU method which is a reliable, cost effective, and valuable practice for the site investigation industry, capable of distinguishing subsurface conditions and obtaining various soil properties. However, the CPTU soil naming system is not official as much as the USCS. On the other hand, the existing textural-based classification systems such as the USCS have a weak link to in situ behavior, since they are measured on disturbed and remolded samples. Thus the CPTU naming system probably has an edge over other systems for future potential.

1.3 Objectives

The objectives of this research are listed as follow:

1. To determine the properties of laterite and sand samples and to appoint names according to USCS method.
2. To investigate the names of laterite and sand samples as mentioned above however using the CPTU method this time and to appoint names according to the CPTU method.
3. To compare the names given by CPTU method against those given by the USCS method.

1.4 Scope of Work

The analysis and classification of several prepared samples were performed using the CPTU method. After determining and identifying the soil names, the result were further evaluated using the Unified Soil Classification System (USCS) method. The names given by the CPTU method were compared against those given by the USCS method.

1.5 Dissertation Outline

This dissertation was divided into five chapters. Chapter 1 include the brief introduction of the research, problem statement, objectives, scope of work and importance of study. Next, in Chapter 2 consist of the review of the literatures. The process and methodology of the research are the Chapter 3. Meanwhile, Chapter 4 contain the results and discussion of the research. Finally, Chapter 5 conclude the research and the recommendation for the better research in future.

1.6 Importance of Study

This research correlated the names given by the CPTU method to those given by the USCS method. Given the above correlations associated with the CPTU and the USCS methods, one would be able to carry out only one of the two methods, and yet getting the interpretation based on the other method. Furthermore, it could also be possible that one particular name from a method corresponds to more than one names from the other method, which this study investigated. The CPTU data also can be used for deriving correlation with engineering soil properties for the purpose of hazard analysis and the design of foundation. The limitation of relying on any particular method in naming a soil were demonstrated in this study where a specific name of one method did not correspond to only another name of another method, and vice versa.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

Soil classification is one of the significant aspects in the discipline of geotechnical investigation. The conventional method by laboratory classification on samples retrieved from borehole drilling is commonly practiced, but in many cases this is both difficult and relatively expensive due to subsoil condition, especially for offshore engineering. Therefore, reliable and repeatable alternatives have been searched which has culminated into the new in situ testing methods, such as the CPTU. Soil classification or interpretation from the CPTU method provides parameters such as the cone resistance (qc) and sleeve friction (fs) data. However, for nonhomogeneous soil profiles the measurement of sleeve friction is sometimes less accurate and not reliable. The older version of CPTU method, called the CPT, could miss in the interpretation thin layers from a test, although it could be very important parameter in the design of a foundation (Cheng-hou et al., 1990). The newer version, the CPTU, however has overcome these difficulties by providing a continuous vertical profile of cone resistance (qc), sleeve friction (fs) and pore water pressure (u2) in every cm of the subsoil depth. The CPTU test is a useful tool to identify thinner layers with greater accuracy which cannot be employed by other traditional sampling procedures (Hossein and Danial 2016).

2.2 Introduction to Laterite Soil

Laterite is a type of soil that is rich in iron oxide. This soil originates from a wide variety of rocks weathering under strongly oxidizing and leaching conditions. Laterite soils normally formed in regions with tropical and subtropical weather where the climate is humid. The typical laterite is porous, claylike, and well drained, with permeability that can approach that of gravel or clean sand due to its numerous interconnected holes. The specific gravity ranges from 2.76 to 3.50. When excavated by traditional method, the typical percentages of soil components would be about 40% gravel, 30% sand, and usually less than 30% fines. For the Atterberg limits, the liquid limit ranges from 30 to 50%, the plastic limit from 10 to 20% and the plasticity index from 20 to 30%. The USCS could classify this material as clayey or silty gravel (GC or GM) however the classification for a certain laterite soil would depend on the results of from the USCS procedure carried out on that soil which may differ from case to case (Geotechdata, 2008).

2.3 Soil Classification by Unified Soil Classification System (USCS)

A soil classification procedure is often started with a grain size analysis then followed by other procedure such as the Atterberg limit determination for the USCS method. In appointing the name of a sample, for fine-grained soil, the Casagrande plasticity chart is used. For over 70 year these textural based classification based on the Casagrande chart has been used to provide a general guidance through empirical correlation based on field experience (Robertson, 2016).

The Unified Soil Classification System (USCS) was initially proposed by Casagrande in 1942, to be utilized in the field, in construction works undertaken by the United States Army Corps of Engineer. The system was then revised in 1952 with association of the United States Bureau of Reclamation. The USCS today it is a universal method used by engineers all over the world (Braja and Khaled, 2014).

The Unified Soil Classification System (USCS) classifies soil into two broad categories which are coarse-grained soil, categorized as gravel and sand, and fine -grained soil, categorized as silt and clay. Coarse grained soil are those with more than 50 % retained by the No. 200 sieve, which size is 0.075 mm. The coarse grain soil classifications can be further modified based on detailed grain size and fine content analysis. On the other hand, fine grained soils are those with more than 50 % passing the No. 200 sieve. Similarly, the fine grained soils can be further sub-grouped according to their plasticity values as determined by the Atterberg limits tests.

For coarse-grained soils, the group symbols start with prefix of G or S where G is used to indicate gravel or gravelly soil whereas S is for sand or sandy soil (Braja and Khaled, 2014). The symbols of W for well graded and P for poorly graded are used a modifier for any of the group symbols mentioned. On the other hand, the fine-grained soils have the group symbols started with prefixes M that represent the inorganic silt, C for inorganic clay, or O for organic silts and clays. The symbol Pt is used for peat, muck or other highly organic soils. Instead of that, some other symbols were adapted in this classification. Similarly as in the case of coarse grained soils, the symbols L for low plasticity and H for high plasticity are used for any of the group symbols mentioned.

2.4 Soil Behavior

Many publications have attempted to describe the complex characteristics and behaviors of natural soils based on actual observations in the field (Leroueil and Hight, 2003, Atkinson, 2007). For example, soil can change in volume during shear due to the rearrangement of the grains and the space corresponding to the voids. During shear, a soil can either be dilative or contractive, depending on whether it is coarse grained or fine grained, and compact or loose (Robertson, 2012). Besides, soil is essentially frictional as the strength and stiffness increase with increasing normal stress and depth of soil in the ground. These characteristics subsequently lead to classification of the soils into either coarse-grained such as for sands or fine-grained such as for silts and clays. On top of that, soils are essentially inelastic where the loading response is nonlinear beyond the initial very small threshold strain (Robertson, 2016).

Many factors affects the in-situ soil behavior. These are the geologic process which is related to origin such as the depositional and compositional features, environmental factors such as stress and temperature, as well as physical process such as due to aging and cementation (Robertson, 2016). Atkinson (2007) mentioned that the current in-situ soil state can be linked to the basic soil classification. There are several number of ways to define the current in-situ soil state. The current state in terms of over consolidation ratio (OCR) is commonly used to define fine-grained soils related to how much it has been compressed in the past as compared to the current compression. Fine grained ideal soil tend to have a unique normal compression line that is essentially parallel to critical state line. However, as mentioned by Been and Jefferies (1985), it is more popular to use state parameters (Ψ) to define the current state as it is related to the critical-state line.

There is an important distinction between the behavior of soil that are either ‘loose’ or ‘dense’ in evaluating the critical state as highlighted by Robertson (2016). During the critical state, a soil that is loose tend to contract while loaded and drained at the same time, or tends to cause pore pressure increase when the loading is carried out while the soil is undrained. In contrast, a soil that is dense tends to dilate at large shear strains, or tends to cause pore pressure decrease when the loading is carried out while the soil is undrained. The ability of a soil to change in volume during shearing process is called dilatancy and is a primary aspect of soil behavior. Hence, the soil behavior can be categorized into four broad and general groups as stated by Robertson (2012). The four broad groups of soil behavior as discussed in critical-state soil mechanics terms are drained-dilative, drained contractive, undrained-dilative, and undrained-contractive.

Soils can also be classified as either sandlike or claylike in terms of their behavior (Idriss and Boulanger, 2008). Sandlike soils are vulnerable to cyclic liquefaction whereas claylike soils are not vulnerable to cyclic liquefaction. In addition, Idriss and Boulanger (2008) also mentioned that over a fairly narrow range of plasticity index (PI), fine-grained soils can change in behavior from that of more fundamentally like sands to that which behavior is more fundamentally like clays. Furthermore, sandlike soils tend to have PI $<$10% and claylike soils tend to have PI > 18% (Bray and Saccio, 2006). Besides that, most natural soils possess some form of structure that can make their in-situ behavior distinct from those of ideal soils as suggested by Robertson (2016). The term "structure" can be used to relate the features either at the deposit macrostructure scale such as layering and fissures, or at the microstructure particle scale such as due bonding and cementation. On top that, the post-depositional factors can create some microstructural features on older natural soils. Meanwhile, the primary soils tend undergo an ageing process and bonding due to cementation.

2.5 Introduction to Cone Penetration Test (CPT)

The piezocone penetration test (CPTU) is an in situ testing method that is reliable and economical for assessing and classifying soil types (Abbas et al., 2015). Piezocone refer to a CPT cone that is equipped with one or more pore pressure sensor and often indicated with abbreviation CPTU. The piezocone may have between one and three pressure sensor located on different locations on the cone penetrometer surface, such as given in Figure 2.1. If the pore pressure sensor is located on the cone surface it is denoted as u_1, directly behind the cone denoted as u_2, whereas it is denoted as u_3 if located at the top of friction sleeve. For everyday application, most piezocone has the u_2 type of arrangement.

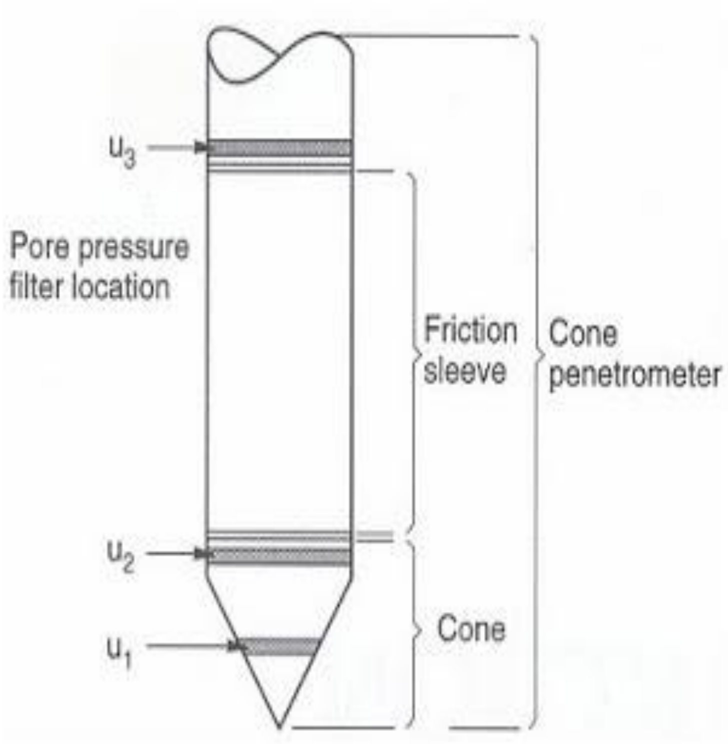
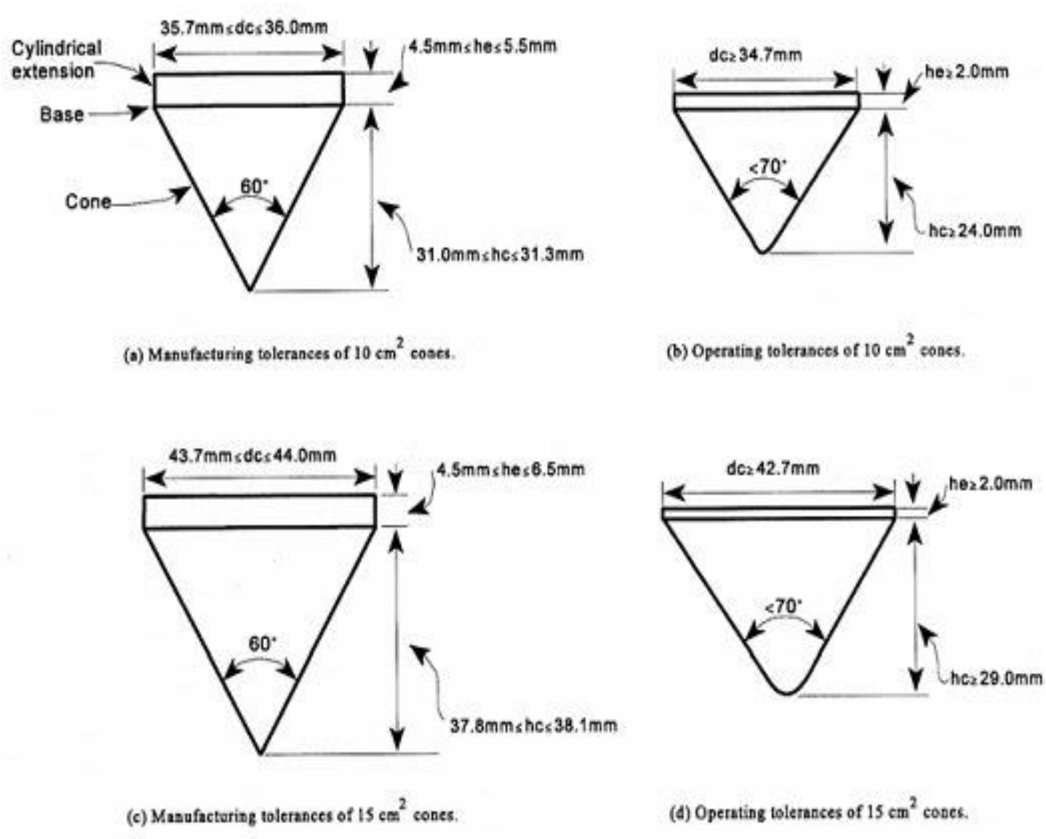


Figure 2.1: Pore pressure filters location (Lunne, 1997)

The test method involved hydraulically pushing a standard cone with typical cross-sectional area 10 cm^2, corresponding to a diameter of 3.6 cm, with the tip facing down, into the ground at an acceptable controlled rate between 1.5 -2.5 cm/s. Cheng-hou et al. (1990) states that in order to obtain a correct interpretation of the pore pressure data, it requires some knowledge whether the penetration is predominantly undrained or drained. For a standard cone with cross sectional area of 10 cm^2, and corresponding to a 2 cm/s penetration rate, a reasonable upper limit to soil permeability for an undrained penetration is in the order of $K=10^{-7}$ m/s. On the other hand, if the permeability is greater than 10^{-4} m/s, i.e., $K>10^{-4}$ m/s, the penetration would be most likely in drained condition.

During penetration, several number of variables are recorded at the cone head or along the sleeve. Cone resistance (q_c) which express the resistance of soils towards penetration is recorded at cone head. The cone resistance is theoretically related to the undrained shear strength of a cohesive soil while the sleeve friction (f_s) is recorded along the cone and indicates the adhesive strength of the material. Figure 2.2 shows a schematic section through an electric friction cone penetrometer. Friction ratio can be calculated from cone resistance and sleeve friction data. Normally, the friction ratio is expressed in term of percentage. The friction ratio is a very important parameter used in classifying the soil which indicates the cone behavior, or the reaction to the cone while being forced through the soil. High friction ratio would mean that the soil has a high clayey material, i.e. high c, low θ; while a low friction ratio would mean that the soil is made up of sandy material. The value of friction ratio normally range between 1% and 10%. The ratio seldom, if ever, exceed 15%. Sand are generally identified by exhibiting a friction ratio of < 1%.



CONE BASE AREA	NOMINAL			TOLERANCE MANUFACTURED (OPERATIONS)		
	BASE DIAMETER	CONE HEIGHT	EXTENSION			
cm^2	dc mm	hc mm	he mm	dc mm	hc mm	he mm
10	35.7	31.0	5.0	+0.3 - 0.0 (≥34.7)	+0.3 - 0.0 (≥24.0)	+0.0 - 0.5 (≥2.0)
15	43.7	37.8	5.0 - 6.0	+0.3 - 0.0 (≥42.7)	+0.3 - 0.0 (≥29.0)	+0.0 - 0.5 (≥2.0)

Manufacturing and Operating Tolerances of Cones (2)

Figure 2.2: Schematic section of electric friction-cone penetrometer tip
(ASTM D3441, 2005)

The components that make up the pore pressure sensors are a pressure transducer and a porous plastic filter which is usually made of plastic resin consisting a small cavity with incompressible, low –viscosity fluid. The porous plastic filter is inserted just behind the tapered head as shown in Figure 2.3. When penetrating a dense layers such as cemented siltstone, sandstone or conglomerate, the piezo filter element become compressed and therefore inducing high positive pore pressures. However, the porous plastic filters do not demonstrate this tendency but instead become brittle with time. Besides that, the pore pressure gradient around the cone may be relatively high when penetrating over consolidated clays. To obtain a fast and accurate reading, the filter and tubing between the filter and transducer must be fully saturated with fluid such as glycerin or silicon oil. In addition, the filter must be replaced regularly so it does not become clogged with soil (Vertek CPT, 2015).

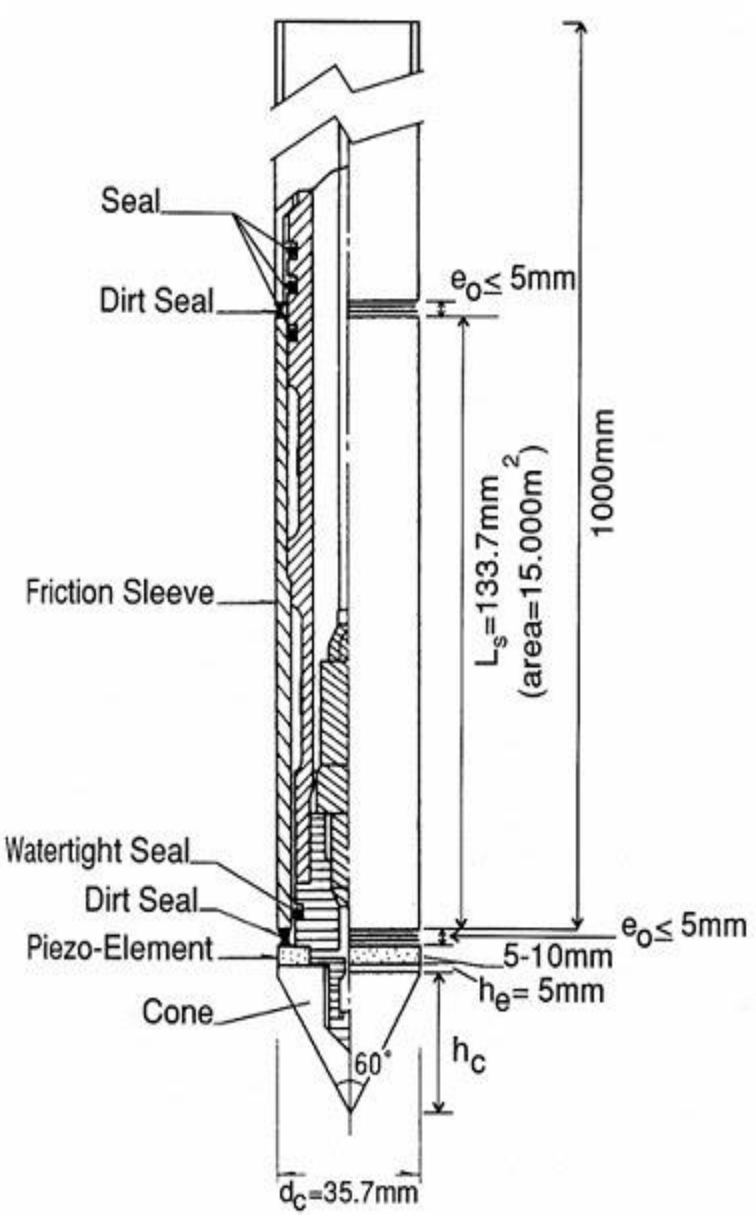


Figure 2.3: Schematic cross section of piezocone head, showing the piezo-element and friction sleeve (ASTM D5778, 2007)

Nevertheless, the procedure for the CPTU test is slightly different than for the basic CPT test. For a CPTU test, the pushing of cone must be paused long enough to take an initial pore pressure reading. Besides that, this is to allow the pore pressure around the cone to dissipate as subsequent reading are taken. As the cone is pushed into the ground, pore pressure builds up around the cone until the in-situ moisture dissipates into the surrounding soil. Soil's coefficient of consolidation which shows the compressibility and permeability of soil are the factor that affect the rate of dissipation (Vertek CPT, 2015)

2.6 Saturation of Piezocone

The piezocone measures the in situ pore pressure in either dynamic mode, i.e. while pushing the cone, or static mode i.e. while holding the cone stationary. The piezocone employs a porous plastic filter with pore size of about 2 µm, inserted just behind the tapered head. These porous plastic filter is usually made of hydrophilic polypropylene with a permeability coefficient of about 0.01 cm/second. Prior to its employment in the field, the filter element and other parts of pore pressure system must be saturated. This saturation must be conserved until the cone penetrometer reaches the groundwater surface or saturated soil (Brouwer, 2007). According to Cai et al. (2015), in order to ensure enough saturation, an all filter must be deaired in the laboratory and stored in a container under vacuum containing 100 % of glycerin for a minimum period of 24 hours. In addition, to prevent loss of saturation, a rubber membrane could be used prior to a testing to protect the transducer from being exposed to the dry surrounding. In the laboratory, each piezocone with filter attached would be saturated by placing it in chamber under a high vacuum for at least 30 minutes before allowing the previous deaired saturation fluid to enter the chamber. Besides, Cai et al. (2015) also recommended checking the effectiveness of saturation procedure by quickly adjust the pressure in the chamber and then comparing the form of the cone resistance and pore pressure response. Normally, in about 0.7 second, a pressure increase of 300 kPa can be recorded.

2.7 Dissipation Test

A dissipation test involves the measurement of the dissipation rates, when the generated pore pressure around the cone dissipates during a stop in penetration (Lunne et al., 1997). There are several factors that affect dissipation test. The factors are coefficient of consolidation and locking of cone rods. The coefficient of consolidation can directly impact the dissipation test results. This coefficient in turn depends on permeability and compressibility of soil. The locking of rods might have a little influence on dissipation rates. This is because the elastic strain energy in the rods can be released, which leads to a slight movement of the cone even when it is clamped.

2.8 Data Recording

Modern piezocone provides a continuous signals that require relatively complex data collection and processing. Usually the signals of piezocone are transmitted via a cable pre-threaded down the standard push rods. Before being passed to the data acquisition system, signals from piezocone transducers were amplified either within the cone or externally. As the piezocone penetrates, the data was saved at the rate of 25 readings per second including the pauses for addition of driving rods. Each saved reading was the average of at least 80 reading recorded at much higher rate in order to save electrical noise. In addition, the digital piezocone penetrometer include amplification and significantly increase the number of channel that can be measured (Cai et al., 2015).

2.9 Pore Pressure Correction

Pore water pressure in the ground generated as the result of cone penetration effect the measured results as mentioned by Lunne et al. 1997. According to Cai et al. 2015 because of the inner geometry of a cone penetrometer the ambient pore water pressure will act on the shoulder area behind the cone and on the end of friction sleeve. This is illustrated in Figure 2.4. This effect influences the total stress determined from the cone and friction sleeve and often referred to as 'the unequal area effect'. The cone area ratio "a" is use to represent the unequal area of cone resistance that is approximately equal to the ratio of the cross-sectional area of the load cell or shaft, A_n, divided by projected area of the cone, A_c as shown in Figure 2.4. The corrected total cone resistance is given by below equation.

$$qt = qc + u2\ (1 + a) \tag{2.1}$$

where:

u_2 = the pore pressure acting behind the cone

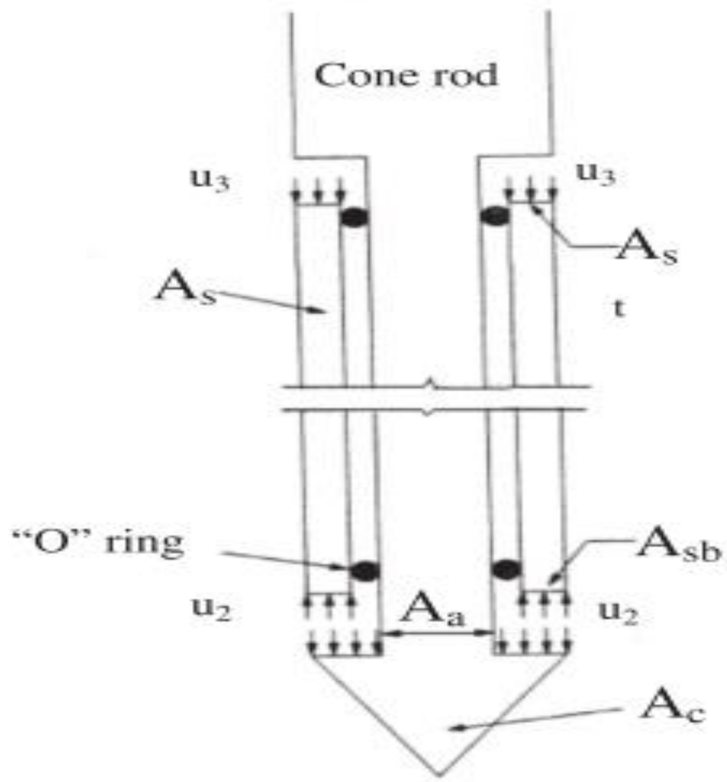


Figure 2.4: Piezocone cross section showing the pore water pressure effects on measured parameters (Cai et al., 2015)

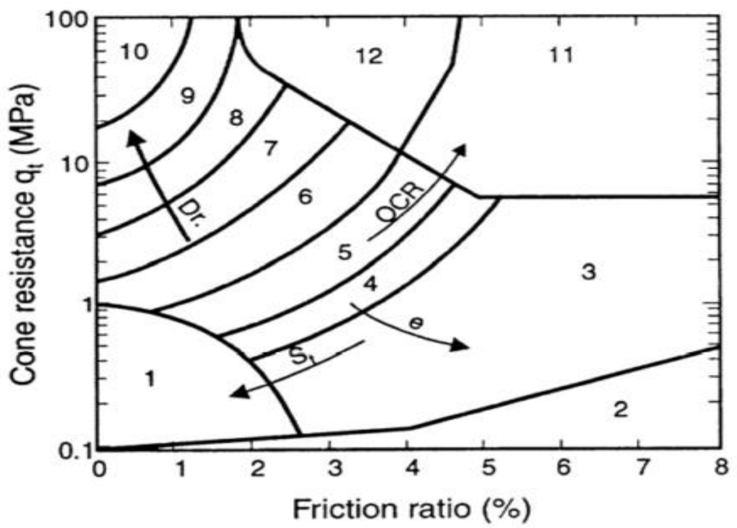
ASTM, D5778 (2007) defines the use of equal end-area friction sleeve to minimize the pore pressure effects. Robertson (2012) found that a strict standard have been implemented on the cone dimensional tolerances. Apart from that to increase the measured values of fs but within the standard tolerances, some cones are manufactured to have sleeve that slightly larger than cone tip. However, the measurement of f_s is generally less accurate than cone resistance especially in fine-grained soils. For most well designed piezocone, the accuracy of strain gauged of load cells is 0.1 % of the scale output (Robertson, 2012). Besides that, most commercial cones are designed is such a way to record a tip stress of 100 MPa. Hence, their accuracy for q_t is around 0.1 MPa (100 kPa).

2.10 Soil Behavior Type (SBT)

The major application of CPTU for determination of soil stratigraphy and identification of soil type is typically being accomplished using the chart that link cone parameter to soil type (Robertson, 2010). A CPT soil classification chart is technically referred to as a Soil Behavior Type (SBT) as it indicates the soil's physical and mechanical properties or how it behaves. Robertson (2010) mentioned that the CPT-based chart were prediction of Soil Behavior Type as the instrumented cone responds to in-situ mechanical soil behavior such as strength, compressibility, stiffness and not directly based on grain size distribution and plasticity soil classification criteria. The soil classification based on grain size distribution and soil plasticity are measured on disturbed samples and fortunately always relate reasonably well to in situ soil behavior. Consequently, there is often good agreement between USCS-based classification and CPT-based SBT (Molle, 2005). Nonetheless, the difference between USCS-based classification and CPT-based SBT are still available. For instance, according to Robertson (2010) in USCS a soil with 60 % sand and 40 % fines may be classifies as 'silty sand' (sand-silt mixtures) or clayey sand

(sand-clay mixtures). If the fines contain high clay content with high plasticity, the soil behavior are more controlled by the clay. Therefore, the CPT-based SBT will reflect this behavior and will predict a more clay-like behavior such as 'clayey silt to silty clay' (SBT zone 5, Figure 2.5). On the other hand, if the fines were non-plastic, the soil behavior will be controlled more by the sand. Thus, more sand-like soil type such as 'silty sand to sandy silt' will be predicted by CPT-based SBT (SBT zone 7, Figure 2.5). Fine-grained soils which is very stiff and heavily over consolidated tend to behave more like coarse-grained soil in that they tend to dilate under shear and can possess high undrained shear strength compared to their undrained strength and can have CPT-based SBT in either zone 4 or 5 (Figure 2.5) this zone is comparable to the one obtained by Robertson (2010). Silt with soft saturated and low plasticity tend to behave more like clays in that they have low undrained shear strength and can have a CPT-based SBT in zone 3 (Figure 2.5). As stated by all these examples, it clearly illustrates that CPT-based SBT may not always agree with traditional USCS-based soil types and the biggest difference is likely to occur in the mixed soil region such as sand mixtures and silt mixtures.

The chart by Robertson et al (1986) as shown in Figure 2.5 has 12 soil types and applies the basic CPT measurements of cone resistance (q_c) and sleeve friction (f_s). Robertson (2010) wrote that early Robertson et al (1986) chart has an advantage to be used in real time as it able to evaluate soil type during and immediately after the CPT because it only requires the basic CPT measurements.



Zone	Soil Behavior Type
1	*Sensitive fine grained*
2	*Organic material*
3	*Clay*
4	*Silty Clay to clay*
5	*Clayey silt to silty clay*
6	*Sandy silt to clayey silt*
7	*Silty sand to sandy silt*
8	*Sand to silty sand*
9	*Sand*
10	*Gravelly sand to sand*
11	*Very stiff fine grained**
12	*Sand to clayey sand**

** Overconsolidated or cemented*

Figure 2.5: SBT chart by Robertson et al. (1986) which is based on cone resistance (q_c) and friction ratio, (R_f) where ($R_f = (f_s / q_c)100\%$) (Robertson, 2010)

Meanwhile, in the past besides chart proposed by Robertson et al (1986) there are other several soil classification chart based on CPTU data that have been proposed. One of the soil classification chart based on CPTU data was chart proposed by Senneset and Janbu. Senneset and Janbu (1982) produced soil classification chart based on pore pressure ratio, B_q.. The pore pressure ratio is given by Equation 2.2. Cheng-hou et al (1990) states that for some overconsolidated and dilative soils, the excess pore pressure can be negative. These factors is however not considered by Senneset and Janbu chart. Instead of that, Senneset and Janbu chart given by Figure 2.6 shows an obvious poor identification of soil type.

$$Bq = \frac{\Delta u}{qt - \sigma vo} \tag{2.2}$$

where :

Δu = excess pore pressure

q_t = cone resistance corrected for pore pressure effects

σ_{vo} = total overburden pressure

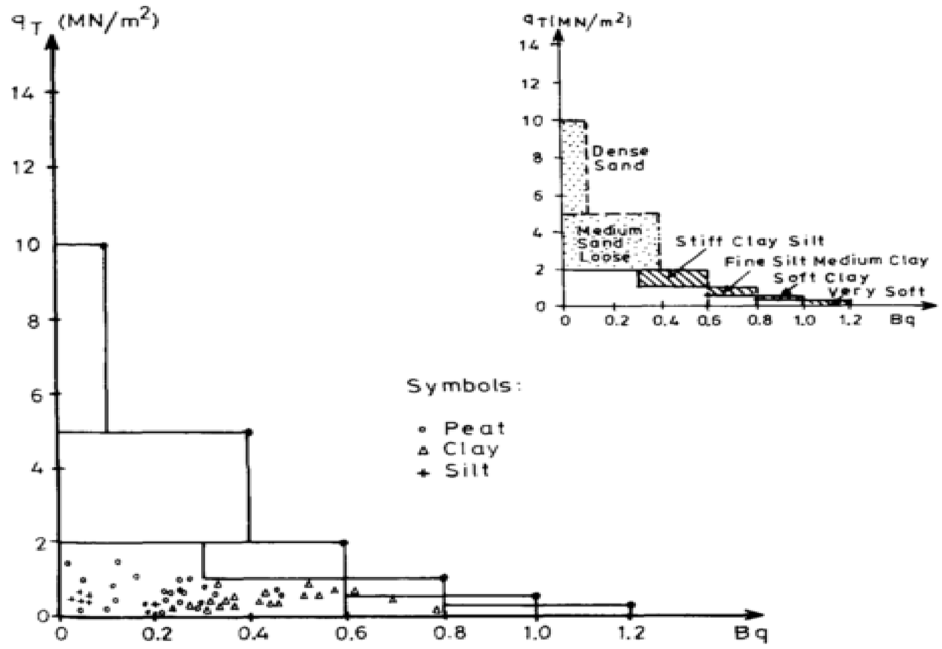


Figure 2.6 : Soil classification chart by Senneset and Janbu (1982)

Jones and Rust (1983) consider the limitation of the Senneset chart and proposed another chart. Chart developed by Jones and Rust (1983) use measured total cone resistance and excess pore pressure mobilized during cone advancement. Figure 2.7 shows that excess pore pressure plotted against net cone resistance. Jones and Rust chart have improved the previous chart given by Senneset and Janbu. Eslami and Fellenius (2000) stated that the chart is interesting as it identifies density which is the compactness condition of coarse-grained soils and consistency of fine-grained soils.

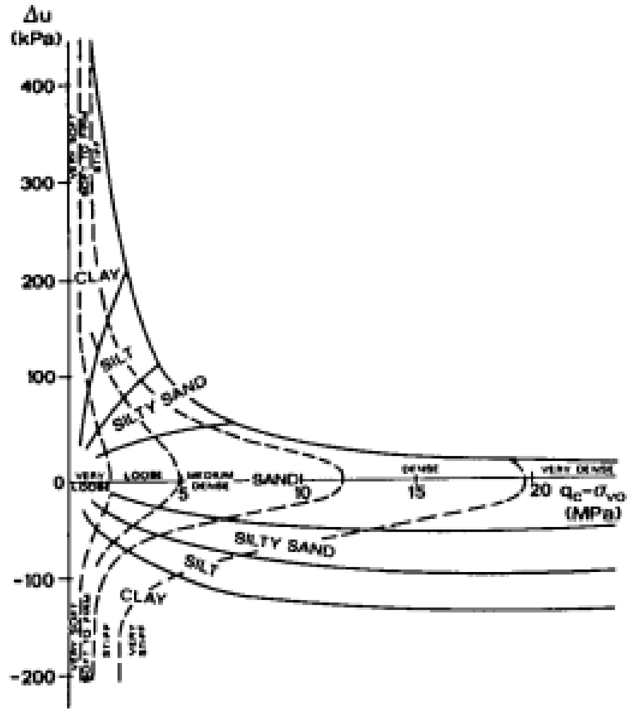


Figure 2.7 : Profiling chart using total cone resistance plotted against excess pore pressure (Jones and Rust 1983)

CHAPTER 3

METHODOLOGY

3.1 Overview

This chapter describes the method used to prepare the samples and to carry out the laboratory tests in classifying and naming the soil samples. The correct method of samples preparation is very important because it will affect the veracity of test results. Thus, samples preparation must be done accurately to assure the quality of data obtained. The flow chart describing the methodology of this research is given in the following Figure 3.1.

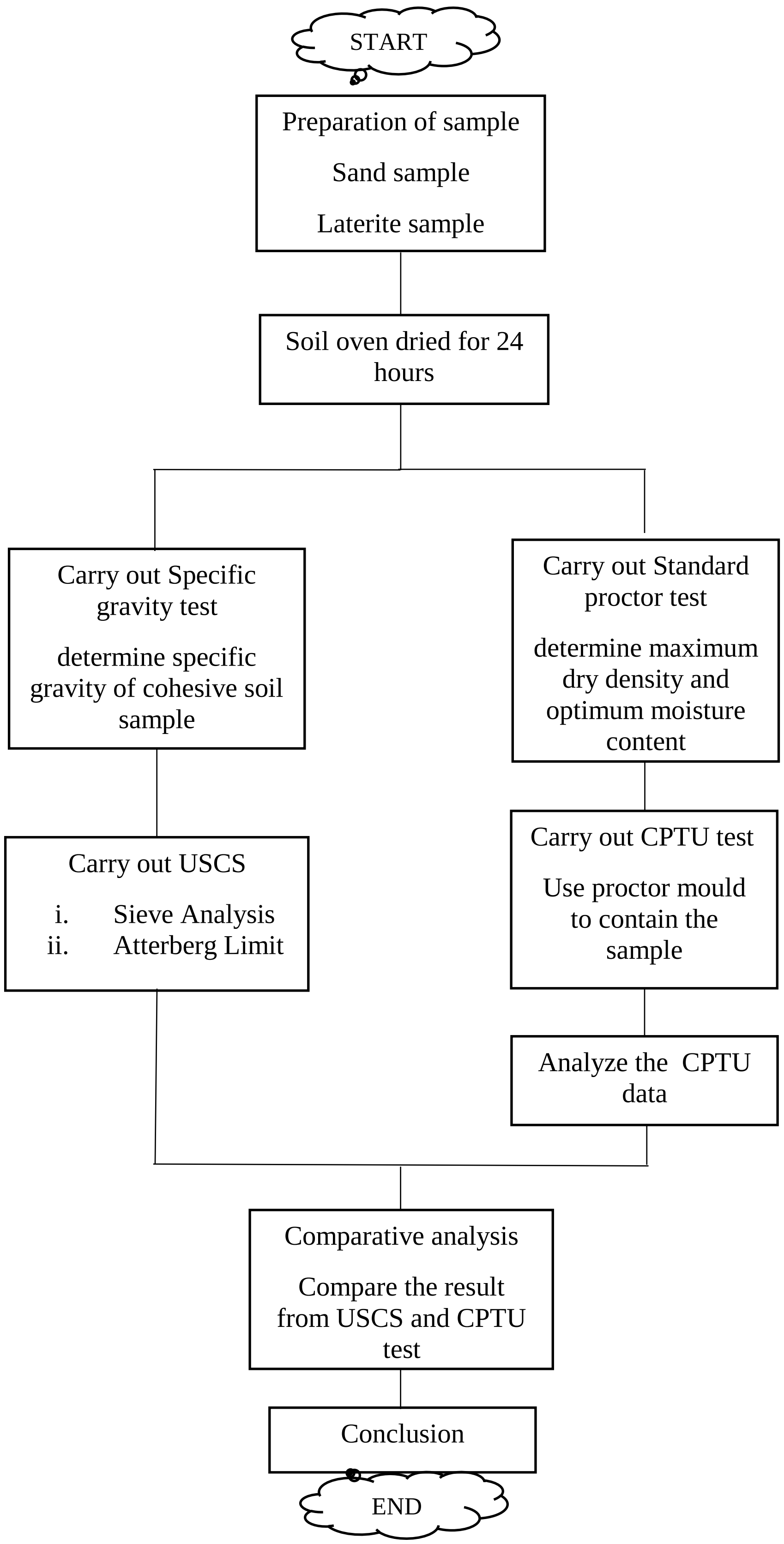


Figure 3.1 : Methodology

3.2 Preparation of Samples

Soil samples were prepared using laterite soil and sandy soil which are already available in the geotechnical laboratory. The laterite soil and sandy soil were obtained from nearby sources, mainly from the area of Bandar Baharu, Kedah. The soils were oven dried for 24 hours prior to the carrying out of tests. The dried samples are shown in Figure 3.2.

Figure 3.2: Laterite soil samples after drying

3.3 Preparation of Soil Samples in Proctor Mould

Proctor mould with inner diameter of 103.0 mm and height of 116.1 mm were used for the compaction tests. Note that each Proctor mould was used with matching base plate as these were both placed under the automatic drop hammer during Proctor compaction. The Proctor mould with base plate and collar is shown in Figure 3.3.