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Academic Session : **2009/2010**

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A special thanks to my dearly loved……

Dad, Mohd Salleh bin Omar

Mom, Wan Maimun bt Wan Nawi

My siblings Along, Angah, Nami

All of my family

And Nur Syamira for giving me all the supports and inspiration to carry on.

And also for all my friends

Thank you very much.

ACKNOWLEDGMENT

First of all, praise be to the Lord of the World, Allah S.WT for His flow able present and may His blessing and peace be upon our Prophet Muhammad and upon all his Family and Companions. From his bless and guidance, so I can complete this thesis successfully.

In preparing this project, I would like to thank my respectful supervisor, Mr. Muzamir Hasan for his non-stop support, guidance, advice, motivation and some valuable knowledge towards accomplishment of this study. I really appreciate all of his kindness in helping me all the time I needed.

I also want to thanks my lovely friends for support me and give support and some good ideas during this study especially Faruq and Audi, who helping me from taking the sample until finish my laboratory together. Thank you also for my other friends for everything you are giving me.

Lastly, a special thanks to all my precious and lovely family members and Nur Syamira, for their encouragement and support which had given me the strength to finish up the Final Year Project successfully. A thousand thank for all your inspiration and love. Thank you so much.

ABSTRACT

The strength and stability of the soil surface need to be define before any construction begin. The purposes of this study are to obtain the basic characteristic and shear strength of silty clay at the some location in Temerloh district. It has low compressibility and most of the structure constructed on it usually will be affected especially in stabilization and settlement. In order to prevent that from happen, the engineering properties of silty clay must be determined before the beginning of any design work. In this study, some disturbed samples of silty clay were required, taken from 2 different locations which are Kg Buntut Pulau and Kg. Sanggang for soil identification, classification and properties test. For the determination of basic properties of that silty clay, a laboratory test of sieve analysis and Atterberg Limit were conducted in order to get the relationship between moisture content, liquid limit and plasticity index of every sample. Then, the Unconsolidated-Undrained test was choosed as the suitable method of Triaxial Test for determining shear strength parameters for the silty clay. From the result, the moisture content of the sample from both sites is 34.52% and 50.8% respectively. Meanwhile, for the Atterberg limit result, both site recorded the reading of 51.10% and 54.87% respectively for liquid limit and also 25.31% and 28.78% for the plastic limit. Then the value of plasticity index which obtain form the liquid and plastic limit is 25.79% and 26.09%. Then for the particle size distribution test, site A was recorded percentage of silt and clay as the highest with 51% and for same as for site B with 56% of silt and clay for the most dominant particle. The shear strength test results have clearly shown the weakness of Kg Buntut Pulau and Kg. Sanggang soil which are within soft clay soils strength with the average recorded value of 37.91kPa and 26.41kPa respectively. The data and result from this project can be used as a preliminary forecast for further investigation of soil properties and shear strength in the future construction and development.

ABSTRAK

Kekuatan dan kestabilan permukaan tanah tersebut mesti ditakrif terlebih dahulu sebelum pembinaan bermula. Tujuan kajian ini dijalankan adalah untuk mendapatkan ciri-ciri asas dan kekuatan ricih tanahliat berkelodak di sesetengah lokasi di daerah Temerloh. Ia mempunyai kebolehmampatan yang rendah dan kebanyakan struktur yang dibina di atasnya kebiasaanya akan dipengaruhi oleh kestabilan dan kemendapan. Untuk mengelakkan kejadian seumpama itu berlaku, sifat kejuruteraan tanah liat berkelodak mesti ditentukan sebelum kerja rekaan bermula. Dalam kajian ini, sedikit sampel tanah liat berkelodak terkacau diperlukan yang diambil dari 2 lokasi berbeza iaitu Kg Buntut Pulau dan Kg. Sanggang untuk penentuan, pengkelasan dan ciri-ciri tanah. Untuk penentuan ciri-ciri asas tanah liat berkelodak ini, ujian makmal yang merangkumi ujian taburan partikel tanah dan had Atterberg dijalankan untuk mendapatkan hubungan antara kandungan air, had cecair dan indeks keplastikan setiap sampel. Ujian ketidakmendapan-ketidakaliran sebagai kaedah yang sesuai untuk ujian kekuatan ricih untuk penentuan parameter kekuatan ricih tanah liat berkelodak. Daripada keputusan, kandungan air untuk kedua-dua tempat ialah 34.52% dan 50.8%. Untuk keputusan ujian had Atterberg pula, kedua-dua tempat mencatatkan nilai 51.10% dan 54.87% untuk had cecair dan juga 25.31% dan 28.78% untuk had plastik. Untuk ujian taburan partikel tanah pula, tempat A mencatatkan peratus tanah kelodak dan tanah liat sebagai yang tertinggi dengan 51% begitu juga dengan tempat B 56% untuk tanah kelodak dan tanah liat sebagai partikel tanah yang paling dominan. Ujian terhadap kekuatan ricih tanah menunjukkan kelemahan yang ketara terhadap kekuatan ricih tanah tersebut iaitu di dalam lingkungan bacaan kekuatan bagi tanah liat lembut dengan bacaan purata 37.91kPa dan 26.41kPa. Segala data dan keputusan yang diperoleh daripada ujian-ujian di dalam kajian ini dapat digunakan sebagai ramalan awal untuk kajian ciri-ciri asas dan kekuatan ricih yang lebih lanjut untuk pembinaan masa hadapan.

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S	-	Shear strength
c'	-	effective stress cohesion intercept
Θ'	-	effective stress angle of friction
w,MC	-	moisture content
PL	-	plastic limit
PI	-	plasticity index
LL	-	liquid limit
c_u	-	shear strength

CHAPTER 1

INTRODUCTION

1.1 Introduction

Malaysia has mission to be one of the modern and sophisticated countries in the world as stated in Wawasan 2020. Recently, this scenario caused a lot of massive development in our country. There are a lot of mega structures and skyscrapers which have been magnificently constructed all over the place in Malaysia such as PETRONAS Twin Tower, Kuala Lumpur International Airport (KLIA) and many more. Lately, engineers are so eager to get more money which effectuates them to do constructions without considering the quality of the soil. Consequently, there are some constructions turns out to be a disaster since they were built at low quality soil structure. But, many said it caused by natural disasters and no one could blame the engineers. Actually, there are many factors related to the failure of a project, which is mainly caused by natural disaster and due to development activities surrounding those areas. But one of the factors which also become the reason of a soil failure is the strength of the soil itself.

In order to build a good construction project in this industry, we have to consider some factors that could affect the project. The most important thing that we need to think about is the condition of the chosen site. The main aspect that related to site condition is the condition of the soil. When a construction project needs to develop, type of soil at the area will be the most important element to be considered. Many soils have prove to be problematic in geotechnical engineering because of the way they expand, collapse, disperse, undergo excessive settlement and have a distinct lack of strength (Fauziah, 2007). The strength and stability of the soil surface needs to be defined before begin any construction by doing some experiments.

The purposes of this study are to obtain the basic properties and shear strength of silty clay at some location in Temerloh district in Pahang. Silty clay is defined as a clay type soil which has the combination of silt soil and the percentage of clay soil itself was greater than silt respectively. It has low compressibility and most of the structure constructed on it usually will be affected especially in stabilization and settlement. To prevent it from occur, the engineering properties of silty clay must be determined before design work start. Therefore, geotechnical engineer may avoid any problems related to construction on the soft soil. The most critical silty clay problem is the differential settlement which will cause the building to crack and provide other types of destruction as well. That is why the research related to this type of soil needs to be continued as the data and result from this project can be used as a guideline for further investigation of soil properties and shear strength in the future construction and development especially in Temerloh district.

1.2 Objective

The objectives of this study are as follow:

i. To determine the basic properties of Temerloh silty clay.
ii. To determine the shear strength of Temerloh silty clay.

1.3 Scope of Study

This study is conducted at some locations in Temerloh, Pahang Darul Makmur. It is one of the famous districts in Pahang because of its “Gulai Tempoyak Ikan Patin” which located near to Pahang River, the longest river in peninsular of Malaysia. Since it is just a stone throw away to Pahang River, most of the soil in this town is found to be silty clay. That is why this place was chosed for the study. This study only focuses on determination of the basic properties and shear strength of silty clay soil. All the testing for this study is conducted in laboratory by using British Standard 1377:1990 as a reference.

Figure 1.1: Study Location at Temerloh (www.ppdbera.net).

1.4 Problem Statement

Malaysia is growing as a development country throughout the years. Due to development of our country, construction industry become more rapid and the use of soft soil area like silty clay in Malaysia is increasing as the other land with the better soil condition is decreasing. Because of this, some problem occurred regarding to the use of silty clay on construction such as the stability and settlement of the soil. There are many cases happen due to failure of the building occur because the area which contain of the silty clay are not reinforced with proper ground improvement technique. Because of that, we need to do more investigation on the characteristic and strength of the soil with the hope that it can help engineers to design better structures on silty clay and reduce any failure.

1.5 Importance of Study

This study is so important to overcome the problem of the silty clay soil in construction especially in Temerloh, Pahang. In this study, the basic properties and shear strength of silty clay soil in Temerloh will be determined by some laboratory testing such as moisture content, sieve analysis, Atterberg limit and Unconsolidated Undrained Test. This study is also important because many of location in Pahang are not much explore by engineers, especially at Temerloh district. Hopefully, the outcomes will become a part of data of silty clay soil in Pahang. The result from this study can be referred by engineers as useful guideline for them to apply in construction on silty clay soil.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The development of construction project on silty clay area was increasing lately due to insufficient of other suitable area. Silty clay commonly occurs as soft, wet unconsolidated surficial deposits that are integral parts of the wetlands systems (Edward, 2006). They are known as problematic soils, with high compressibility and low shear strength. Silty clay usually present at the location which is nearer to river area all over the world. All building or construction project built on this type of soils normally faces with the crisis of weak foundation soil conditions. But still, this type of soil have chosen to be develop because there no longer an option to choose the suitable ground. The most important aspect is the best and strategic areas which been looking by some engineers to construct a structure.

Silt soils represent the excessive form of soft soil in Malaysia. If it is possible, engineers would avoid silty soil as the foundation beneath their structure or construction project. They are difficult to sample and test using normal soil techniques and in fact there is less adequate engineering system in place for classifying these soils (Edward, 2006).

The data relate to soil properties are composed during the soil analysis at site. Site survey work is needed and extremely important in order to get the detail about any site before construction begin. Soil characteristic was the first thing need to be determined by field inspection of the soil and by laboratory testing of some group of selected soil.

2.2 Soil Classification

Soil classification is carried out in order to define a small number of different groups of soil on any site. Each soil group may consist of a stratigraphically defined geological unit of a site. Particle size, plasticity and organic content may be more important to the geotechnical engineer than time of deposition. The three main tools used to classify soil are soil description, particle size distribution analysis and plasticity testing (Whitlow, 2001)

Soil classification, although introducing a further stage of data acquisition into site investigation, has an important role to play in reducing the costs and increasing the cost-effectiveness of laboratory testing. Classification tests allow the soils on a site to be divided into a limited number of random groups, each of which is estimated to contain materials of similar geotechnical properties.

Different soils with similar properties may be classified into groups and sub-groups according to their engineering behavior (Das, 2006). Currently, there are two major soil classification systems are available for general engineering use which are American Association of State Highway and Transportation Officials (AASHTO) classification system and the Unified Soil Classification System (USCS). Both systems take into consideration the particle-size distribution and Atterberg limits for their classification.

Atkinson (2007) mentions that, it is important to distinguish between soil description and soil classification. Description is simply what can see with the eyes and how the soil responds to simple test. A classification is a scheme for separating soils into broad groups, each with broadly similar behavior.

An engineering soil classification system is only useful for feature applications. In the constructions of important soil structures the classification must be supplemented by laboratory tests other than those needed for classification. The experiment is made using disturbed samples recovered from site as well as undisturbed samples from boreholes and excavations (Aysen, 2005).

According to Atkinson (2007), there are various classifications schemes for different purpose; there are agricultural classifications (Figure 2.1) based on how soils support crops and geological classifications (Figure 2.2) based on the age of the deposit or nature of the grains. For civil engineering purposes soil classifications should be based mainly on mechanical behaviour.

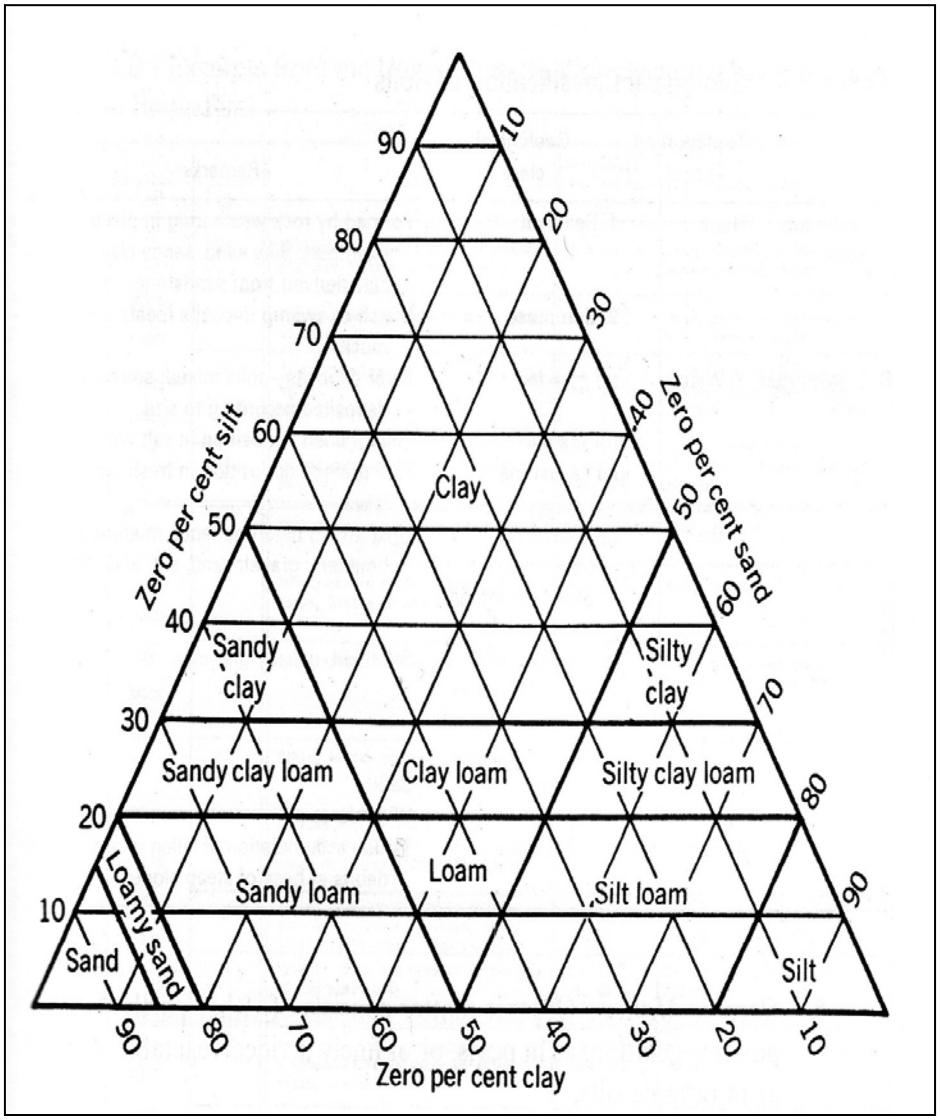


Figure 2.1: Agricultural Classification of Soil (Das, 2001).

Group	Transporting agent	Geological class	Remarks
A. Sedentary soils	None	1. Residual	Formed by rock weathering in place. Examples: Silty sand, sandy clay, or silty clay derived from sandstone.
		2. Cumulose	Marsh or swamp deposits (peats and mucks).
B. Transported soils	1. Water	(a) Alluvial	River deposits—soils mixed, sorted, and deposited according to size.
		(b) Marine	Fine-grained deposition in salt water.
		(c) Lacustrine	Fine-grained deposition in fresh water lakes.
	2. Ice	(a) Glacial till (moraines, till plains, drumlins, etc.)	Unstratified heterogeneous mixture of boulders, gravel, sand, silt, and clay.
		(b) Fluvio-glacial deposits (eskers, terraces, outwash plains, etc.)	Stratified, usually granular.
	3. Wind	(a) Dunes	Sand.
		(b) Loess	Windblown silt.
	4. Gravity	(a) Colluvial	Talus—accumulation of fallen rock and rock debris at base of steep slopes.

Figure 2.2: Geological Classification of Soils (Das, 2001).

2.2.1 AASHTO Classification System

According to Das (2006), the AASHTO Soil Classification System (Figure 2.3) was developed by the American Association of State Highway and Transportation Officials in 1929 as the Public Road Administration classification system. It is used as a guide for the classification of soils and soil-aggregate mixtures for highway construction purposes. The classification system was first developed in 1929, but has been revised several times since then, with the present version proposed by the Committee on Classification of Materials for Sub-grades and Granular Type Roads of the Highway Research Board in 1945 (ASTM designation D-3282; AASHTO method M145).

General Classification	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve Analysis, % passing											
2.00 mm (No. 10)	50 max	...	...	...	...	...	...	...	...	...	...
0.425 (No. 40)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
0.075 (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid Limit	...		...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[1]
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand				silty soils		clayey soils	
General rating as a subgrade	excellent to good							fair to poor			

Figure 2.3: AASHTO Soil Classification System (from AASHTO M 145 or ASTM D3282).

2.2.2 Unified Soil Classification System

The Unified Soil Classification System (USCS) as shown in Figure 2.4 below was originally proposed by Casagrande in 1942 and then was revised in 1952 by the Corps. of Engineers and the U.S. Bureau of Reclamation (Das, 2001). At present, it was commonly used by geotechnical engineers, various organizations and building codes. There are two categories which classifies soils in this system which is:

a) Coarse grained soils that are generally and sandy in nature with less than 50% passing through the No. 200 sieve. The group symbols start with a prefix of G or S. G stands for gravel or gravelly soil, and S for sand or sandy soil.

b) Fine-grained soils are with 50% or more passing through the No. 200 sieve. The group symbols start with prefixes of M, which stands for inorganic silt, C for inorganic clay, or O for organic silts and clays. The symbol Pt is used for peat, muck, and other highly organic soils.

Other symbols used for the classification are shown in Table 2.1 below.

Table 2.1: Symbol for Soil Classification.

Symbol	Description
W	well graded
P	poorly graded
L	low plasticity (liquid limit less than 50)
H	high plasticity (liquid limit more than 50)

Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 3.2)	GM
			$PI > 7$ and plots on or above "A" line (Figure 3.2)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 3.2)	SM
			$PI > 7$ and plots on or above "A" line (Figure 3.2)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 3.2)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 3.2)[e]	ML
		Organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$; see Figure 3.2; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 3.2)	CH
			PI plots below "A" line (Figure 3.2)	MH
		Organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$; see Figure 3.2; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

[a] Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.

[b] Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.

[c] $C_u = \frac{D_{60}}{D_{10}}$; $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$

[d] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 3.2, use dual symbol GC-GM or SC-SM.

[e] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 3.2, use dual symbol CL-ML.

Figure 2.4: Unified Soil Classification System (Aysen, 2005).

2.2.3 Classification Based on Plasticity

The engineering behavior between coarse-grained soil and fine-grained soil has a clear division between themselves. The engineering behavior of a coarse-grained soil is based on grain size distribution while fine-grained soil is based on plasticity characteristic (Day, 1999).

Figure 2.5 show an alternate classification system known as the inorganic soil classification based on plasticity (ISBP). According to the ISBP, a nonplastic soil is defined as a soil where the minus No. 40 fraction cannot be rolled at any water content, or the plastic limit is equal to or greater than the liquid limit (Day, 1999). For plastic soils, the subdivisions are based on plasticity characteristics (LL and PI) while for the nonplastic soils, the subdivisions are based on grain size distributions.

The ISBP classification system has the advantage which is the soil is generally arranged from the best to worst inorganic coil type in terms of shear strength, compressibility and expansion potential. It is also does not have dual symbols. Meanwhile the disadvantage of this system is that it does not include organic soils, but they should probably be separately classified because of their unique engineering properties.

Major divisions (1)	Subdivisions (2)	ISBP symbol (3)	Typical names (4)	Laboratory classification criteria (5)
Nonplastic soils	Gravels (Greater fraction of total sample is retained on No. 4 sieve)	GW	Well-graded gravels, sandy gravels, silty-sandy gravels	$C_u \geq 4$ and $1 \leq C_c \leq 3$
		GP	Poorly graded gravels, gravel-sand-silt mixtures	Does not meet C_u and/or C_c criteria listed above. In addition, the % passing No. 200 sieve < 15%
		GM	Poorly graded, nonplastic silty gravels, gravel-silt mixtures	Does not meet C_u and/or C_c criteria listed above. In addition, the % passing No. 200 sieve ≥ 15%
	Sands (Greater fraction of total sample is between No. 4 and No. 200 sieves)	SW	Well-graded sands and gravelly sands	$C_u \geq 6$ and $1 \leq C_c \leq 3$
		SP	Poorly graded sands or sand-gravel-silt mixtures	Does not meet C_u and/or C_c criteria listed above. In addition, the % passing No. 200 sieve < 15%
		SM	Poorly graded, nonplastic silty sands, sand-silt mixtures	Does not meet C_u and/or C_c criteria listed above. In addition, the % passing No. 200 sieve ≥ 15%
	NP silt	MN	Nonplastic silts, rock flour. Gravelly silts and sandy nonplastic silts	Greater fraction of the total sample passes the No. 200 sieve. Silts are nonplastic
Plastic soils	Plastic silts (Minus No. 40 fraction plots below *A*-line)	GM*	Plastic silty gravels, gravel-silt mixtures	50% or more particles retained on the No. 200 sieve with the greater fraction of gravel size
		SM*	Plastic silty sands, sand-silt mixtures	50% or more particles retained on the No. 200 sieve with the greater fraction of sand size
		ML MI MH	Plastic silts, sandy silts, and clayey silts	For silt of low plasticity (ML) PI ≤ 10 For silt of intermediate plasticity (MI) 10 < PI ≤ 30 For silt of high plasticity (MH) PI > 30
	Clays (Minus No. 40 fraction plots on or above *A*-line)	GC*	Clayey gravels, gravel-clay mixtures	50% or more particles retained on the No. 200 sieve with the greater fraction of gravel size
		SC*	Clayey sands, sand-clay mixtures	50% or more particles retained on the No. 200 sieve with the greater fraction of sand size
		CL CI CH	Clay, sandy clays, and silty clays	For clay of low plasticity (CL) PI ≤ 10 For clay of intermediate plasticity (CI) 10 < PI ≤ 30 For clay of high plasticity (CH) PI > 30

Figure 2.5: Inorganic Soil Classification Based on Plasticity, ISBP (Day, 1999).

2.3 Soil Description

Soil description is basically what can see with the eyes and how the soil responds to simple tests (Atkinson, 2007). It is helpful to have a simple system to describe the fundamental features. There are several methods published in National Standard and to some extent, these reflect the characteristics of the most common soils in the county. A simple and universal scheme for soil description is as follow:

a) The nature of the grains – The most important features of soil grains are their size and the grading, together with the shape and surface texture of the grains and their mineralogy.

b) The current state of the soil – The important indicators of the state of a soil is the current stresses, the current water content and the history of loading and unloading. These are reflected by the relative strengths and stiffnesses of samples of the soil.

c) The structure of the soil – This consists of fabric and bonding. Natural soils are rarely uniform and they contain fabric features, such as layers, which are seen in small samples and in large exposures. In some natural soils the grains are weakly bond together. If the grains are strongly bonded material has become a rock.

d) The formations of the soil – Soils are formed in different ways. They may be deposited naturally from water, ice or wind; they may be the residual products of rock weathering; they may be compacted by machines into embankments and fills.

Atkinson (2007) stated that, more completed scheme for description of soils is given in BS 5930:1999. It is more detailed and provides useful quantitative values for some visual observations. Usually, the characteristic of the soil does not change during typical civil engineering works. The grading changes when some of the weak and fragile soil particles break during loading. In contrast, the condition of a soil does change as soils near foundations and excavations are loaded or unloaded and compress or swell.

The behavior of formation of a soil will influence its nature, its initial state and its structure. Since most natural soils have some structure it is important to always test some sample, but their behavior should be inspected within the basic structure recognized for reconstituted samples.

Table 2.2(a) below shows a systematic and standardized order of description similar to one which has been established. Some terms has been used for a description of colour and particle shape like; the colour of a soil should be described in the moist condition as pale, dark, or mottled and black, white, grey, red, brown, orange, yellow, green and blue. Equidimensional particles maybe described as *rounded, sub-rounded, sub-angular*, or *angular* as shown in Figure 2.6. Meanwhile, Table 2.2(b) shows the consistency of cohesive soils which has been described in terms of its undrained shear strength and the consistency of non-cohesive soils is described in terms of the density index according to Table 2.2(c) (Aysen, 2005).

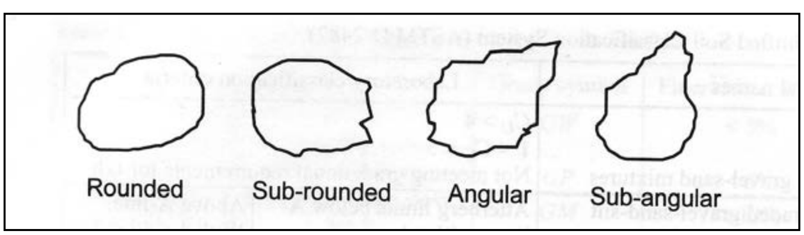


Figure 2.6: Equidimensional Particles (Craig, 2004).

Table 2.2(a): Systematic and Standardized Order of Description (Aysen, 2005).

Order of description	**Details of description**
Composition	Group symbol, soil name, plasticity or particle characteristics, colour, secondary components, and minor components.
Conditions	Moisture condition (disturbed or undisturbed), consistency (undis.)
Structure	Zoning, defects, cementing (undisturbed)
Additional observations	Soil origin and other matters if significant.

Table 2.2(b): Consistency of Cohesive Soils (Aysen, 2005).

Term	**c_u (kPa)**	**Field guide to consistency**
Very soft	≤ 12	Exudes between the fingers when squeezed in hand.
Soft	$> 12 \leq 25$	Can be moulded by light finger pressure.
Firm	$> 25 \leq 50$	Can be moulded by strong finger pressure.
Stiff	$> 50 \leq 100$	Cannot be moulded by fingers; can be indented by thumb.
Very stiff	$> 100 \leq 200$	Can be indented by thumbnail.
Hard	> 200	Can be indented with difficulty by thumbnail.

Table 2.2(c): Consistency of Non-Cohesive Soils (Aysen, 2005).

Term	**Density index**
Very loose	≤ 0.15
Loose	$> 0.15 \leq 0.35$
Medium dense	$> 0.35 \leq 0.65$
Dense	$> 0.65 \leq 0.85$
Very dense	> 0.85

2.3.1 Moisture Content

Moisture content is one of the elements which could affect the properties of soil. The percentage of moisture content in a soil is very important to attain its characteristic. There are many factors which can influence moisture content at site such as weather, soil types and also ground water table (Ariffah, 2008). For example, the non-stop rain during monsoon season can increase the amount of moisture content at site. The soil profile below the ground surface also can affect the moisture content by soil at site.

There are some techniques can be carry out in order to determine the moisture content in a soil sample such as oven drying method, sand bath method, alcohol method, calcium carbide method and pycnometer method (Punmia, 2004). The moisture content also can be determined by performed the laboratory test of Standard Proctor Test and Atterberg Limit. For a typical clay soil, the moisture content might be in the range 20% to 70% and the unit weight might be 18 to 22kN/m^3 about twice of the moisture content (Atkinson, 2007).

2.3.2 Particle Size Distribution

According to Craig (2004), the particle size analysis of a soil sample involves determining the percentage by mass of particles within the different size ranges. Sieve analysis method can be conducted to determine the particle size distribution of a coarse soil. The soil sample is passed through a series of standard sieves having successively slighter mesh sizes. The mass of soil retained in each sieve is determined and the cumulative percentage by mass passing each size is calculated. If fine particles are present in the soil, the sample should be treated with a deflocculating agent and washed through the sieves.

Meanwhile for the particle size distribution of a fine soil or the fine fraction of a coarse soil can be determined by the sedimentation method. This method is based on Stokes' Law which governs the velocity at which spherical particles settle in a suspension: the larger the particle the greater is the settling velocity and vice versa. The law does not apply to particles smaller than 0.0002mm (Craig, 2004).

A particle size distribution curve describes the percentage by mass of particles of the different size ranges as shown in Figure 2.7 below. The horizontal axis represents the particle size on a logarithmic scale. Meanwhile the vertical axis represents the percentage by weight of particles that are finer than a specific size on the horizontal axis.

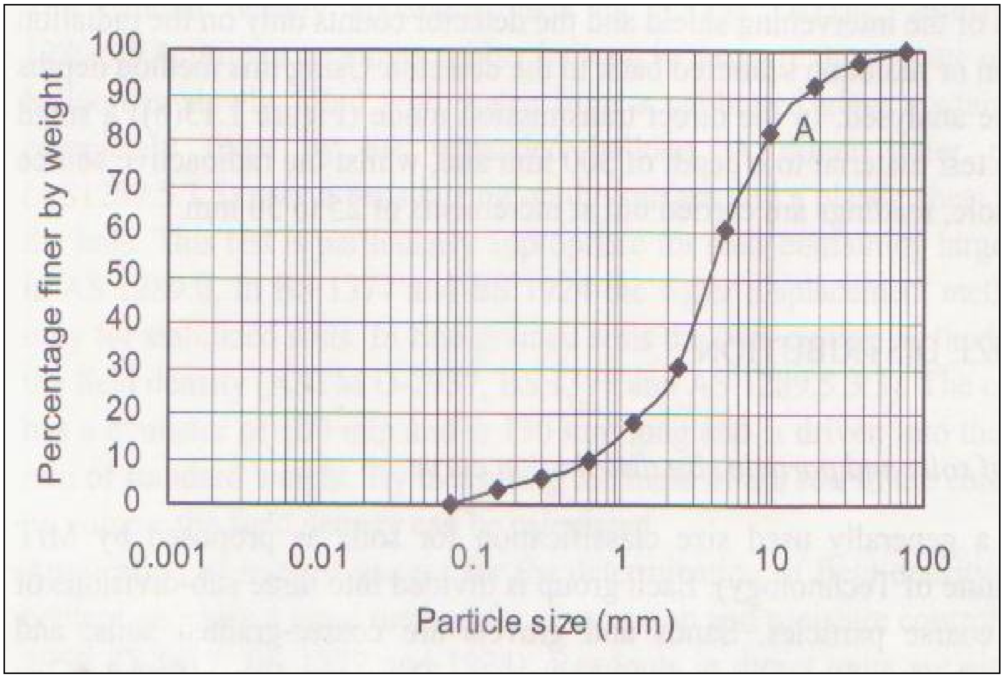


Figure 2.7: Particle Size Distribution Curve (Aysen, 2005).

The sizes of particles that make up soil vary over a wide range. Soils generally are called gravel, sand, silt or clay, depending on the predominant size of particles within the soil. According to Das (2006), gravels are pieces of rocks with occasional particles of quartz, feldspar, and other mineral. Sand particles are made of mostly quartz and feldspar. Other mineral grains also may be present at times. Silts are the microscopic soil fractions that consist of very fine quartz grains and some flake-shaped particles that are fragments of micaceous minerals. Clays are mostly flake-shaped microscopic and submicroscopic particles of mica, clay minerals and other minerals. Clay have been defined as those particles which develop plasticity when mixed with a limited amount of water.

2.4 Soil Consistency

Atterberg, a Swedish scientist has developed a method to describe the consistency of fine-grained soils with varying moisture content in the early 1990s (Das, 2006). Soil is behaving more like solid at very low moisture content. When the moisture content is very high, the soil and water may flow like a liquid. Hence, depending on the moisture content, the behavior of soil can be divided into four basic states – solid, semisolid, plastic and liquid.

2.4.1 Atterberg Limits

Atterberg Limits are a series of tests which are used to give empirical information on the soils reaction to water. This information is of a qualitative nature and tells us the plastic limit, the liquid limit, the plasticity index and linear shrinkage of the materials (Figure 2.8). The Atterberg limits relate to the moisture contents of cohesive soils corresponding to empirical defined boundaries between states of consistency (liquid, plastic, solids) of the fraction of soil passing the 425 micron sieve. These boundaries and the soil phases they define are accurate.

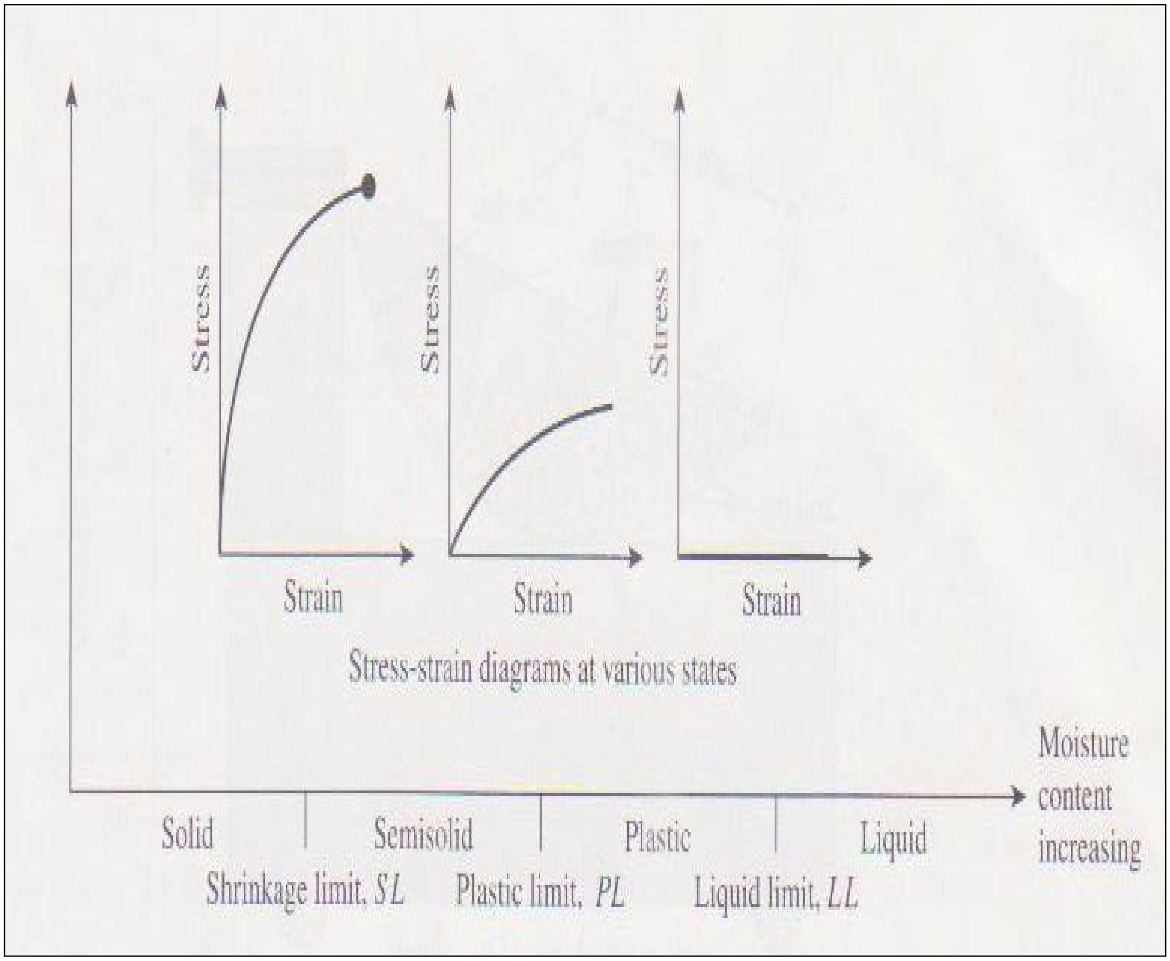


Figure 2.8: Atterberg Limits (Das, 2006).

2.4.1.1 Liquid Limit

Liquid limit is the moisture content at the point of transition from plastic to liquid state (Das, 2006). It is defined as the minimum water content at which the soil in the liquid state, but has a small shearing strength against flowing which can be measured by standard available means with references to the standard liquid limit device. The concept of the liquid limit is to keep adding water to soil until it flows, and measure the moisture content at that point by oven-drying a representative sample.

The liquid limit of a soil can be determined using the cone penetrometer or the Casagrande apparatus (BS 1377:1990). One of the major changes introduced by the 1975 British Standard (BS 1377) was that the preferred method of liquid limit testing became the cone penetrometer. This preference is reinforced in the revised 1990 British Standard which refers to the cone penetrometer as the ‘definitive method’. The cone penetrometer is considered a more satisfactory method than the alternative because it is essentially a static test which relies on the shear strength of the soil, whereas the alternative Casagrande cup method introduces dynamic effects. In the penetrometer test, the liquid limit of the soil is the moisture content at which an 80g, 30° cone sinks exactly 20mm into a cup of remoulded soil in a five (5) second period. At this moisture content the soil will be very soft (Das, 2006).

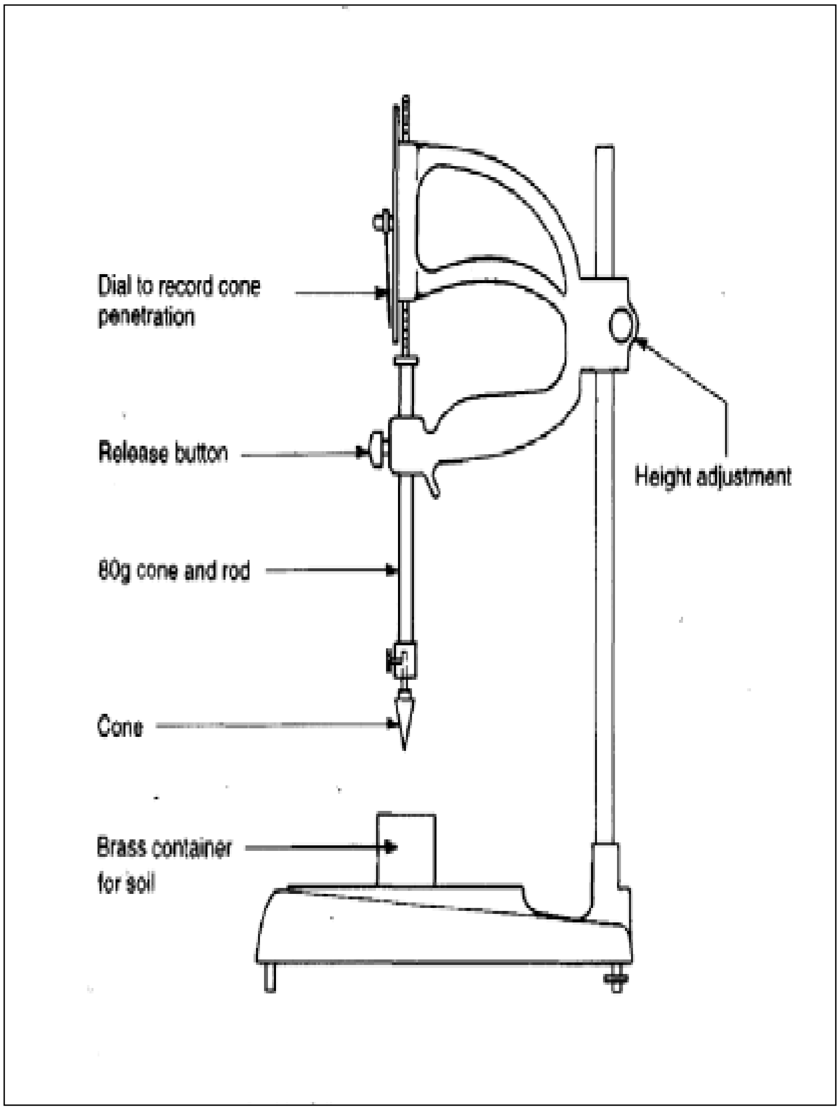


Figure 2.9: Cone Penetrometer Apparatus (www.allsoillabtesting-pdf.html).

2.4.1.2 Plastic Limit

Plastic limit is defined as the moisture content at which soil crumbles when rolled down into threads 3mm in diameter (Helwany, 2007). It is the moisture content at the point of transition from semisolid to plastic state. It is the lower limit of plastic stage of soil. The test is simple and performed by repeated rolling of ellipsoidal size of soil mass by hand on a ground glass plate.

	Soil B	Soil G	Soil W
Plastic limit (%)			
Mean	18	25	25
Range	13—24	18—36	20—39
S.D.	2.4	3.2	3.1
Coefficient of variation	13.1	12.8	12.7
Liquid limit (%) (Four-point method)			
Mean	34	69	67
Range	29—38	59—84	55—85
S.D.	2.4	5.2	5.3
Coefficient of variation	7.1	7.5	7.9

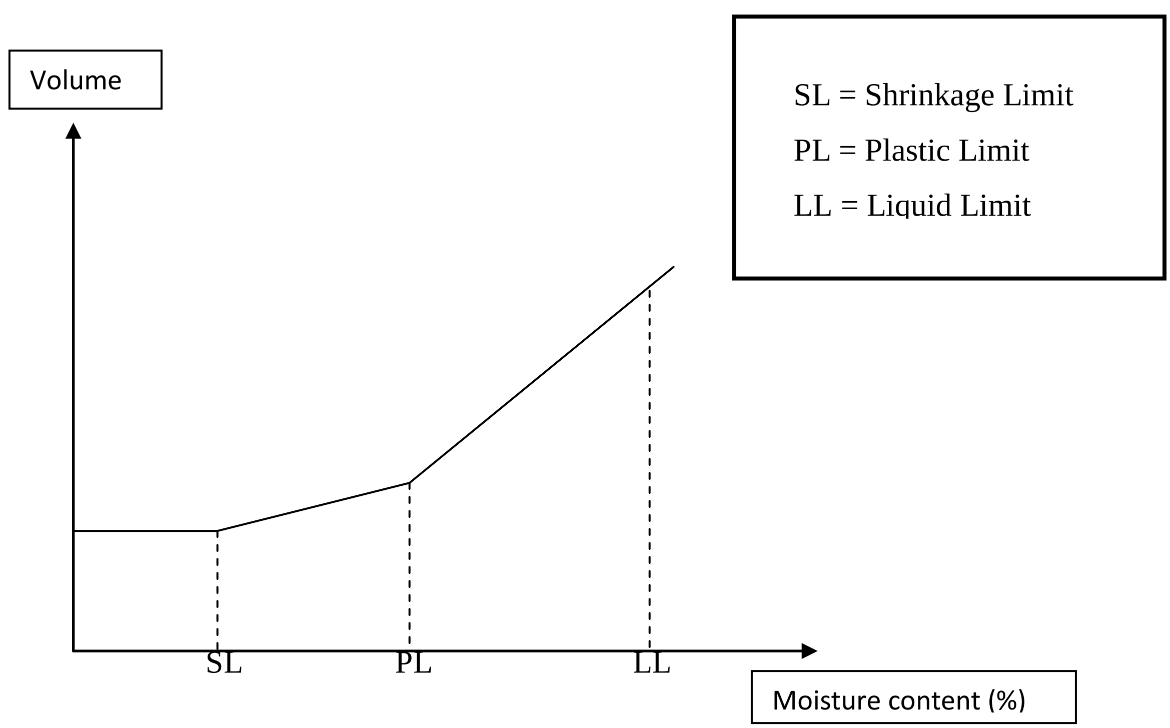
Figure 2.10: Results of Comparative Testing Programme (Aysen, 2005).

2.4.1.3 Plasticity Index

Das (2006) mention that, the plasticity index (PI) is the different between the liquid limit and plastic limit moisture content. Whereas the two limits that are used to define a PI are directly applicable to certain fields conditions, the plasticity index is mainly used to characterize the soil, where it is measure of cohesive properties. The plasticity index indicates the degree of surface chemical activity and hence the bonding properties of clay mineral in a soil. It is used along with the liquid limit and particle size gradation to classify soils according to their engineering behavior.

$$PI = LL - PL \tag{2.1}$$

The relationship between the plasticity index and the liquid limit of a soil gives us what is commonly referred to as the plasticity chart. It is linked back to soil classifications and is useful for linking the subjective classification system with the empirical evidence of the Atterberg tests.



2.5 Soil Structure

Soil structure is defined as the geometric arrangement of soil particles with respect to one another. It is an important factor which influences many soil properties such as permeability, compressibility and shear strength. There are also some factors that affect the structure which are the shape, size and mineralogical composition of soil particles, and the nature and composition of soil water. Generally, there are two groups of soils which are cohesionless and cohesive (Das, 2006).

2.5.1 Structures in Cohesionless Soil

Structures generally encountered in cohesionless soils consist of two major categories which are single-grained and honeycombed. In single-grained structures, soil particles are in stable positions, each particle in contact with the surrounding ones. The denseness of packing is influence by the shape and size distribution of soil particles and their relative position (Figure 2.11) thus giving a wide range of void ratios.

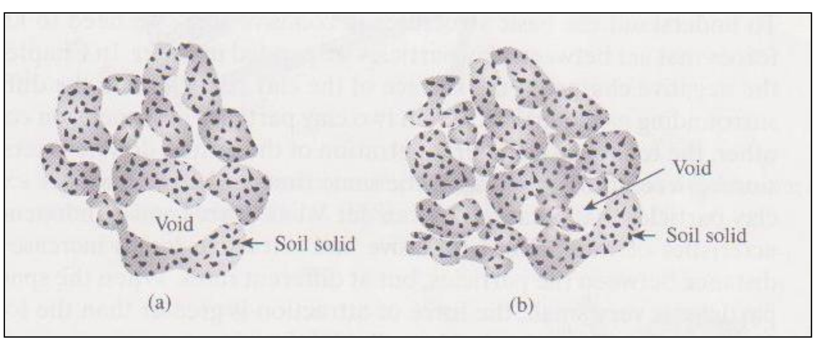


Figure 2.11: Single-grained Structure: (a) Loose; (b) Dense (Das, 2006).

The smaller-size particles may occupy the void spaces between the larger particles, thus the void ratio of soils are decreased compared with equal spheres. However, the irregularity in the particle shapes generally produces an increase in void ratio of soils. These two factors result of encountered of void ratio in real soils have approximately the same range as those obtained in equal spheres (Das, 2006).

In honeycombed structure (Figure 2.12), virtually fine sand and silt form small arches eith chains of particles. Soils that exhibit a honeycombed structure have large void ratio, and they can carry an ordinary static load. However, the structure breaks down when subjected to shock loading or under a heavy load which results in a large amount of settlement.

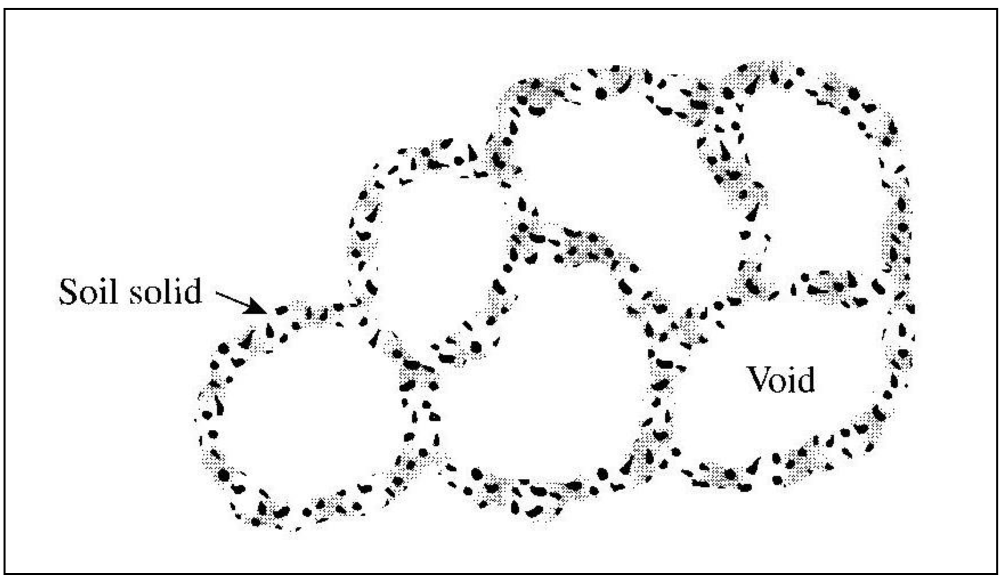


Figure 2.12: Honeycombed Structure (Das, 2006).

2.5.2 Structures in Cohesive Soils

Basic structure in cohesive soils is related to the types of force that act between clay particles suspended in water. When two clay particles in suspension come close to each other, the tendency for interpenetration of the diffuse double layers results in repulsion between particles. Both repulsive and attractive forces increase with decreasing distance between the particles, but different rates. When the spacing between the particles is very small, the force of attraction is greater than the force of repulsion. These are the forces treated by colloidal theories (Aysen, 2005).

According to Das (2006), if the clay particles initially dispersed in water come close to one another during random motion in suspension, they might aggregate into visible flocs with edge-to-face contact. In this instance, the particles are held together by electrostatic attraction of positively charge edges to negatively charged faces. This aggregation is known as flocculation. When the flocs become large, they settle under the force of gravity. The sediment formed in this manner has a flocculent structure.

2.6 Soil Sampling

Soil sample are divided into two categories which are undisturbed and disturbed. Undisturbed samples are obtained by techniques which aim at preserving the in-situ structure and water content of the soil (Craig, 2004). It is required mainly for shear strength and consolidation test. In boreholes, undisturbed samples can be obtained by withdrawing the boring tools and driving or pushing a sample tube into the soil at the bottom of the hole. Normally, the sampler is attached to a length of boring rod which can be lowered and raised by the cable of the percussion rig. It is impossible to obtain a sample that is completely undisturbed, no matter how careful the ground investigation might be.

A disturbed sample is one having the same particle size distribution as the in-situ soil but in which the soil structure has been significantly damaged. The water content may be different from that of the in situ soil. It is mainly used for soil classification test, visual classification and Atterberg limit (Ariffah, 2008).

All samples should be clearly labeled to show the project name, date, location, borehole number, depth and method of sampling. In addition, each sample should be given serial number. The sampling method used should be related to the quality of sample required (Ariffah, 2008).

Figure 2.13: Undisturbed Sample in a Sampler

Figure 2.14: Disturbed Sample

2.7 Shear Strength

According to Das (2001), the shear strength of soils is an important aspect in many foundation engineering problems such as the bearing capacity of shallow foundations and piles, the stability of the slopes of dams and embankments, and lateral earth pressure on retaining walls. It is defined as the shear resistance offered by the soil to overcome applied shear stresses. Shear strength is to soil as tensile strength to steel (Helwany, 2007).

Because it must support its own weight, shearing stresses exist everywhere within soil even though the soil is stable (Handy & Spangler, 2007). The maximum available shearing resistance is the shear strength, which enables soil to remain in place on a hillside or in an embankment or earth dam. Shearing strength also reduces soil pressure against retaining walls and is responsible for the bearing capacity of foundations and piles.

2.7.1 Shear Strength of Saturated Clays

If a saturated clay specimen is allowed to consolidate in the triaxial apparatus under a sequence of equal all-around pressure, sufficient time being allowed between successive increments to ensure that consolidation is complete, the relationship between void ratio and effective stress can be obtained. It is referred to isotropic consolidation.

According to Das (2006), the relationship between void ratio and effective stress depends on the stress history of the clay. The clay is said to be normally consolidated if the present effective stress is the maximum to which the clay has ever been subjected. On the other hand, the effective stress at some time in the past has been greater than the present value, the clay is said to be overconsolidated. The maximum value of effective stress in the past divided by the present value is defined as the overconsolidation ratio (OCR). A normally consolidated clay thus has an overconsolidation ratio of unity, an overconsolidated clay has an overconsolidation ratio greater than unity.

2.7.2 Unconsolidated-Undrained Triaxial Test

The unconsolidated – undrained (UU) triaxial test is one of the laboratory methods in determining the shear strength of soil. It is usually performed on undisturbed saturated sample of fine-grained soils (clay and silt) to measure undrained shear strength, $c_{u.}$ The soil specimen is not allowed to consolidate in stage one under the confining pressure applied. It is also not allowed drain during shearing. Identical soil specimens exhibit the same shear strength under different confining pressure, as indicated in Figure 2.12. When a fully saturated soil specimen is subjected to additional confining pressure (total stress), it generates an equal excess pore water pressure, which means that the additional confinement does not cause additional effective confining pressure. The effective stress principle indicates that the shear strength of the soil specimen depends on the effective confining pressure (Helwany, 2007).

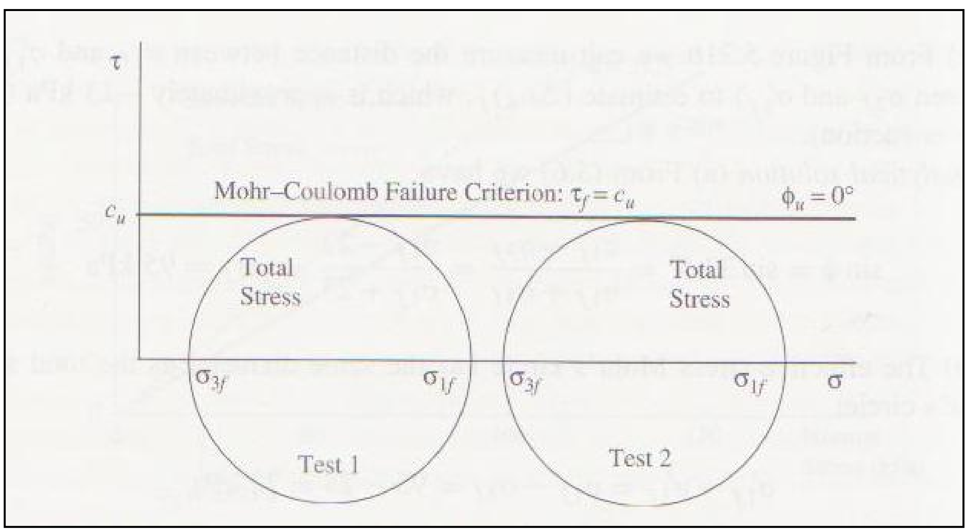


Figure 2.15: Typical Results of an Unconsolidated-Undrained Triaxial Test (Helwany, 2007).

2.7.3 Field Inspection Vane Test

Subsoil exploration is necessary to be done in soil before any construction can be made on it. This is to ensure the soil is specifically safe to be used later on. In this test, undisturbed samples which are mostly clay are required to be tested because vane shear tools can only relevant to be used for soft soil. The tools are small and easily damage by the hard surface of rocks and coarse soil. In other words, this test is best conducted to cohesive soils, which appear to be clay type of soil. The undisturbed and strength obtained are useful for evaluating the sensitivity of soil.

According to Das (2004), the test is done to explore the soil with depth in order to know the undrained shear strength of the soil. This test is the simplest, easiest and cheapest exploration soil test, regarding of what other test can offer, such as Deep Boring test. The way the results were produced is just based on the value of the rotation which specified in the arrow where it has a device that measures the required Torque. The undrained shear strength s_u of the clay can then be calculated by using the following equation, which assumes uniform end shear for a rectangular vane:

$$s_u = \frac{T_{\max}}{\pi(0.5\,D^2 H + 0.167 D^3)} \tag{2.2}$$

Where $T_{\max}$ = maximum torque

H = height of the vane

D = diameter of the vane

This method also not applicable in sands, gravels or other high permeability soils. Soil with higher permeability, in rapid shear, can dilate or collapse and generate negative or positive pore pressure. This test is often performed in drilled boreholes or with self-push or self-drilling or pushed methods. This method also applies to hand held vane shear tests performed at shallow depths, however, hand held equipment may be less accurate, because it may be more difficult to maintain rod stability and verticality.

Powrie (2004) stated that the height of vane is usually equal to twice the overall diameter which is H = 2B. For the used in weaker soils ($\tau_u < 50$ kPa), field vanes are generally 150 mm long while field vanes for use in stronger soils (50 kPa $< \tau_u <$ 100 kPa) are 100 mm long.

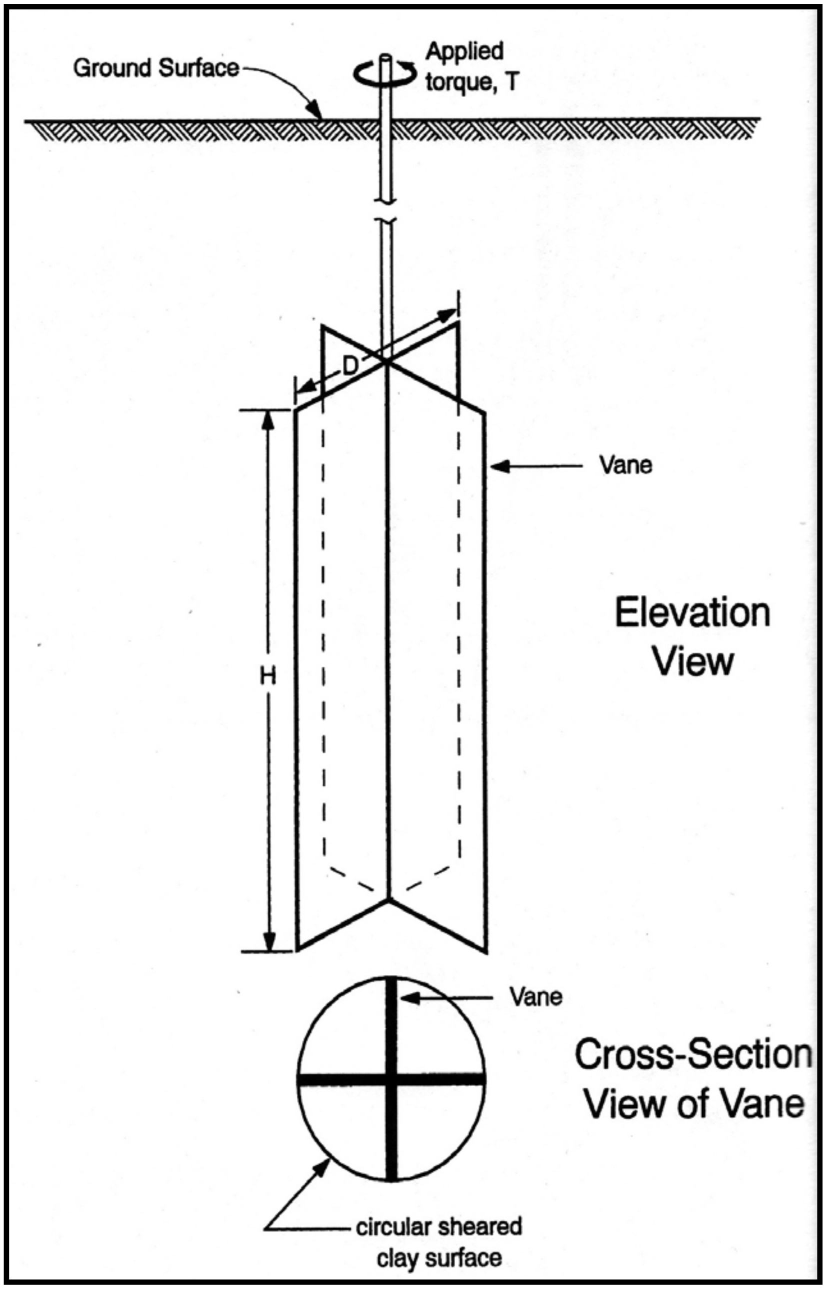


Figure 2.16: Diagram Illustrating the Field Vane Test (Day, 1999).

2.8 Clay Soil

According to Whitlow (2001), clay is classified as a fine soil with a particles size less than 0.002mm. Its visual identification is clay dry lumps can be broken but not powdered between the fingers, disintegrate under water but more slowly than silt, smooth to touch, exhibits plasticity but no dilatancy. Clay also sticks to finger and dried slowly and shrinks appreciably on drying usually showing cracks.

Clay is a mineral combination of hydrous aluminum, silicates, quartz, feldspar, carbonate, oxides, hydroxides, and organic materials. is produced from weathering process, hydrothermal activities, or settled as sediment. The Unified Soil Classification System (USCS) classifies clay soil as small particle soil that 50% pass sieve No. 200 Specification US (0.075mm) (Fauziah, 2007).

2.9 Silty Soil

The silty soil is slightly granular or silky to touch. It is also easy to disintegrate in water which is lumps dry quickly and can posses cohesion such as powdered easily between fingers. Silty soil will break into polyhedral fragment along fissures. The internal scale for spacing of discontinuities maybe used (Whitlow, 2001).

According to Handy & Spangler (2007), the binder fraction of a coarse grained soil with silty fines will exhibit plasticity characteristics similar to those of ML soils. The binder of a coarse grained soil with clayey fines will be similar to CI soils. The color of silt soil mostly dark or drab shades of gray or brown to nearly black indicate fine grained soils containing organic colloidal matter, whereas brighter colors including medium and light gray, olive green, brown red, yellow and white are generally associated with inorganic soils.

CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter is very important to show every stage of work from the case study in order to achieve the objectives which are to determine the basic properties and the shear strength of Temerloh silty clay. This project is mainly based on experimental works including soil preparation and laboratory testing. As well as, there were some preparation have done prior to the laboratory testing such as literature review, soil sample collection, and preparation calibration of the equipment. The methodology of this project has been summarized as shown in Figure 3.1.

The first step was discussion the title for this study with the supervisor. Then, the objective of this study was decided as approaching by the supervisor. After the title and objectives of this study have been identified, literature review was carry out to gather useful information of past research regarding this study which help to understand about the topic deeply.

The investigation and data collection were the main part of this study. It is started with the collection of the sample from the chosen location, which is Temerloh. Then, laboratory test such as sieve analysis, Atterberg limit and Uncosolidated Undrained test were conducted to obtain the result for the sample. The result was analysis and some discussions were made as a actual proof about this study.

Finally, the result obtain from this study are compared with the objectives and conclusion for the outcomes was made.

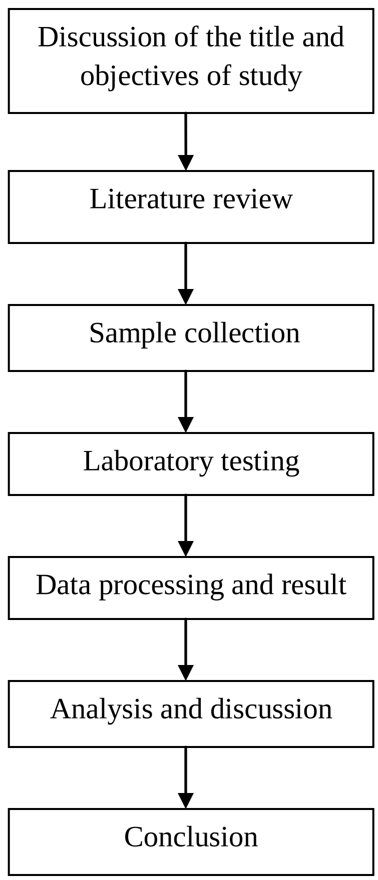


Figure 3.1: Flowchart of methodology

3.2 Objective of the Study

A goal or objective is a projected state of affairs that a person or a system plans or intends to achieve. It is a very important as a guide to complete given task successfully. Objective must be related with the project title so that the expected result can be achieved. Someone will eager to do their work properly if they know what is the objective to doing so. The objective of this study was decided by the discussion with the supervisor and referring to the all related information that gather from the site location.

3.3 Literature Review

Literature review is a process of finding and collecting information that related to this study. This stage is very important to gain knowledge and information and understand deeply about the study. It is contain the information about the past research which connected to what we want to do. The result that achieved from this study can be compared with all those previous result (Muzamir, 2006).

The sources of literature review can be finding from references books, previous journal, paper and also from the internet which is related with the title. In the literature review stage of this study, all information regarding the properties and shear strength of silty clay especially has been collected.

3.4 Sample Collection

After all information of the study was gathered from the literature review, the soil samples from the site at Temerloh are collected. The samples are taken from two different locations and which are at Kg. Buntut Pulau and Kg. Sanggang and at each locations, there are three samples taken from different points. This mean there are total 6 samples which will be take at the different spot.

There are two methods to collect the sample that has been used for two type of sample which are undisturbed samples and disturbed sample. For disturbed sample, the soil samples are taken by digging the ground using a hoe and then it will seal off in plastic bags. These samples are using to obtain the moisture content and classification of the soil. Meanwhile for the undisturbed sample, the samples are taken by dig up the ground surface and bury the soil sampler in one meter depth. The sampler then taking off and were cover by a layer of wax dilution. It is important to avoid the physical properties of soil sample from change especially its moisture content.

Then, those samples are brought to laboratory to be test by suitable laboratory test related to this study such as moisture content, sieve analysis, Atterberg limits and shear strength test.

3.5 Laboratory Testing

Laboratory testing is part of the physical survey. As an integral part of site investigation, the need for laboratory tests will often dictate the type and frequency of sample to be taken. In general, soil is tested in order to assess its variability and in order to obtain parameters for particular geotechnical calculations. These two distinct reasons for testing lead to very different testing programmes. Routine tests carried out to allow the soil on a site to be divided into groups should ideally be scheduled for an initial phase of testing.

3.5.1 Moisture Content Test

In this study, there are several laboratory tests that need to be conduct. The first laboratory testing that had been done in this project is moisture content test. The moisture content test needs to be done early to avoid the soil sample from affected by environment during storage process.

3.5.2 Particle Size Distribution

Particle size distribution then was conducted by using the same disturbed sample in moisture content in order to determine the classification of the soil. This test is conducted by using sieve set as the range of particles size and the percentage of particles are obtained.

3.5.3 Atterberg Limit

After that, the Atterberg limit test is conducted. Atterberg Limits are a series of tests which are used to give empirical information on the soils reaction to water. This information is of a qualitative nature and tells us the plastic limit, the liquid limit, the plasticity index and linear shrinkage of the materials. The Atterberg limits relate to the moisture contents of cohesive soils corresponding to empirical defined boundaries between states of consistency (liquid, plastic, solids) of the fraction of soil passing the 425 micron sieve. The liquid limit (LL) is arbitrarily defined as the water content, in percent, at which a part of soil in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove for a distance of 13 mm (1/2 inch) when subjected to 25 shocks from the cup being dropped 10 mm in a standard liquid limit apparatus operated at a rate of two shocks per second. The plastic limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3.2 mm (1/8 inch) diameter threads without crumbling.

3.5.4 Shear Strength Test

Lastly, the shear strength test is done in order to determine the strength of soil sample. There are many test can be conducted to obtain shear strength parameter but in this study, the Unconsolidated Undrained Test has been chosen. In this test method, the compressive strength of a soil is determined in terms of the total stress, therefore, the resulting strength depends on the pressure developed in the pore fluid during loading. In this test method, fluid flow is not permitted from or into the soil specimen as the load is applied, therefore the resulting pore pressure, and hence strength, differs from that developed in the case where drainage can occur.

3.6 Analysis and Discussion

The result that obtain from the laboratory testing will be analyze about the problem encounter during the test is handled. The result also will be compare with the previous study to check whether both studies has differential or not. Some discussion is made with the supervisor to make and produce the suitable solution if there are error occur (Muzamir, 2006).

3.7 Conclusion and Recommendation

This is the last stage of the research whereby conclusion is made based on the analysis carried out. The conclusion is made by considering the objective of this study. If the study achieved the objectives, then it can be conclude as successful. Then, some recommendation of research is propose to prevent same problem happen and improve future research.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Introduction

This chapter discuss and analyze the various type of data obtained in order to determine the basic properties and shear strength of some silty clay sample in Temerloh district. The data are gathered from three different locations in Temerloh so that the result can be compared in order to get the conclusion.

Results are produced from both in-situ and laboratory testing based on the standard and method of BS 1377:1990. These data was analyzed accordingly so that analysis of result could be done. It is originally produce from the taken sample at site as no supplementary source in use. Results obtained were presented and tabulated in tables, figures and graph so that it can easily examine and analyzed.

After discussing with the supervisor, the areas whereby the samples are taken were intentionally chosen nearby the river as the characteristic of the soil potentially matched with the required sample. The sample then was taken after the initial observation was satisfied as the conditions of the soil at the sites are found as exceptional soft. The colour of the sample also show that it is a clayey type as it was grey in colour. The undisturbed samples were taken using the 1m tube sampler and then were covered with a layer of diluted wax to prevent it from bare to environment. Meanwhile, the disturbed samples were taken by digging the ground using a hoe and then it will seal off in plastic bags.

4.2 Field Vane Shear Test

The field vane shear test is the most widely used method for measuring the undrained shear strength of soft to stiff clays. It is enables quick and easy determination of undrained shear strength of clay. The tools are small and easily damage by the hard surface of rocks and coarse soil. In other words, this test is best conducted to cohesive soils, which appear to be clay type of soil. The undisturbed and strength obtained are useful for evaluating the sensitivity of soil. It is not applicable for sandy soils which may allow drainage during the test.

The result of Field Vane Shear test will be affected by:

i. the rate of rotation of the vane
ii. the time elapse between the insertion of the vane
iii. the height to diameter ratio of the vane blade
iv. the drainage condition around the vane
v. strength anisotropy and progressive failure of the soil around the vane

In this test, the results were produced is just based on the value of the rotation which specified in the arrow where it has a device that measures the required Torque. The result is shown in Table 4.1 below.

Table 4.1: The Undrained Shear Strength of Soil at Temerloh.

	Blade 1		Blade 2		Blade 3		Blade 4	
Site	**A**	**B**	**A**	**B**	**A**	**B**	**A**	**B**
Height of vane, H (cm)	2		3		4		5	
Diameter of vane, D (cm)	1		1.5		2		2.5	
Deep of penetration at 5cm (kPA)	2	2	8	7	18	14	22	24
Deep of penetration at 10cm (kPA)	6	3	10	10	20	16	29	29
Deep of penetration at 15cm (kPA)	8	6	14	11	23	21	44	34

A torque applied to rotate the vanes is related to the shear strength of the soil. By referring to the results above, there is linearity performed by the shear strength result. Table 4.1 shown that, supposedly, shear strength will increase with depth. Means, the deeper vane shear test is conducted, the greater the value of shear strength of the soil. Soil is getting more compacted at the bottom of the ground surface level. Shear is always expected to happen at low strength and low compacted soil, especially near the surface of soil. That is why, the value of shear strength is expected to be low at the beginning of the depth, but will increase as depth go further and deeper. Deeper depth of soil is always more compacted than the nearer surface of soil. This is supported by the fact that the deeper soils are due to support more particles of its own mass.

Even though the blades are different at every location, by right, the value should have been linearity increased with depth. From observation, when the diameter of blade are getting bigger, the value of the shear strength also getting increased. So, there is relationship between diameter of the vane and the shear strength value produced.

The more compacted soil will make the vane blade harder to oscillate. The harder the blade to oscillate, the greater or bigger the value of shear strength produced. More compacted clay soils have it particles bonded tightly to each other. To disturb the bond of the particles, will require greater efforts and energy. The greater energy required will relates to the shear strength results produced which means shear strength will increase. Supposedtedly, at the depth of 15cm, the blade will experience greater efforts to oscillate and at the end, will produce higher value of shear strength.

Based on the result shown, the undrained shear strength value at both site A and B are low which is between 2kPa until 44kPa and between 2kPa until 34kPa. It shows that the soil specimen at those sites is quite soft and can be categorized as the soft clay with the low compacted condition. From the first observation through visual identification and when it was rubbing with hand, the condition of soil clarify that it is a soft clay type. That is why the site is still considered as clay type and the investigation was proceed.

There are some possible errors that might have occurred in this test which have disturbed the accuracy of the reading taken. Firstly, the equipment might not in the good condition. The extension of rod might have loose and not joint properly. It could agitated the reading taken because the reading is depends on the rotation of the rod. If the rod is loose, the value might be different from the real one. Besides, the instruments might have not stood vertically straight above the ground. The instrument need to be keep vertically while taking the measurements. It is to prevent the equipment from twist the moment it in the ground and to ensure that the result is not bothers.

4.3 Soil Index Properties

4.3.1 Natural Moisture Content

Moisture content is an important characteristic in clay. Moisture content is one of the required characteristic in for the purpose of a detail classification. Clay and silt usually possess high water content than any other soil types. The natural moisture content of clay in South East Asia is usually in a range from 50% to 100% (Cox, 1968). As a precaution step, the moisture content of a soil needs to be taken as soon as the sample arrived in the laboratory. It is very important to prevent the loss of moisture content to surrounding.

From the data collected, the moisture content values obtain from the two (2) sites as shown in the table below. For the site A, the average moisture content that consist form all the three samples is only 34.52%. It is very low if compare to the standard range of a moisture content in a silty clay. Meanwhile, the average moisture content of site B is 50.8% respectively. It was higher than site A and closer to the range of standard range of natural moisture content of silty clay that which recommended by Cox (1968).

Table 4.3 show the result of moisture content by some past researcher for the same type of soft clay in the area of Pahang state. By comparing to their result, it is noticeable that the result for all three sites in this study at Temerloh is considered as high. So it is show that the natural moisture content of some soft clay type in some area in Pahang is low and the result of moisture content in this study is fairly acceptable.

The lower result of this natural moisture content also can be related to the result of sieve analysis. Based on the particle size analysis result, the percentage of clay particle is not so dominant. Eventhough Site A and B recorded clay as the particle with the highest percentage, but there are just have small margin with the percentage of sand particle which is come second. This percentage of sand particle is in fact, has big influence for the value of natural moisture content because sand particle has high porosity which means it cannot hold much water. That is why the percentage of moisture is actually low.

There are some factors that could affect the accuracy of the result taken which are mostly come from weather, depth of sample taken and also moisture loss to environment. The moisture content of the sample was absolute affected by condition of the site. This could be happen because of the weather at Temerloh which very hot and not received any rain during the sample was taken. Besides, the trip to the site from laboratory is very far which is about 80km. This situation has completely affected the state and water content in the sample because it might have disturbed by the condition of the surrounding during that trip. According to Muzamir (2006), moisture content of clay is decrease with the depth. This is another factor which has affected the natural moisture content of the soil.

Table 4.2: Average Natural Moisture Content in Each Site at Temerloh.

Site	Boreholes	Natural Moisture Content (%)	Average (%)
Site A	BH1	31.88	34.52
	BH2	37.30	
	BH3	34.38	
Site B	BH1	50.82	50.8
	BH2	51.45	
	BH3	50.13	

Table 4.3: Natural Moisture Content by Past Researchers for Clay Soil.

Researchers	Locations	Depth	Type Of Soil	Moisture Content (%)
Fauziah (2008)	Pekan, Pahang	1.0m	Clay	26
Noorazura (2008)	Pekan, Pahang	1.0m	Clay	33
Hawa (2008)	Temerloh, Pahang	1.0m	Clay	26

4.3.2 Atterberg Limits

4.3.2.1 Liquid Limit

Liquid limit (LL) is the moisture content at 20mm cone penetration. The liquid limit results for both sites are shown at Table 4.4 below. The range of liquid limit for both sites is in between 44% to 59%. For site A, the average of liquid limit is 51.1% as shown in Figure 4.1 below. Meanwhile the average of liquid limit for site B is 54.9% as shown in Figure 4.2. Liquid limit increases with the increase of clay content. The characteristic of clay particles which tend to pull or adsorb water to soil surface particle making the liquid limit to be much higher.

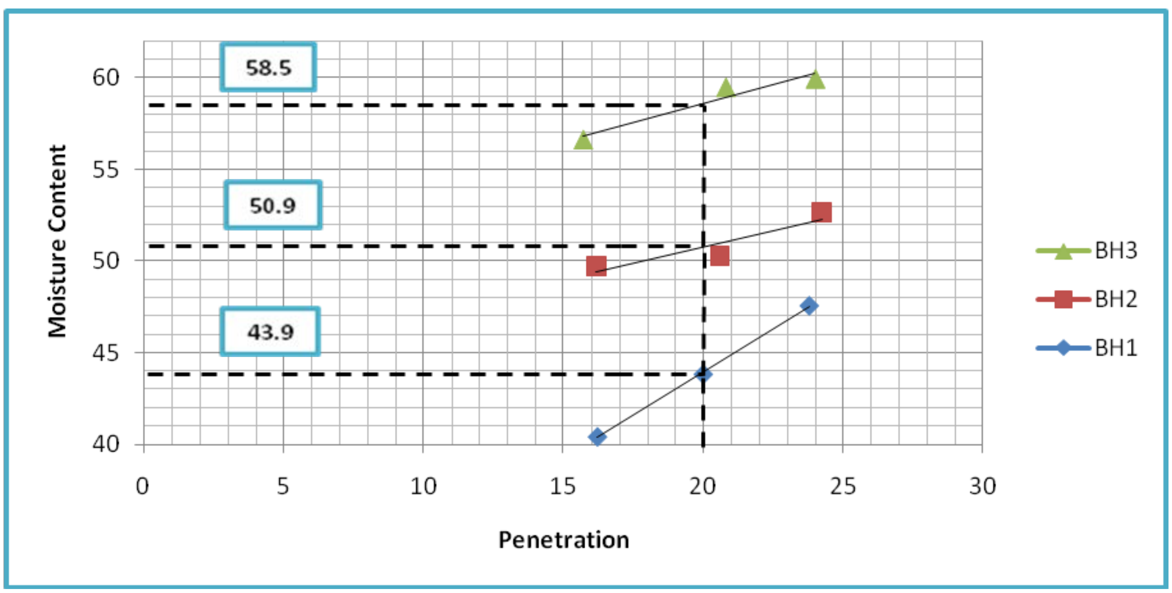


Figure 4.1: Graph Moisture Content against Penetration to Determine Liquid Limit for Site A.

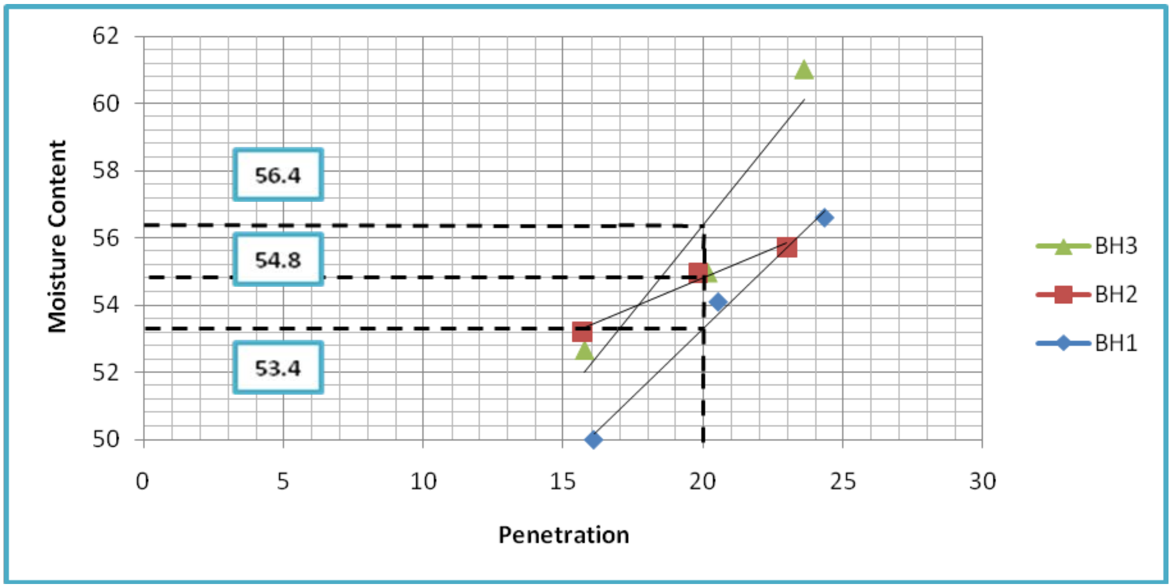


Figure 4.2: Graph Moisture Content against Penetration to Determine Liquid Limit for Site B.

Table 4.4: Average of Liquid Limits at Temerloh.

Site	Boreholes	Liquid Limit, LL (%)	Average (%)
Site A	BH1	43.9	51.10
	BH2	50.9	
	BH3	58.5	
Site B	BH1	53.4	54.87
	BH2	54.8	
	BH3	56.4	

4.3.2.2 Plastic Limit

The plastic limit (PL) is the moisture content at the lower limit of the plastic range. It is the water content where soil starts to exhibit plastic behavior. A thread of soil is at its plastic limit when it is rolled to a diameter of 3 mm and crumbles. To improve consistency, a 3mm diameter rod is often used to gauge the thickness of the thread when conducting the test.

From the Table 4.5 below, the range of plastic limit in this study is between 18% to 30%. For site A, the average plastic limit is 25.31%. Meanwhile for site C, the average plastic limit is 28.78%.

Table 4.5: Average Plastic Limit in each site at Temerloh.

Site	Boreholes	Plastic Limit, PL (%)	Average (%)
Site A	BH1	18.85	25.31
	BH2	29.58	
	BH3	27.50	
Site B	BH1	29.90	28.78
	BH2	28.46	
	BH3	27.97	

The shear strength of a clay soil can be indicating by the effect of liquid and plastic limit tests. The shear strength of the clay at the plastic limit is about 70 times that at the liquid limit (Whyte, 1982).

There are some factors that might have contributed to the error which occur in this Liquid Limit (LL) and Plastic Limit (PL) and affected the accuracy of the result. One of the most influence factors is the loss of weight during mixing it with water. Some of the weight of that soil has lost during mixing process when the soil has touched by the finger. Actually, the heat which comes from hand temperature can absorb the moisture content of the soil and affect the result of this test.

4.3.2.3 Plasticity Index

The plasticity index (PI) is a measure of the plasticity of a soil. The plasticity index is the size of the range of water contents where the soil exhibits plastic properties. It is the numerical difference between the liquid and plastic limit moisture contents. According to Handy and Spangler (2007), plasticity index is mainly used to characterize a soil, where it is a measure of cohesive properties. The plasticity index indicates the degree of surface chemical activity and the bonding properties of clay minerals in a soil. It is used along with the liquid limit to classify soils according their engineering behavior.

A large plasticity index indicates low shear strength and can be calculated from this formula;

$$PI = LL - PL \tag{4.1}$$

Where PI = Plasticity Index

LL = Liquid Limit

PL = Plastic Limit

Table 4.6: Atterberg limits in Every Site at Temerloh.

Site	Boreholes	Atterberg Limits (%)		Average (%)		
		LL	**PL**	**LL**	**PL**	**PI**
Site A	BH1	43.9	34.85	51.10	25.31	25.79
	BH2	50.9	45.58			
	BH3	58.5	43.50			
Site B	BH1	53.4	40.90	54.87	28.78	26.09
	BH2	54.8	39.46			
	BH3	56.4	38.97			

Table 4.6 above has shown the relationship between Liquid Limit, Plastic Limit and also Plasticity Index at every site. Form the table, the highest plasticity index is at site B which is 26.09% while the lowest value of plasticity index is at site A which is 25.79%. Raj (2008) has stated that the plasticity index represents the range of water content over which a soil is plastic. The greater the plasticity index, the higher will be the attraction between the particles of the soil and the greater the plasticity of the soil.

4.3.2.4 Plasticity Chart

According to Day (1999), Casagrande (1932, 1948) has developed the plasticity chart by using the Atterberg limits. It is used in the Unified Soil Classification System to classify soils. Figure 4.4 below show the result of plasticity chart of the liquid limit (LL) versus the plasticity index (PI) for the study location at Temerloh.

A-line that approximately parallels the PI versus LL plot for the particular soil groups is called the A-line, which was proposed by A. Casagrande. It is used to separate clays, which plot above the A-line and silts, which plot below the A-line. It is defined as:

$$PI = 0.73\,(LL - 20) \tag{4.2}$$

Where PI = Plasticity Index

LL = Liquid Limit

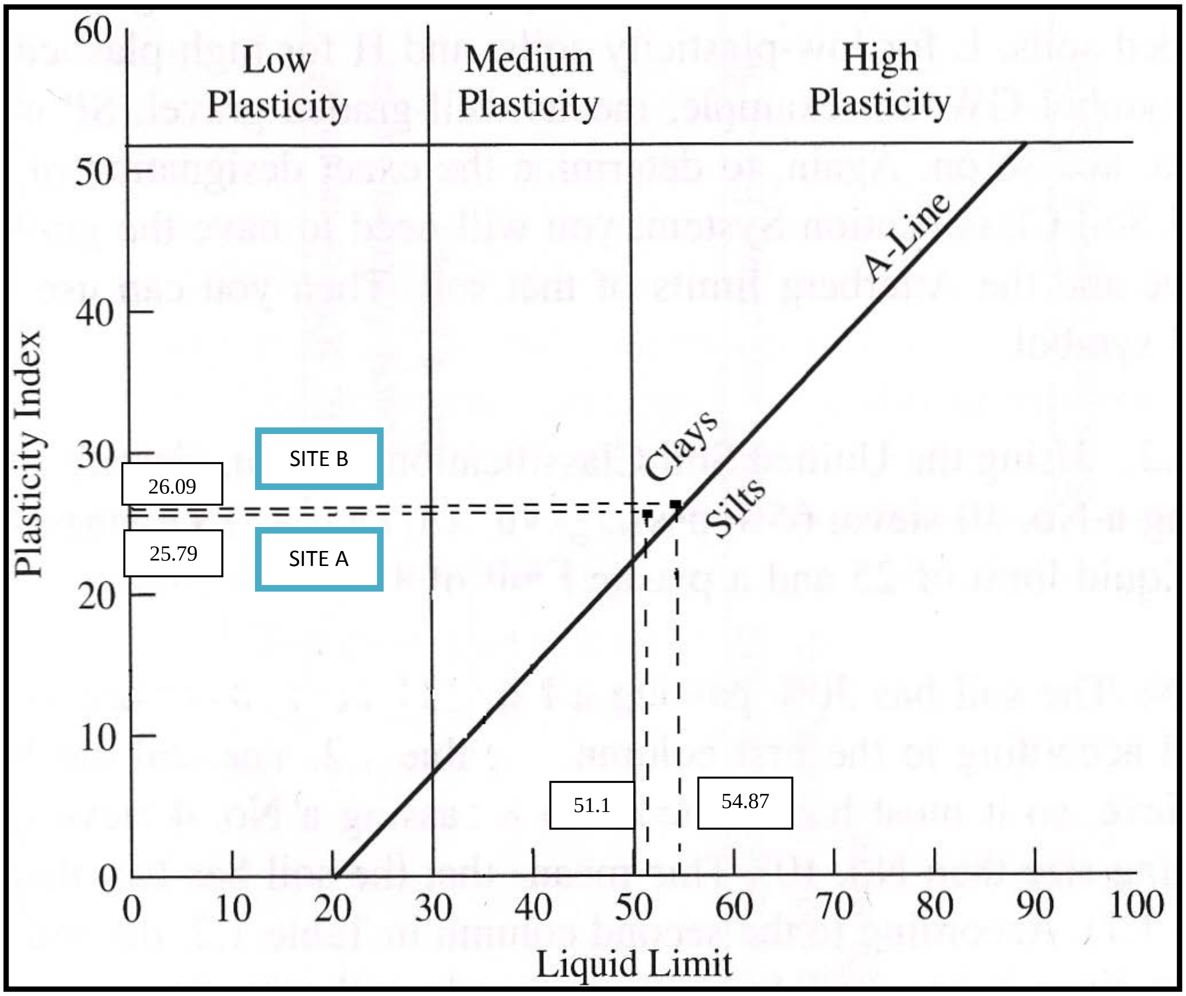


Figure 4.3: Correlation between Plasticity Index and Liquid Limit.

The plasticity chart above was shown the correlation between plasticity index (PI) and liquid limit (LL) for all the study location at Temerloh. From that plasticity chart, it is clearly showed that the type of soil at Temerloh is consisting of clay particle. But the content of clay particle and silts is differing by only slender amount. It is also can be stated that those sites contain of silty clay soil.

4.4 Particle Size Analysis

Soil particle distribution data is obtained from sieve analysis that had been done at laboratory. The particle size distribution of the silty fractions of soil is of interest and these sizes lie below 0.063 mm. The sieved soils are following the standard presented by American Association of State Highway and Transportation Officials (AASHTO).

Table 4.7 below show the result of particle size distribution at every location. For both site A and B, the percentage of silt and clay is dominant which is 51% and 56% respectively compare to other soil which are sand and gravel. It is proved that these two sites contain soil particle of clay type. The condition of the specimen especially colour may prove that it is clay type of soil. Depth of the samples taken could also become one of the reasons that could affect the percentage of the soil particle size because clay could genuinely be more dominant with depth.

Table 4.8 show the particle size distribution by past researcher at some location at Temerloh. From the result from Ariffah (2008), it can be reveal that the study location is contain of sand as the most dominant particle by slightly different with the clay which is 51% to 47% respectively. Meanwhile from the result that show by Hawa (2008), it is clearly stated that clay is the most dominant particle found at the study location with the massive different with other type of soil like sand and gravel. Those results clarify that some location at Temerloh is totally dominant by clay and even if it is not be the highest or major element, it is still have high percentage and fewer only by small amount of particle.

Table 4.7: Particle Size Analysis in Each Site at Temerloh.

Site	Boreholes	Particle Size Distributions (%)			Average (%)		
		Gravel	Sand	Clay	Gravel	Sand	Silt and Clay
Site A	BH1	2	48	50	2	47	51
	BH2	2	44	54			
	BH3	1	48	51			
Site B	BH1	1	40	59	2	42	56
	BH2	2	41	57			
	BH3	3	45	52			

Table 4.8: Particle Size Analysis by Past Researchers at Temerloh.

Researchers	Location	Depth	Soil Particle Distribution (%)		
			Clay	Sand	Gravel
Ariffah (2008)	Temerloh, Pahang	1.0m	47	51	2
Hawa (2008)	Temerloh, Pahang	1.0m	72	21	2

There are some problems that might happen during the experiment and affect the result. One of the factors is because of the loss of dusts when sieve shaker was shaken and which might have change the weight of the specimen. It is maybe occur during the sample is put into the sieve set. The dust from the sample might have flown away to the air especially when it was hit by wind which comes from fan in the laboratory. Besides, the soil sample is might not have been well dried and clean properly. The sample has contained some unnecessary ingredient like leaf or grass. This needless particle needs to be throwing away and the sieve set also must be clean properly to provide the better result.

4.5 Shear Strength Test

The Unconsolidated Undrained Test (UU) is the test of Triaxial Test which has been performs to determine the shear strength of the specimen that had been taken. This test is very important to prove and achieve the objective of this case study which is to determine the shear strength of Temerloh silty clay soil. Das (2007) has stated that, the strength of the clay particle is generally less than 40kPa. But for silty clay particle, it is might be lesser that that.

Table 4.9 below has shown the result of shear strength of every specimen at each site. Two (2) undisturbed samples were testing for each site. For site A, the two samples have been tested with the pressure of 20kPa and 60kPa respectively. From mohr circles which shown in Figure 4.4 below, the first specimen has produces the shear strength of 37.48kPa while for second mohr circles, the shear strength produced was 38.33kPa.

Table 4.9: Shear Strength of Each Study Location at Temerloh.

Site	Sample	Cell Pressure, (kPa)	Shear Strength, (kPa)
Site A	1	20	37.48
	2	60	38.33
Site B	1	40	26.23
	2	80	26.58

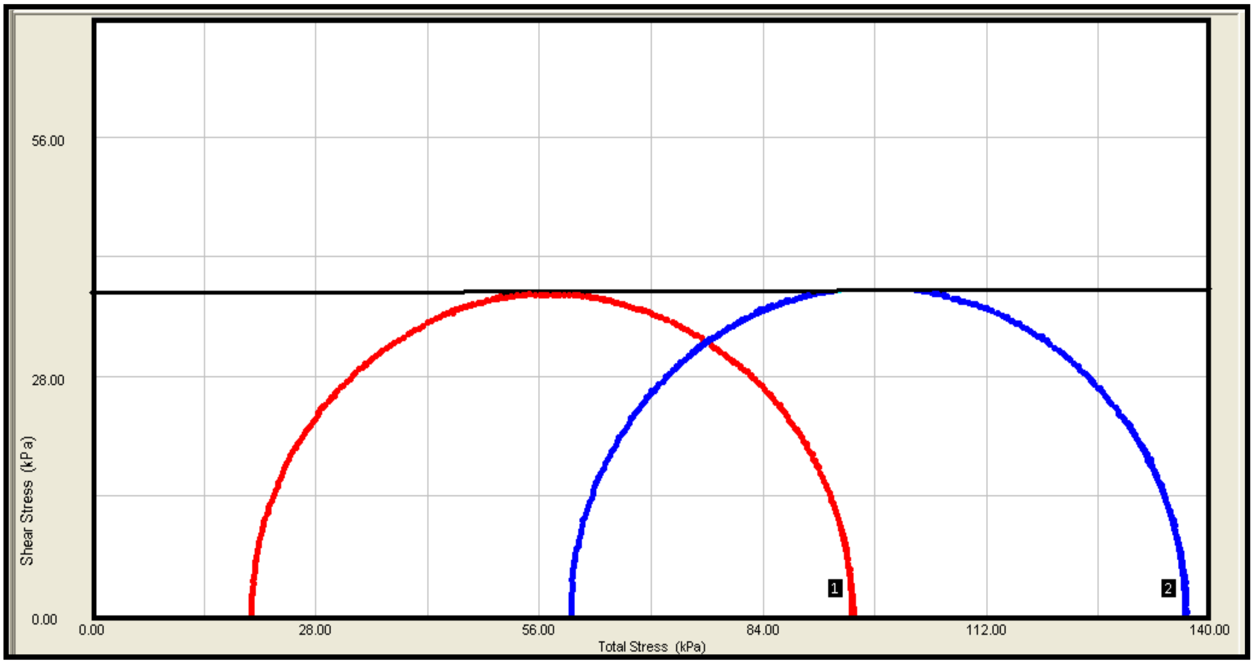


Figure 4.4: **Result of Total Triaxial Mohr Circle for Site A.**

Then for Site B, the value of cell pressure which has been used is different with site A which are 40kPa and 80kPa respectively. For the first sample with the 40kPa cell pressure, the value of shear strength is 26.23kPa, which the lowest shear strength that produce compares to other site. Meanwhile for the second sample, the shear strength is 26.58kPa. The value of shear strength for this site B also has small different which make the mohr circle look almost in same shape. The result of mohr circle for site C is shown in Figure 4.5 below.

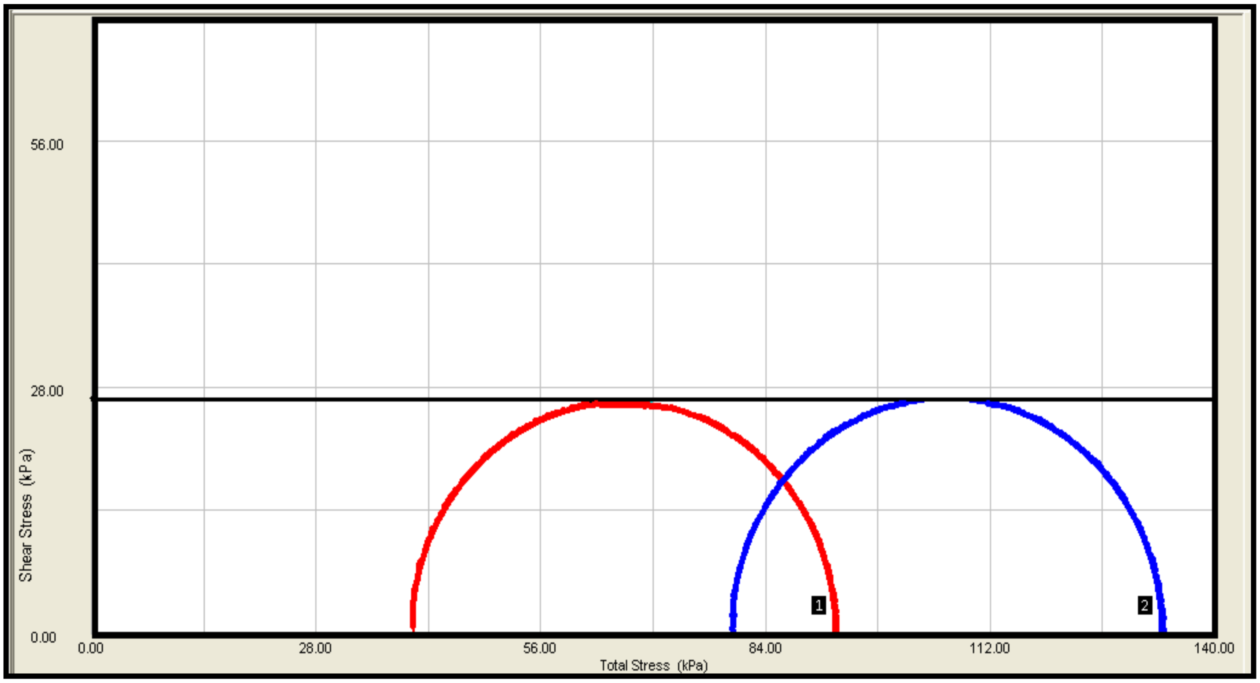


Figure 4.5: Result of Total Triaxial Mohr Circle for Site B.

Table 4.10: Relationship of Consistency and Unconfined Compression Strength of Clays (Das, 2007).

Consistency	**Unconfined Compression Strength of Clays (kN/m^2)**
Very soft	0 - 24
Soft	24 - 48
Medium	48 - 96
Stiff	96 - 192
Very stiff	192 - 383
Hard	> 383

Table 4.10 above show the relationship between consistency and unconfined compression strength of clays which is stated by Das (2007). Based on the result in this study, the value of shear strength is in the range between 26kN/m^2 to 38 kN/m^2. Based on table above, it is reveal that the shear strength of the clay sample at Temerloh can be categorized as soft. So the result is satisfactory because silty clay also is can be categorized in the soft clay soil.

In this experiment, there are several factors which have occurred and affected the accuracy of the result. One of the main factors that could affect the shear strength of the sample is the duration of the storing process. We might have store the sample too long inside the thin sampler tube. This situation has completely affected the state and water content in the sample because it might have disturbed by the condition of the surrounding during the duration. So as the solution, every thin sampler tube have been pour with the dilution wax in order to prevent its moisture content from lose to surrounding. If the moisture content has loose, the sample might have been dry and easy to brittle. The error also can come from the technical problem during the shear strength test was conducted. Sometimes, the error happens during the setting process of Unconsolidated Undrained (UU) test equipment. Some of the equipment did not work properly. So it might have also disturbed the accuracy of the reading taken.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

From the result obtained in this study, the basic properties and shear strength for some specimen of Temerloh silty clay has been determined. There are 6 samples which has been taken from three sites at Temerloh. The basic properties such as moisture content, particle size distribution and also atterberg limit for every samples has been obtained. Meanwhile an Unconsolidated Undrained Test was performed in order to determine the shear strength of the undisturbed samples at each site. All the experiment has been performed based on standard and method of BS 1377:1990.

From the analysis that had been done in previous chapter, some conclusion can been made such as:

i. The undrained shear strength for site A is between 2kPa to 44kPa and for site B is between 2kPa to 34kPa which shows that the soil specimen at those sites is soft.

ii. The moisture content of some sample taken for every site in Temerloh is between 34% to 51%. It slightly passes the range of standard range of moisture content in silty clay which is between 50% to 100%.

iii. The value of moisture content at every site might have been low because of some factors which are depth of sample taken and also moisture loss to environment.

iv. The liquid limit, plastic limit, and plasticity index with moisture content show that all the parameters increase with the increase of moisture content.

v. The highest plasticity index recorded at site A which is 15.09% and the lowest is recorded at site B which is 26.09%. A large plasticity index indicates low shear strength.

vi. The lower value of liquid limit and plastic limit might have been influence by some error during the experiment in process such as loss of weight during mixing it with water.

vii. The particle size analysis at both site A and B is dominant by clay which is 51% and 56% respectively.

viii. Depth of the samples taken could also become one of the reasons that could affect the percentage of the soil particle size because clay could genuinely be more dominant with depth.

ix. Shear strength value for this study is between 26kN/m^2 to 38 kN/m^2 and it can be classified as soft clay soil based on Das theory.

5.2 Recommendation for Future Study

In a similar research, the following recommendations and suggestion should be taken into consideration:

i. The disturbed samples should be store properly and the time period of samples storage should not be too long because it can affect the soil properties especially moisture content.

ii. The undisturbed and disturbed samples should be taken as many as we can in order to make it as back up if any problem happen during soil testing.

iii. To get more accuracy in the result, the sample should be taken in deeper depth to the ground with the uses of suitable equipment.

iv. The study of different range of soils will give more understanding for the behavior of the various kind of soil.

v. The characteristic of the soil that need to be analysis should be understand properly before any test is performed.

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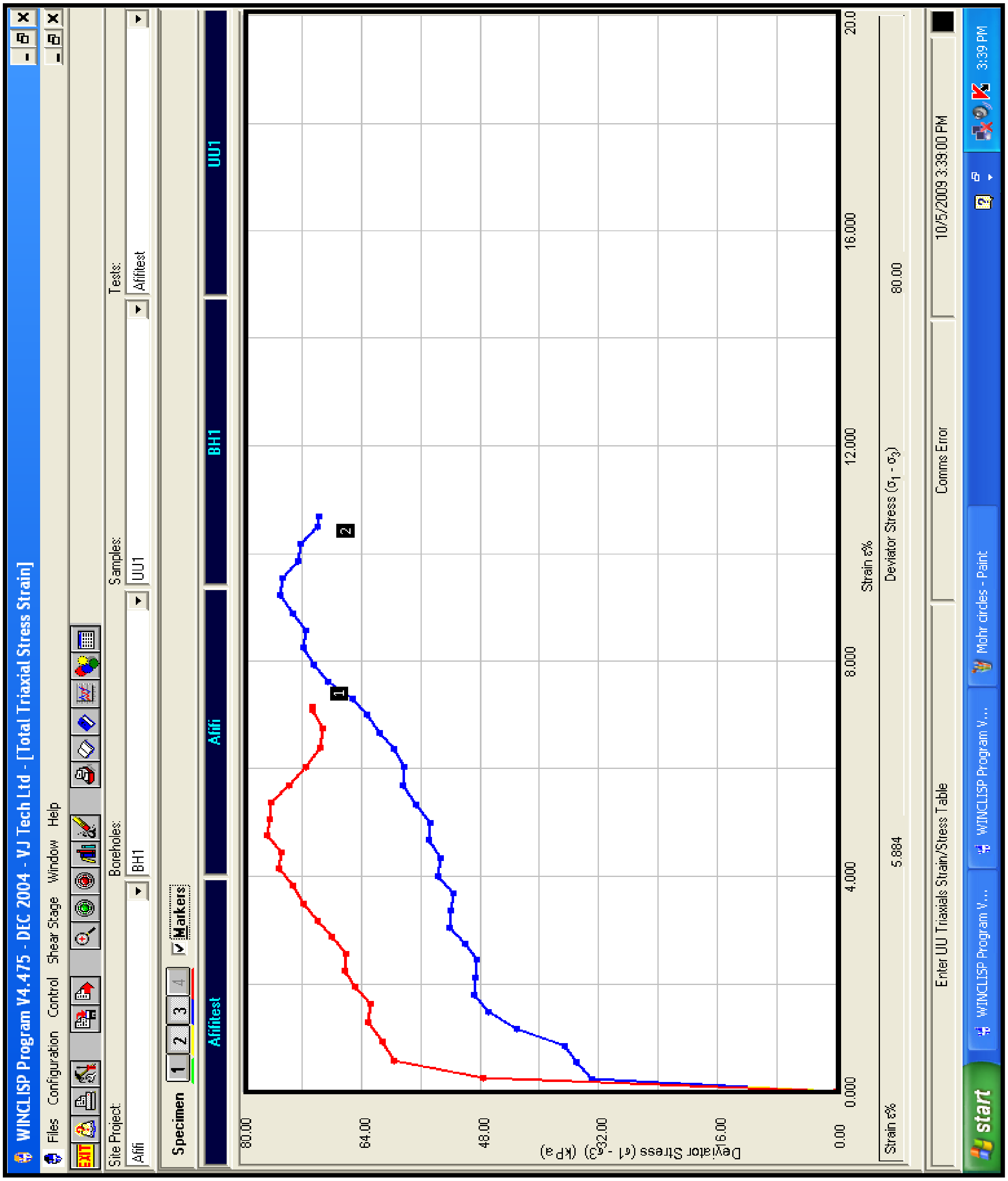
APPENDIX

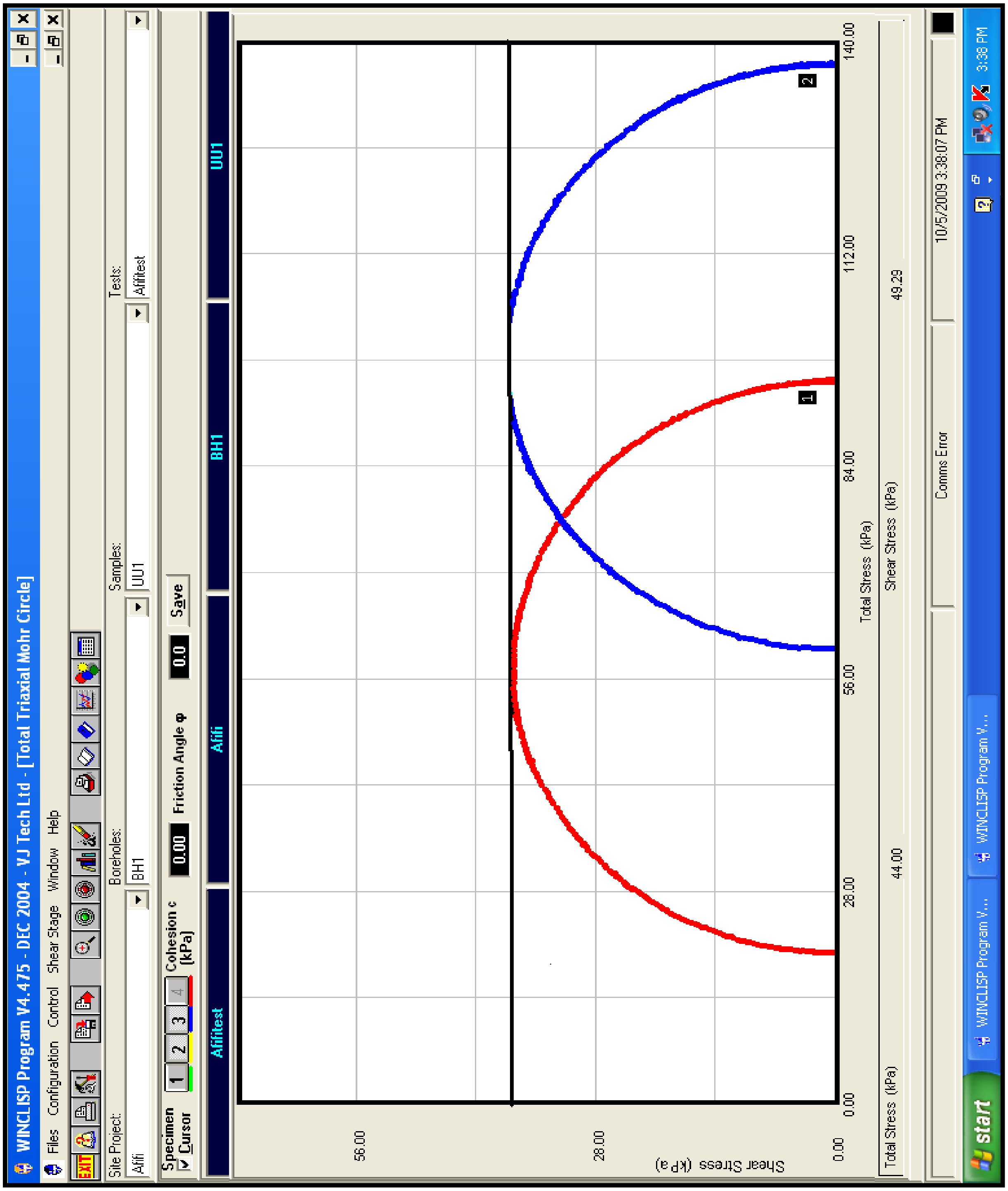
APPENDIX A

GRAPH OF TOTAL TRIAXIAL STRESS-STRAIN OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE A.



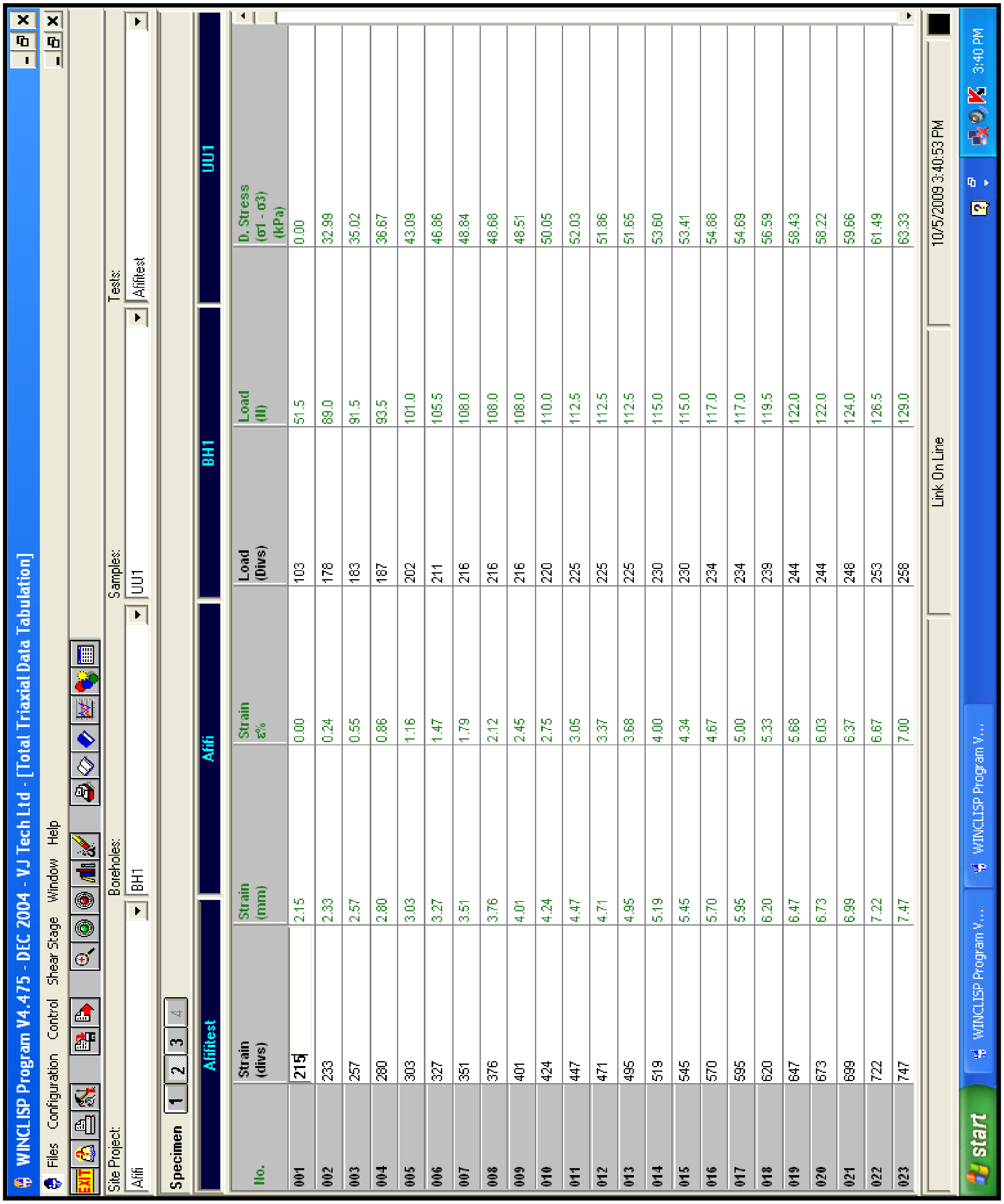
APPENDIX B

TOTAL TRIAXIAL MOHR CIRCLE OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE A.



APPENDIX C

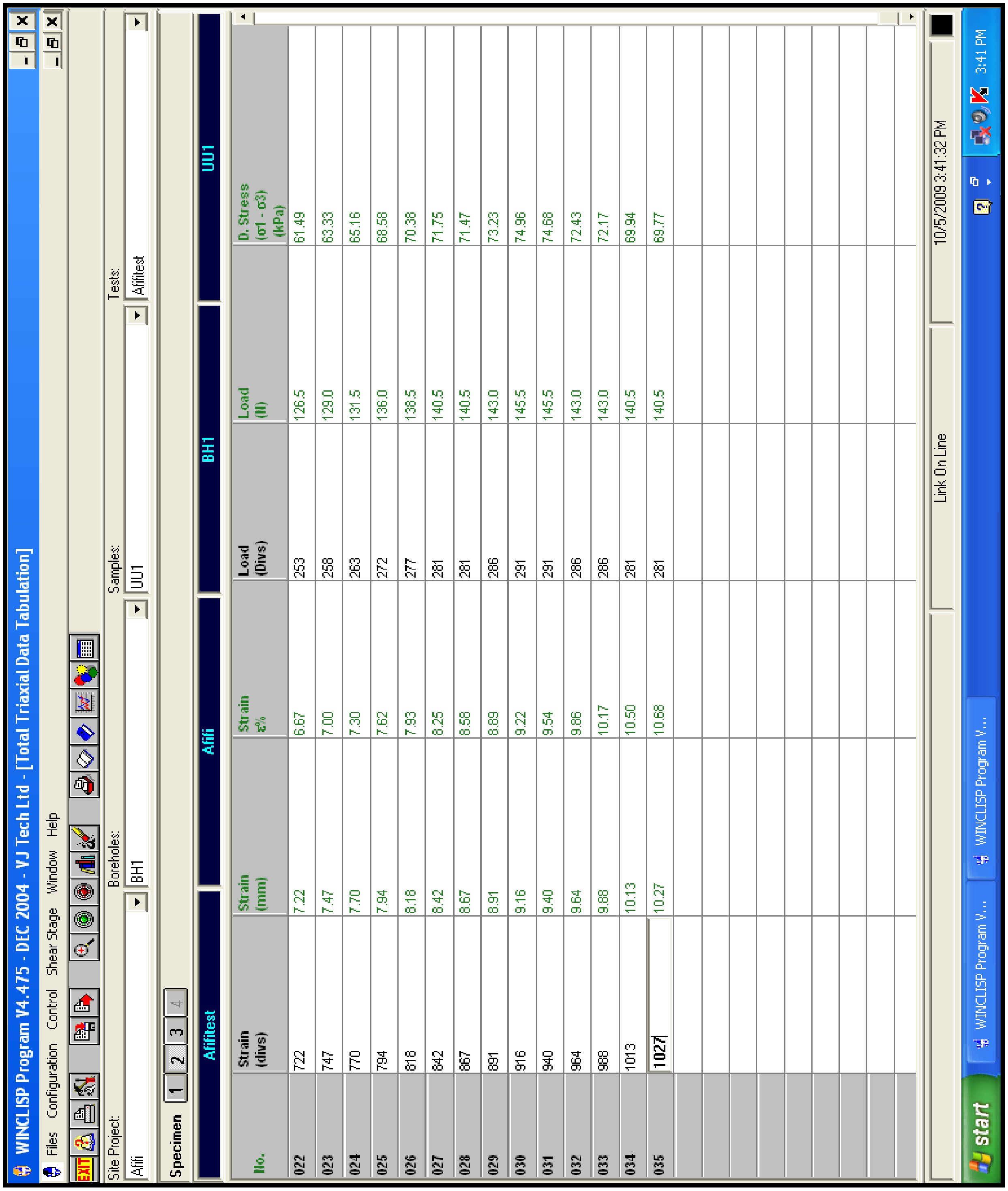
TOTAL TRIAXIAL DATA TABULATION OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE A.



	Afifitest		Afifi		BH1	UU1
No.	Strain (divs)	Strain (mm)	Strain ε%	Load (Divs)	Load (lb)	D. Stress (σ1 - σ3) (kPa)
001	215	2.15	0.00	103	51.5	0.00
002	233	2.33	0.24	178	89.0	32.99
003	257	2.57	0.55	183	91.5	35.02
004	280	2.80	0.86	187	93.5	36.67
005	303	3.03	1.16	202	101.0	43.09
006	327	3.27	1.47	211	105.5	46.86
007	351	3.51	1.79	216	108.0	48.84
008	376	3.76	2.12	216	108.0	48.68
009	401	4.01	2.45	216	108.0	48.51
010	424	4.24	2.75	220	110.0	50.05
011	447	4.47	3.05	225	112.5	52.03
012	471	4.71	3.37	225	112.5	51.86
013	495	4.95	3.68	225	112.5	51.65
014	519	5.19	4.00	230	115.0	53.60
015	545	5.45	4.34	230	115.0	53.41
016	570	5.70	4.67	234	117.0	54.88
017	595	5.95	5.00	234	117.0	54.69
018	620	6.20	5.33	239	119.5	56.59
019	647	6.47	5.68	244	122.0	58.43
020	673	6.73	6.03	244	122.0	58.22
021	699	6.99	6.37	248	124.0	59.66
022	722	7.22	6.67	253	126.5	61.49
023	747	7.47	7.00	258	129.0	63.33

APPENDIX D

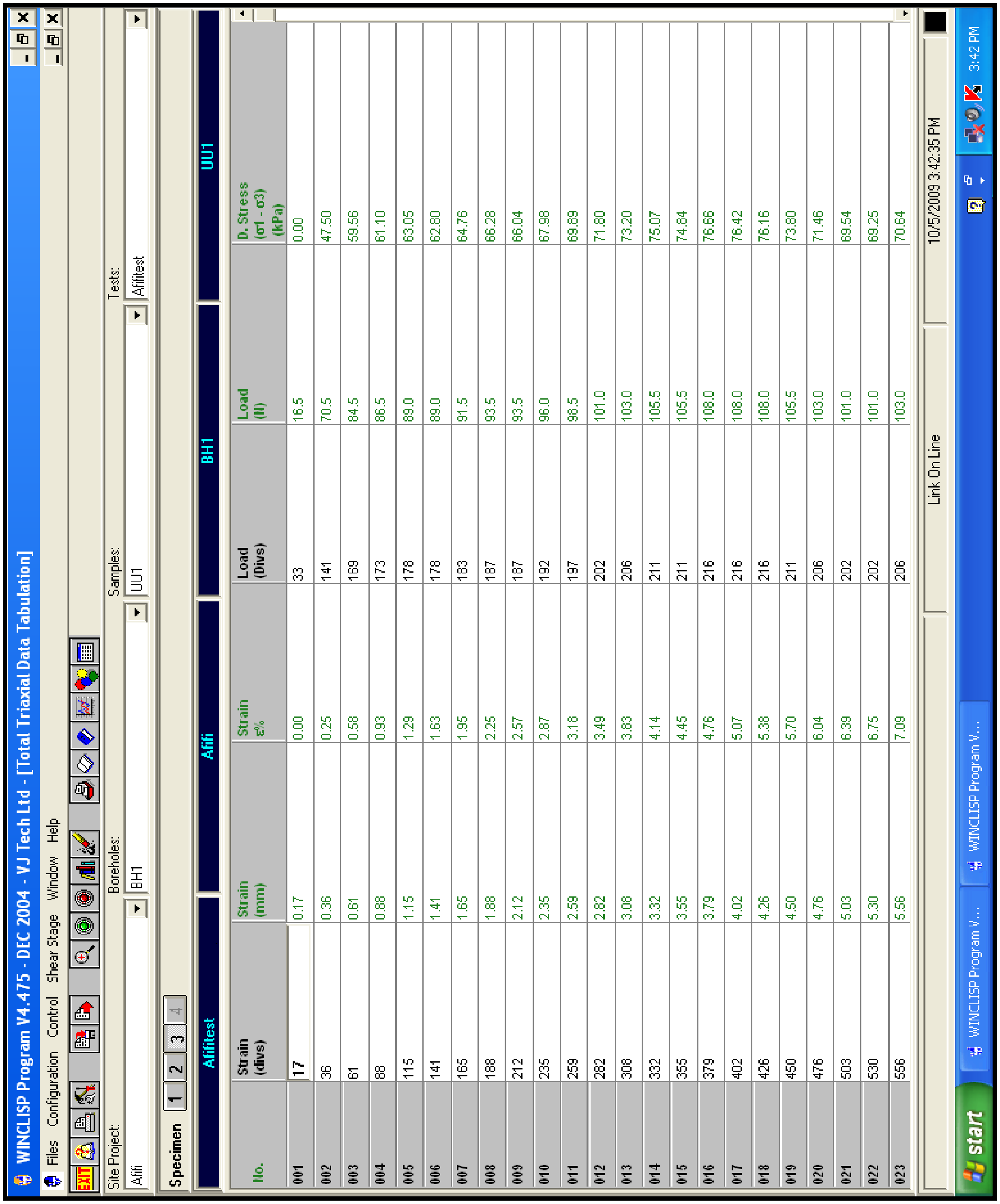
TOTAL TRIAXIAL DATA TABULATION (2) OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE A.



No.	Strain (divs)	Strain (mm)	Strain ε%	Load (Divs)	Load (ll)	D. Stress (σ1 - σ3) (kPa)
022	722	7.22	6.67	253	126.5	61.49
023	747	7.47	7.00	258	129.0	63.33
024	770	7.70	7.30	263	131.5	65.16
025	794	7.94	7.62	272	136.0	68.58
026	818	8.18	7.93	277	138.5	70.38
027	842	8.42	8.25	281	140.5	71.75
028	867	8.67	8.58	281	140.5	71.47
029	891	8.91	8.89	286	143.0	73.23
030	916	9.16	9.22	291	145.5	74.96
031	940	9.40	9.54	291	145.5	74.68
032	964	9.64	9.86	286	143.0	72.43
033	988	9.88	10.17	286	143.0	72.17
034	1013	10.13	10.50	281	140.5	69.94
035	1027	10.27	10.68	281	140.5	69.77

APPENDIX E

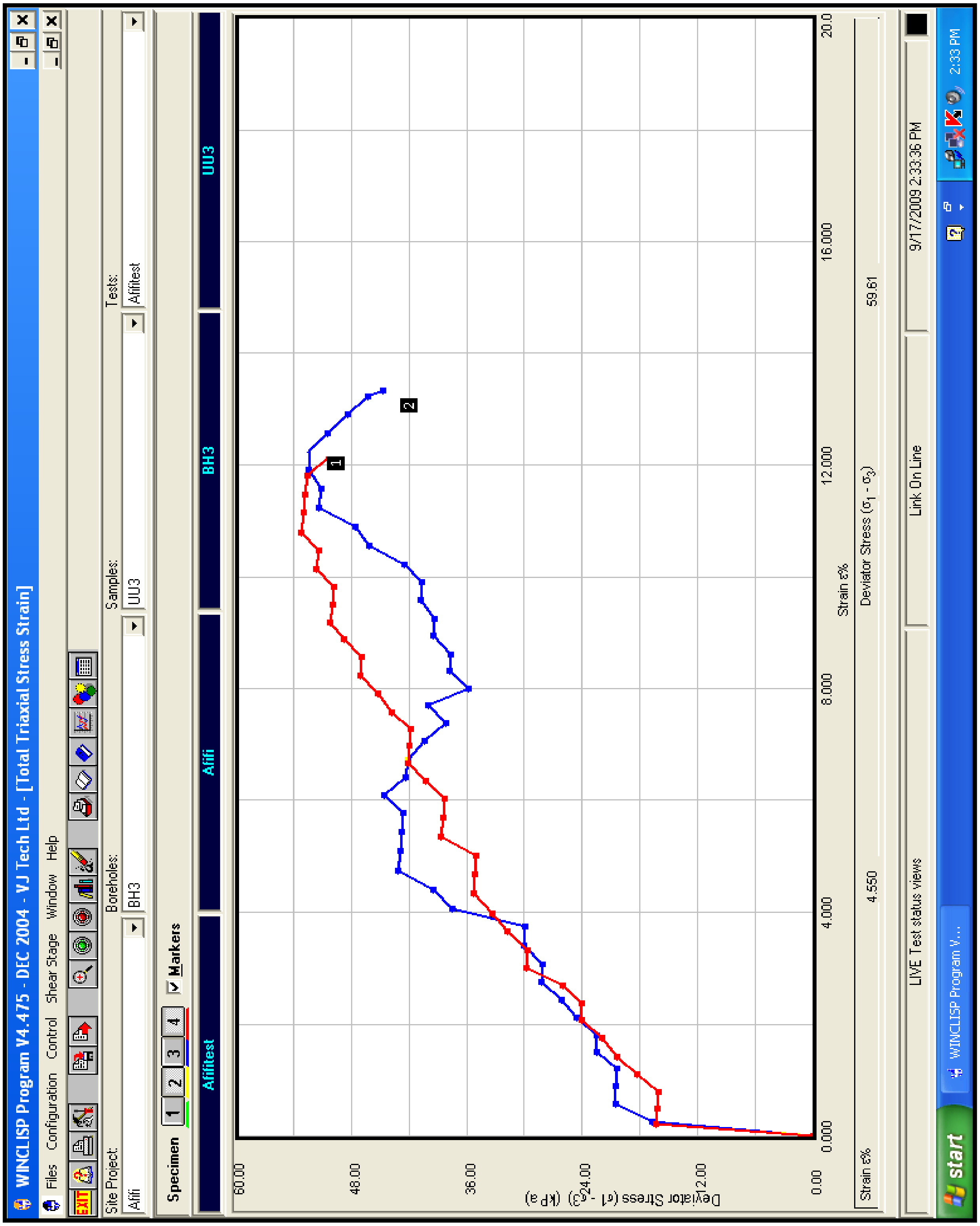
TOTAL TRIAXIAL DATA TABULATION (3) OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE A.



No.	Strain (divs)	Strain (mm)	Strain ε%	Load (Divs)	Load (N)	D. Stress (σ1 - σ3) (kPa)
001	17	0.17	0.00	33	16.5	0.00
002	36	0.36	0.25	141	70.5	47.50
003	61	0.61	0.58	169	84.5	59.56
004	88	0.88	0.93	173	86.5	61.10
005	115	1.15	1.29	178	89.0	63.05
006	141	1.41	1.63	178	89.0	62.80
007	165	1.65	1.95	183	91.5	64.76
008	188	1.88	2.25	187	93.5	66.28
009	212	2.12	2.57	187	93.5	66.04
010	235	2.35	2.87	192	96.0	67.98
011	259	2.59	3.18	197	98.5	69.89
012	282	2.82	3.49	202	101.0	71.80
013	308	3.08	3.83	206	103.0	73.20
014	332	3.32	4.14	211	105.5	75.07
015	355	3.55	4.45	211	105.5	74.84
016	379	3.79	4.76	216	108.0	76.66
017	402	4.02	5.07	216	108.0	76.42
018	426	4.26	5.38	216	108.0	76.16
019	450	4.50	5.70	211	105.5	73.80
020	476	4.76	6.04	206	103.0	71.46
021	503	5.03	6.39	202	101.0	69.54
022	530	5.30	6.75	202	101.0	69.25
023	556	5.56	7.09	206	103.0	70.64

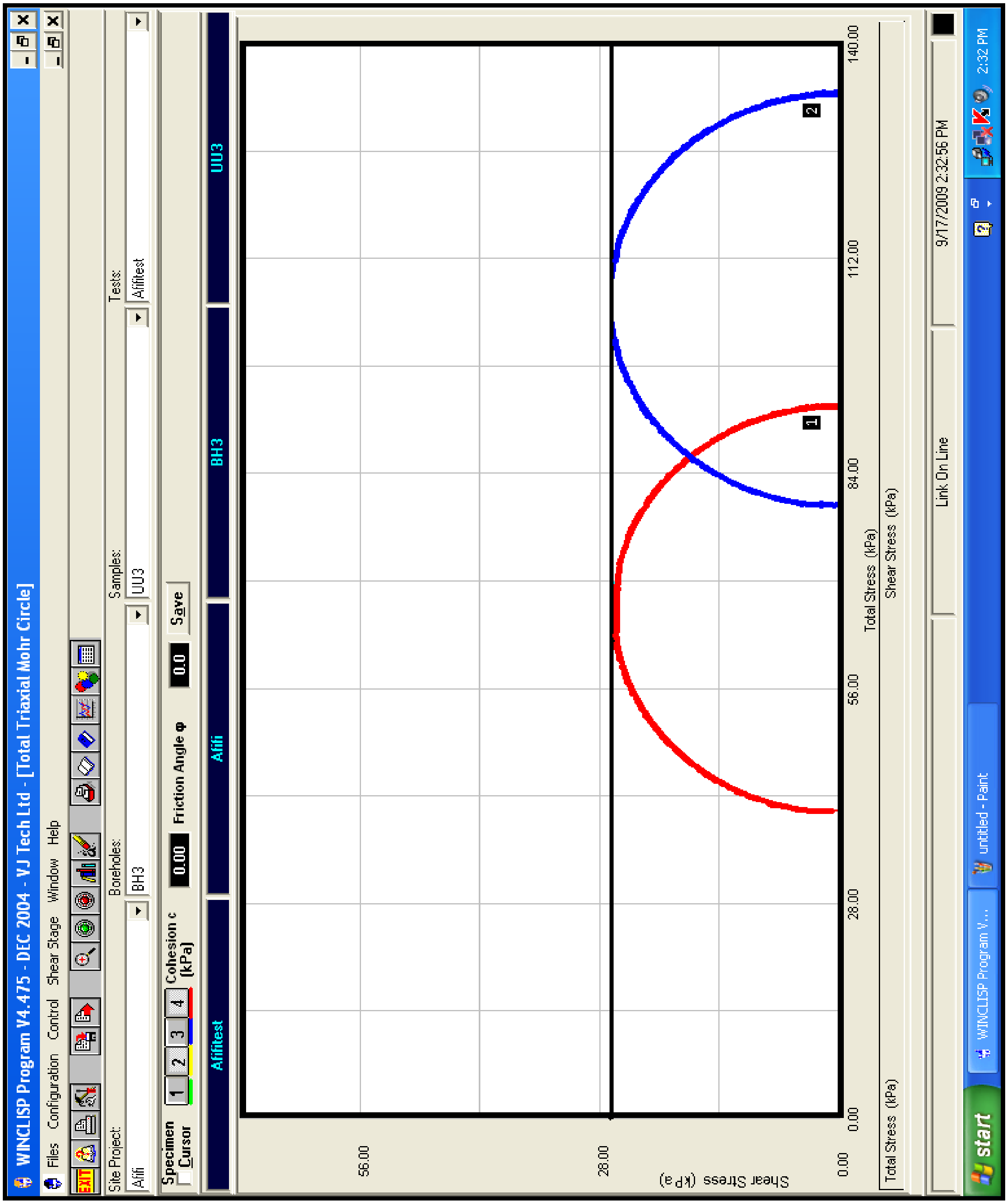
APPENDIX F

GRAPH OF TOTAL TRIAXIAL STRESS-STRAIN OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE .



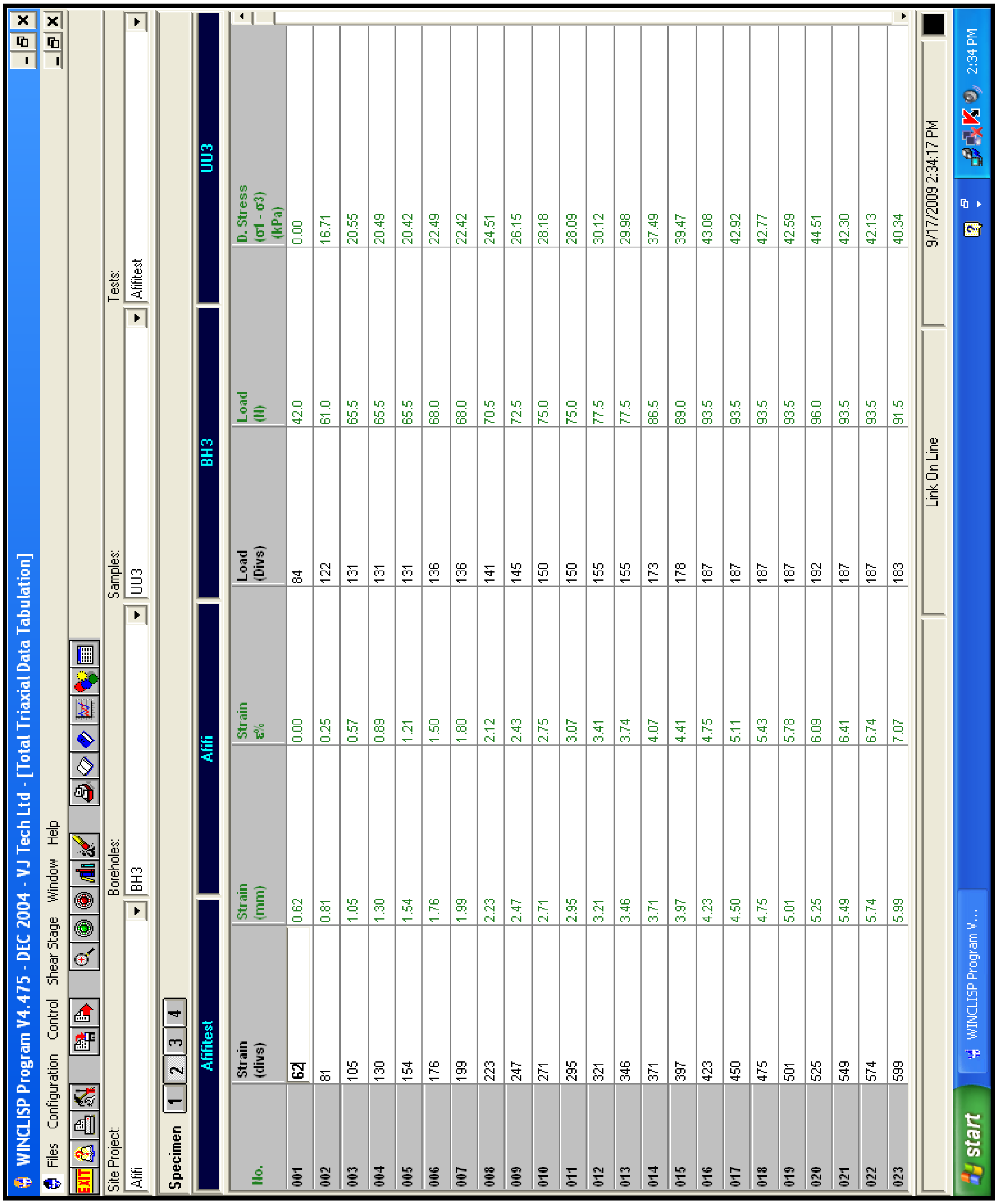
APPENDIX G

TOTAL TRIAXIAL MOHR CIRCLE OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE B.



APPENDIX H

TOTAL TRIAXIAL DATA TABULATION OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE B.


WINCLISP Program V4.475 - DEC 2004 - VJ Tech Ltd - [Total Triaxial Data Tabulation]

Site Project: Afifi | Boreholes: BH3 | Samples: UU3 | Tests: Afifitest

Specimen 1 2 3 4

Afifitest | Afifi | BH3 | UU3

No.	Strain (divs)	Strain (mm)	Strain ε%	Load (Divs)	Load (lb)	D. Stress (σ1 - σ3) (kPa)
001	62	0.62	0.00	84	42.0	0.00
002	81	0.81	0.25	122	61.0	16.71
003	105	1.05	0.57	131	65.5	20.55
004	130	1.30	0.89	131	65.5	20.49
005	154	1.54	1.21	131	65.5	20.42
006	176	1.76	1.50	136	68.0	22.49
007	199	1.99	1.80	136	68.0	22.42
008	223	2.23	2.12	141	70.5	24.51
009	247	2.47	2.43	145	72.5	26.15
010	271	2.71	2.75	150	75.0	28.18
011	295	2.95	3.07	150	75.0	28.09
012	321	3.21	3.41	155	77.5	30.12
013	346	3.46	3.74	155	77.5	29.98
014	371	3.71	4.07	173	86.5	37.49
015	397	3.97	4.41	178	89.0	39.47
016	423	4.23	4.75	187	93.5	43.08
017	450	4.50	5.11	187	93.5	42.92
018	475	4.75	5.43	187	93.5	42.77
019	501	5.01	5.78	187	93.5	42.59
020	525	5.25	6.09	192	96.0	44.51
021	549	5.49	6.41	187	93.5	42.30
022	574	5.74	6.74	187	93.5	42.13
023	599	5.99	7.07	183	91.5	40.34

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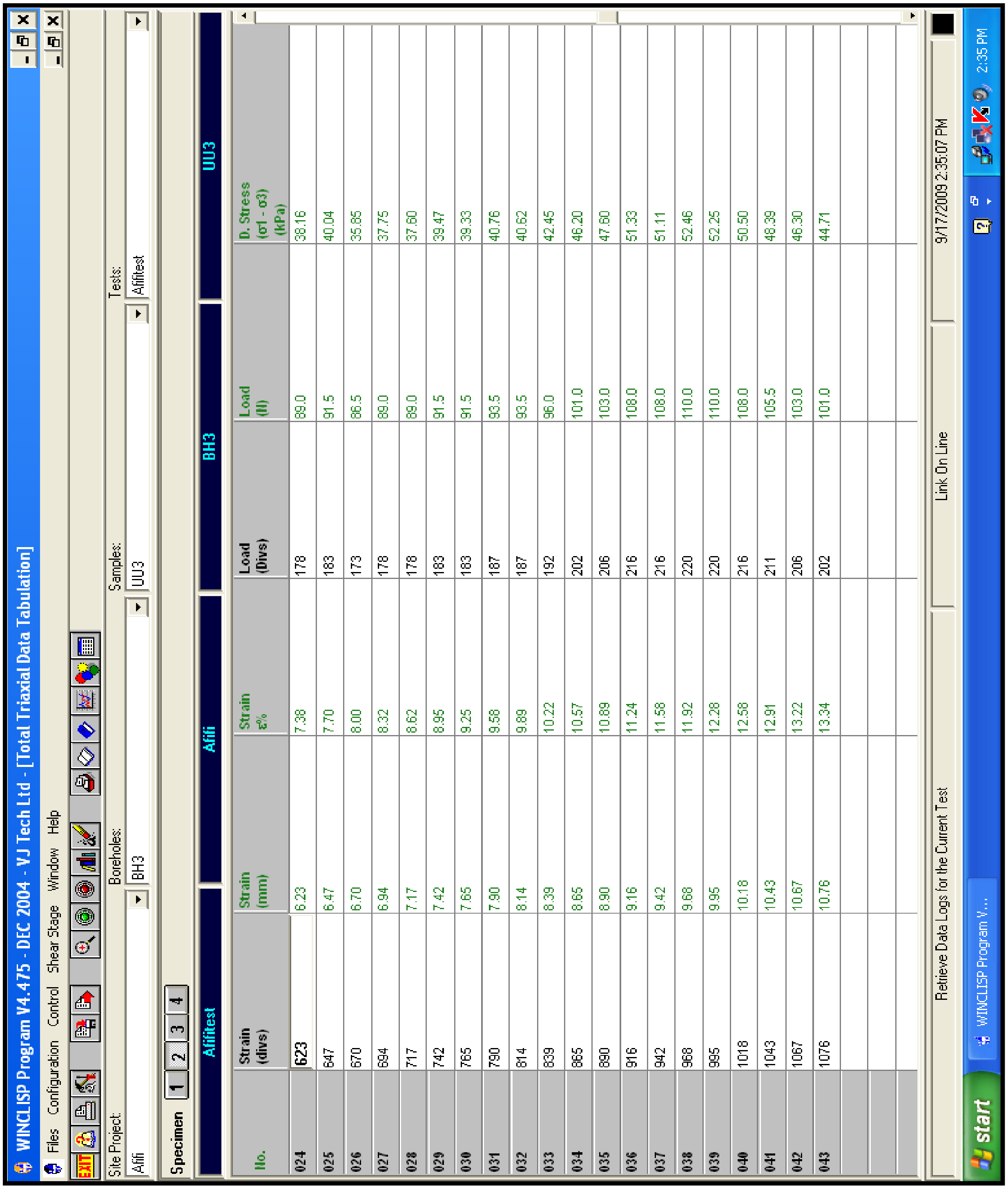
APPENDIX I

TOTAL TRIAXIAL DATA TABULATION (2) OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE B.


WINCLISP Program V4.475 - DEC 2004 - VJ Tech Ltd - [Total Triaxial Data Tabulation]

Files Configuration Control Shear Stage Window Help

Site Project: Afifi | Boreholes: BH3 | Samples: UU3 | Tests: Afifitest

Specimen 1 2 3 4

Afifitest | Afifi | BH3 | UU3

No.	Strain (divs)	Strain (mm)	Strain ε%	Load (Divs)	Load (N)	D. Stress (σ1 - σ3) (kPa)
024	623	6.23	7.38	178	89.0	38.16
025	647	6.47	7.70	183	91.5	40.04
026	670	6.70	8.00	173	86.5	35.85
027	694	6.94	8.32	178	89.0	37.75
028	717	7.17	8.62	178	89.0	37.60
029	742	7.42	8.95	183	91.5	39.47
030	765	7.65	9.25	183	91.5	39.33
031	790	7.90	9.58	187	93.5	40.76
032	814	8.14	9.89	187	93.5	40.62
033	839	8.39	10.22	192	96.0	42.45
034	865	8.65	10.57	202	101.0	46.20
035	890	8.90	10.89	206	103.0	47.60
036	916	9.16	11.24	216	108.0	51.33
037	942	9.42	11.58	216	108.0	51.11
038	968	9.68	11.92	220	110.0	52.46
039	995	9.95	12.28	220	110.0	52.25
040	1018	10.18	12.58	216	108.0	50.50
041	1043	10.43	12.91	211	105.5	48.39
042	1067	10.67	13.22	206	103.0	46.30
043	1076	10.76	13.34	202	101.0	44.71

Retrieve Data Logs for the Current Test | Link On Line | 9/17/2009 2:35:07 PM

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APPENDIX J

TOTAL TRIAXIAL DATA TABULATION (3) OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE B.

WINCLISP Program V4.475 - DEC 2004 - VJ Tech Ltd - [Total Triaxial Data Tabulation]

Files Configuration Control Shear Stage Window Help

Site Project: Afifi | Boreholes: BH3 | Samples: UU3 | Tests: Afifitest

Specimen 1 2 3 4

Afifitest | Afifi | BH3 | UU3

No.	Strain (divs)	Strain (mm)	Strain ε%	Load (Divs)	Load (lb)	D. Stress (σ1 - σ3) (kPa)
001	9	0.09	0.00	94	47.0	0.00
002	25	0.25	0.21	131	65.5	16.28
003	46	0.46	0.49	131	65.5	16.23
004	69	0.69	0.79	131	65.5	16.13
005	92	0.92	1.09	136	68.0	18.26
006	117	1.17	1.42	141	70.5	20.38
007	142	1.42	1.75	145	72.5	22.00
008	166	1.66	2.07	150	75.0	24.09
009	189	1.89	2.37	150	75.0	24.02
010	213	2.13	2.68	155	77.5	26.06
011	237	2.37	3.00	164	82.0	29.82
012	262	2.62	3.33	164	82.0	29.72
013	287	2.87	3.66	169	84.5	31.71
014	312	3.12	3.99	173	86.5	33.29
015	338	3.38	4.33	178	89.0	35.28
016	364	3.64	4.67	178	89.0	35.13
017	390	3.90	5.01	178	89.0	35.00
018	416	4.16	5.36	187	93.5	38.63
019	441	4.41	5.68	187	93.5	38.47
020	467	4.67	6.03	187	93.5	38.33
021	492	4.92	6.36	192	96.0	40.26
022	516	5.16	6.67	197	98.5	42.16
023	539	5.39	6.97	197	98.5	42.02

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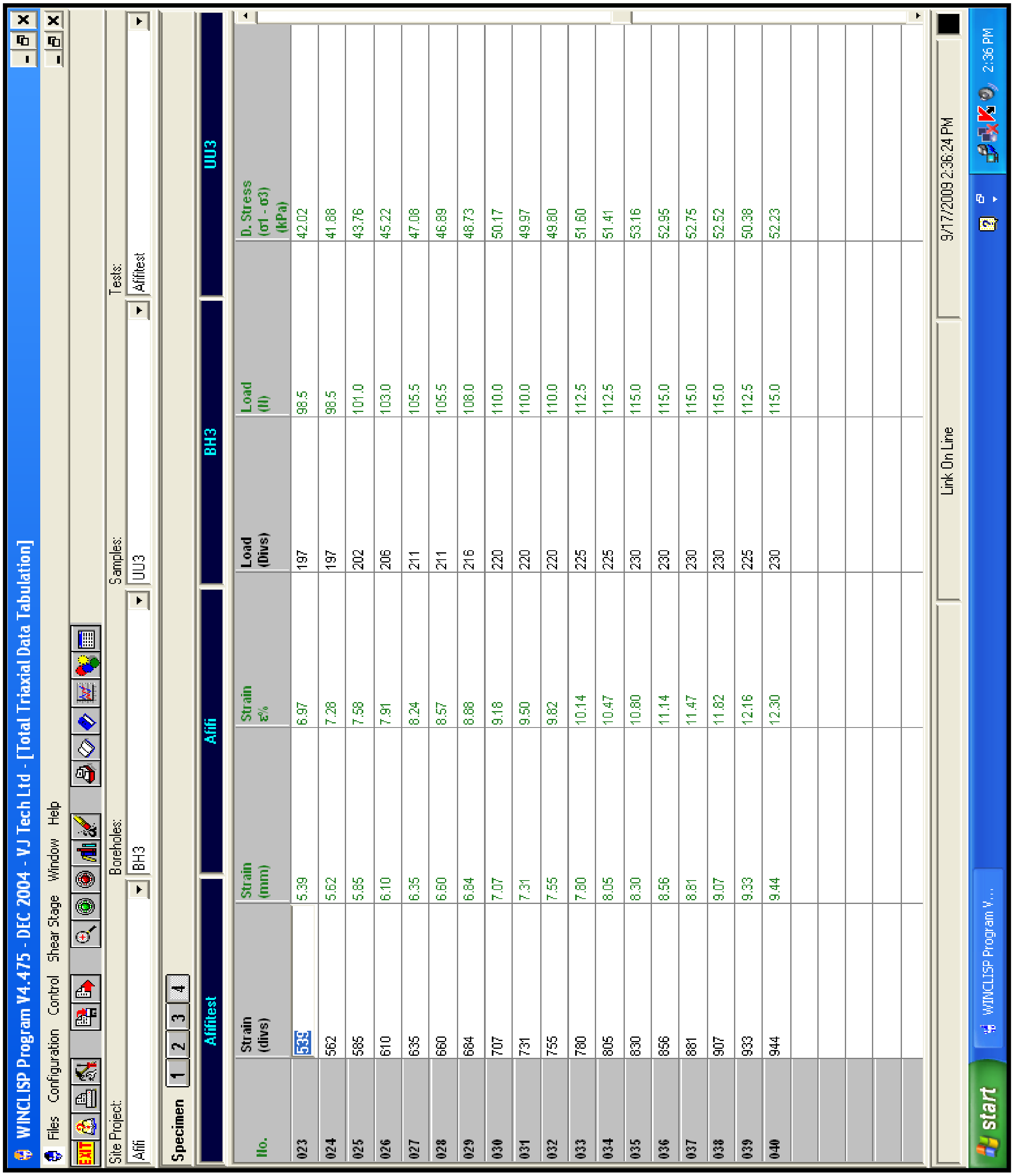
APPENDIX K

TOTAL TRIAXIAL DATA TABULATION (4) OF UNCONSOLIDATED-UNDRAINED TEST FOR SITE B.



No.	Strain (divs)	Strain (mm)	Strain ε%	Load (Divs)	Load (lb)	D. Stress (σ1 - σ3) (kPa)
023	539	5.39	6.97	197	98.5	42.02
024	562	5.62	7.28	197	98.5	41.88
025	585	5.85	7.58	202	101.0	43.76
026	610	6.10	7.91	206	103.0	45.22
027	635	6.35	8.24	211	105.5	47.08
028	660	6.60	8.57	211	105.5	46.89
029	684	6.84	8.88	216	108.0	48.73
030	707	7.07	9.18	220	110.0	50.17
031	731	7.31	9.50	220	110.0	49.97
032	755	7.55	9.82	220	110.0	49.80
033	780	7.80	10.14	225	112.5	51.60
034	805	8.05	10.47	225	112.5	51.41
035	830	8.30	10.80	230	115.0	53.16
036	856	8.56	11.14	230	115.0	52.95
037	881	8.81	11.47	230	115.0	52.75
038	907	9.07	11.82	230	115.0	52.52
039	933	9.33	12.16	225	112.5	50.38
040	944	9.44	12.30	230	115.0	52.23