CERTIFICATION OF APPROVAL

The Compression Behaviour of Intact Marine Clay in Malaysia

by

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CERTIFICATION OF ORIGINALITY

This to certify that I am responsible for the work submitted in this project, that the original work is my own except specified in the references and acknowledgement, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

(MOHAMMAD FITRY BIN MOHD SHUKOR)

ACKNOWLEDGEMENT

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This final year project makes me realized on the positive value of having my own research in geotechnical engineering area. Other additional value gained is to produce the thesis where it is very challenging and push me to the limit to finish it. A lot of new things with valuable experiences were gained during the two semester of final year project.

I would like to thank to all geotechnical laboratory technician for lending big help during this research. Last but not least, deepest appreciation to all my colleagues for their time spent, support and advice throughout this project.

Table of Contents

LIST OF FIGURES

LIST OF TABLES

Abstract

Recent development in economic and the search of new energy has brought to the usage of area with plenty of marine clay. This soil creates several complexities in geotechnical design due to high compression behaviour and low bearing capacity. 8 oedometer tests were performed in this project to study the compression curves under one-dimensional loading from shallow to deeper depth of soil sample taken. Result shows that the compression index increased from 0.564 – 0.611 as the percentage finer increased, thus the compressibility of soil is mainly controlled by the fines particles. The preconsolidation pressure for all samples ranged from 200 – 500 kPa which can be compared with the presence literature for the compression behaviour towards denser soil. In conclusion, the compression behaviour of intact soil is close related with the basic soil properties and structure effect of soil.

1 Introduction

1.1 Background of study

In many part of coastal regions, marine clay is one of the fine-grained soils found below the ocean bed. Marine clay can also be located onshore as well and it is a microcrystalline in nature. Marine clay can be seen widely deposited along the coastal areas of Peninsular Malaysia. According to Taha et al. (2000), marine clay found in Klang area consists of soft clays, peat and other soft organic deposits.Generally, the thickness of the soft clay found in the Southeast Asian region (which includes countries such as Malaysia, Indonesia, Singapore, Thailand, etc.) ranged from very shallow thin layers to depths of 40 m. Ting et al (1988) also reported that the thickness of soft soil deposits ranged from 15 m to 30 m for the coastal sites and from 2 m to 9 m for inland areas. The soil then followed by layering of sand, peat and other soft clay until it reaches quartzite bedrock at a depth of 80m below ground surface. These deposits were formed about 10,000 years ago due to change in sea level.

According to Oh and Chai (2006), the occurrence of thick deposits of marine clay in the North-western part of Peninsular Malaysia is due to the result of quaternary deposition. Kaniraj and Joseph (2007) stated that most coastal areas and major river valleys in Malaysia result in quaternary erosion, accentuated by climatic and sea level changes. These processes have produced widespread and thick alluvial deposits. They reported that the soil has poor ground condition and posed numerous technical challenges in the implementation of development projects.

Tan et al. (2004) has carried out some research works for Klang marine clay and they found that the region selected for the work is generally soft, inorganic, possess medium to extremely high plasticity, compressible with high Liquidity Index.

1.2 Problem Statement

Over the past few years, the development in Malaysia economy has caused an increase in number of construction thus led to more demand of land usage. The extensive usage of the land led human to reclaim the coastal region in order for continuous development. These areas that received impetus for such developments include the coastal regions where ports and highways are located. Most of the construction or operations in offshore areas such as the jack-up installation deal with this problematic marine clay soil. It poses major construction and maintenance problems due to low bearing capacity and high deformation behaviour. The condition of the soil that is very soft creates several complexities in design, construction and maintenance of structures. Other characteristics associated with this soil are high in water content and larger void ratio.

There are two parts in geotechnical problems in dealing with soft clay engineering namely settlement and stability (Tan et al., 2004). Recent development in road construction in Malaysia has brought some discovery of deep soft marine clay along the North-South Highway. The soil found was located in the southern stretch of the highway that known as Muar flat. To assess the problems of stability and settlements of highway embankments on soft soil, from 1986 to 1989 MHA constructed several trial embankments sections with different soil improvement methods and studied their behaviour.

1.3 Objectives

This project is carried out to understand the compressibility behavior of intact marine clay located at East coast of Peninsular Malaysia and Sarawak region. The knowledge of this behaviour is important as to predict the compression behavior of marine clay at different depth. Thus, the following objectives are presented as below:

1. To analyze the compression behavior of intact marine clay under one dimensional loading stress.
2. To analyze the effect of variation of void ratio to the compression curve of intact marine clay.
3. To analyze and compare the pattern of compression curves of marine clay from shallow to deeper depth.

2 Literature Review

2.1 Marine Clay Structures

One of the ways to assess soft soil like marine clay is by its structure. It has been well accepted that the structure is formed during their depositional processes, where complex interactions between the mechanical, chemical and biological factors are involved. Clay structure usually is formed by the group of crystalline minerals of kaolinite, illite and montmorillonite. These minerals share similar chemical composition but having a different layering structure. During deposition, the mineral particles are arranged into structural frameworks that known as soil fabric. The particles were connected to each other by a random arrangement and form the soil fabric. There are 2 types of soil fabric commonly seen in the soil and they are flocculated and dispersed as shown in Figure 2.1. A flocculated structured is where the particles arranged in perpendicular to each other whereas the dispersed structure is a result of parallel arrangement of a particle layer. This arrangement will determine the spaces inside the soil which can affect the void ratio. The type of structures found in this soil formation is related to the changes in sea level during soil deposition. The changes in sea level have a great influence in the various materials mixed which were deposited in different depositional environment.

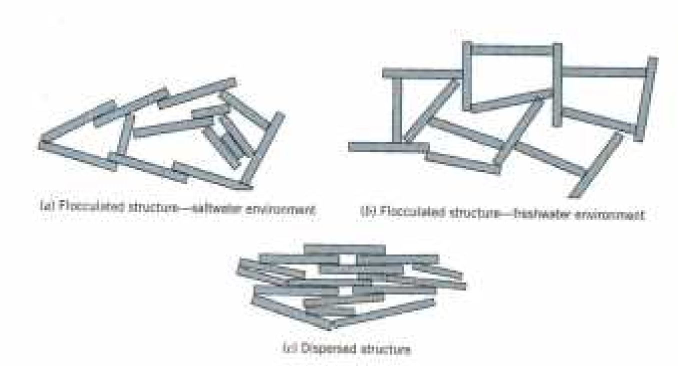

Figure 2.1: Marine Clay Structures

2.2 Consolidation Theory

When soil is subjected to a continuous loading, it will deform due to changes in the soil volume. This process is known as the consolidation. In general, consolidation theory proposed by Terzaghi in 1925 deals with the response of soil systems to impose load and predicts stresses as well as displacements. It started with the elastic settlement of soil where the volumes change is close related to the elasticity of soil. The moisture content inside the soil will remain constant. As soon as the elastic settlement has completed, the soil then will undergo primary consolidation and later on continued with secondary consolidation. During the primary consolidation stage, the pore water pressure develops inside the soil, taken up most of the load given. Effective stress will be equal to the pore water pressure at the beginning of the time. In cohesive soil, low in permeability caused the water to dissipate over a long period of time. As a result of completely loss in pore water pressure, the increase in load will cause the soil to undergo secondary consolidation. The soil deform because of the plastic adjustment of soil fabrics. Secondary consolidation only happens if the soil received constant effective stress. This mechanism of soil deformation can be described by using the simple spring analogy as shown in Figure 2.2. The soil was described as a spring surrounded with water in the initial stage. When the loads were applied to the piston, it was carried by water as the valve was closed. When the valve was opened for drainage, the water is allowed to dissipate out from the container. The loads were then transferred to the spring thus showing the settlement of soil by the reduction of spring length.

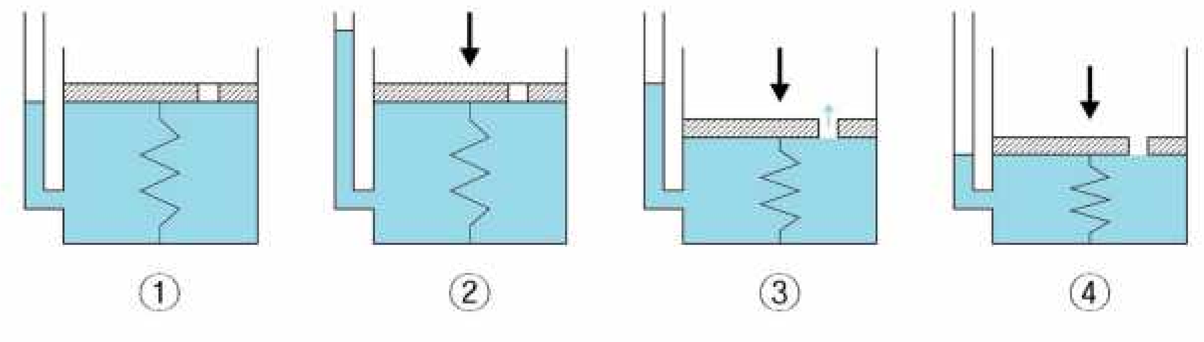

Figure 2.2: Spring Analogy Theorem (Budhu, 2000)

The one-dimensional consolidation test was first introduced by Terzaghi (1925). The test is carried out to investigate the compression behaviour for three stages of compression in the soil which are elastic, primary and secondary consolidation. During the test, the soil will deform vertically as the load is increased and measured in function of time. Two stages of loading and unloading process will be performed in the test as to determine several compressibility parameters. Time prediction for consolidation process is important because of the concern in settlement.

2.3 Compressibility properties

The result for the oedometer test can be represented by determining the final void ratio from the specimens tested. The data will be plotted with a graph of void ratio against the vertical effective stress. The graph in Figure 2.3 shows the relationship between the void ratios with the effective stresses from the consolidation test. The soil will behave as an elastic material at the beginning of the time until it reaches the yielding point indicated as point M shown in Figure 2.3. Yielding point is a turning point where the soil starts to behave like a plastic material. The stress at the turning point on the *e log curve σ* is the stress at which soil has previously experienced the historical maximum consolidation in the field (Ishibashi and Hazarika, 2011). The deformation of the soil is said to be permanent due to constant decrease in volume of the soil. Water inside the soil dissipated slowly until maximum stress point applies before unloading process happens. Compression index, *Cc* can be obtained during this stage by assuming the linear portion of the graph. Compression index is an important parameter in this plot as it will predict the amount of consolidation for the primary stage. The recompression index, *Cr* is used when the soil is overconsolidated. During the unloading stage, the soil will start to swell by absorbing the water. The coefficient of consolidation, *Cv* will decreases as the liquid limit of soil increases. It is used as a parameter to describe the rate at which the soil consolidates when subjected to pressure.

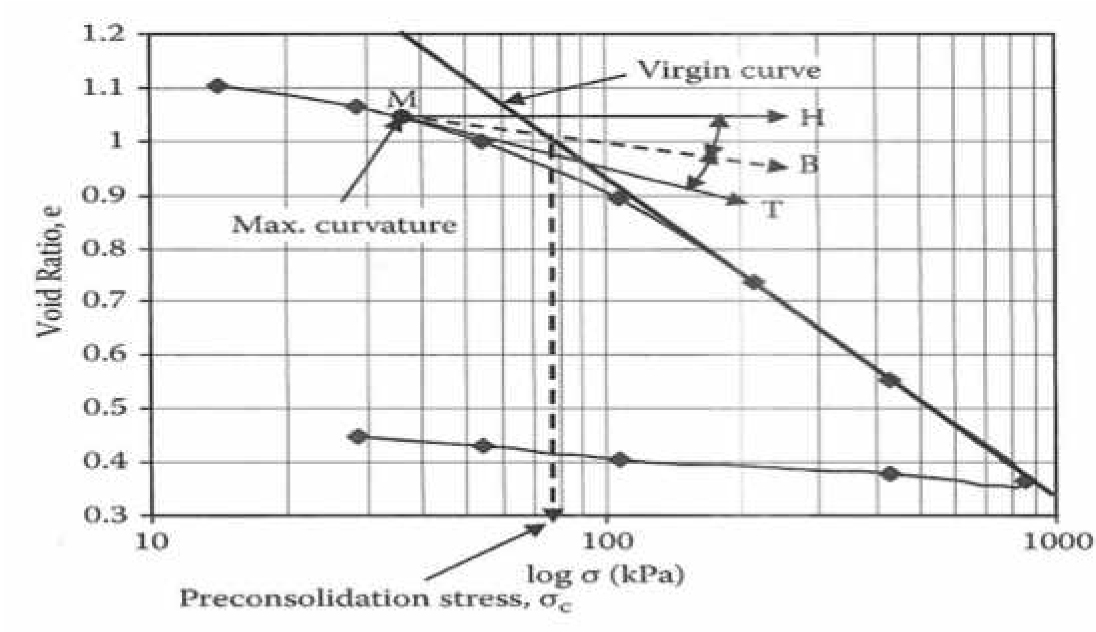

Figure 2.3: e – log σv curve for soil compression (Ishibashi and Hazarika, 2011)

2.4 Yielding in Soft Clay

As the soil received continuous loading, it will deform depending on how much the pressures are being applied onto the soil. The soil will change in terms of the structure which gives the behaviour either elastic or plastic material. The soil is said to yield as soon as it reached the maximum value it can sustain before the volume starts to change by means of expulsion of water from the void. The decrement of void ratio is linear to applied pressure after the yielding point. The yield point gives an indication to identify the soil whether is normally or overconsolidated based on past overburden pressure. Most natural, light overconsolidated natural clays show a relatively stiff behaviour when subjected to the stress. It is well known that the geological processes that determined the stress-strain behavior of natural clayey soil. There are several factors that actually contributed to the mechanical behaviour of the clayey soil. The rate of applied stress can be sustained by the soil depending on the particles arrangement and inter-particle bonds due to deposition and geological processes. Callisto and Calabresi (1995) stated that yielding was then associated with a rather abrupt disruption of such a structure (destructuration).

2.5 Behaviour of Intact Clays

Clays which are highly sensitive and stiff fissured of various geological origins possess definite macroscopic and microscopic structures. The variation in structures of clay soil can influence the strength and deformation properties. Macroscopic structures are defined as visible features that include fissures, joint, stratification and other discontinuities in an otherwise intact soil mass. According to Lo & Hichberger (2007), microscopic structures would include soil fabric and cementation bonds identifiable through the scanning electron microscope techniques. The importance of soil structure on the behaviour of soils is fully recognized and is as important as void ratio and stress ratio for the understanding of the behaviour of natural soils (Leroueil & Vaughan, 1990). Hight & Leroueil (2002) stated that the soil structure lead to increased pre-yield stiffness, higher yield strength, less ductile behaviour and adding to natural variability and anisotropy. Hong et al, 2012 carried out an experiment on comparison of natural and reconstituted of several marine clay. They performed few numbers of oedometer tests on several clays shown in Figure 2.4 to find out the behavior of soil in natural state. They divided the samples into two types which are type one and type two. Based on the curves in Figure 2.4, Type two soils have a transitional drop zone after the consolidation yield exceeded. The transitional drop in the curves shows a zone where the soil experienced transitional stress compare to Type one soil. They claimed that all behaviour described in the graph implies that the shape of compression curves and degradation of the soil structure associated with the irreversible deformation for various natural clays may depend on the specific mechanism response for the presence of soil structure. The transitional state presence in the curves proved that the soil has gradual change in microstructure during consolidation with relatively large deformation shown in the regime. They further studied that the compression behaviour of natural clays can be classified into three regime: 1) the pre-yield starting with small compressibility when the effective stress is lower than the consolidation yield stress; 2) the transitional regime with gradual loss of soil structure when the effective stress is lower than the transitional stress; and 3) the post-transitional regime when the effective stress is higher than the transitional stress.

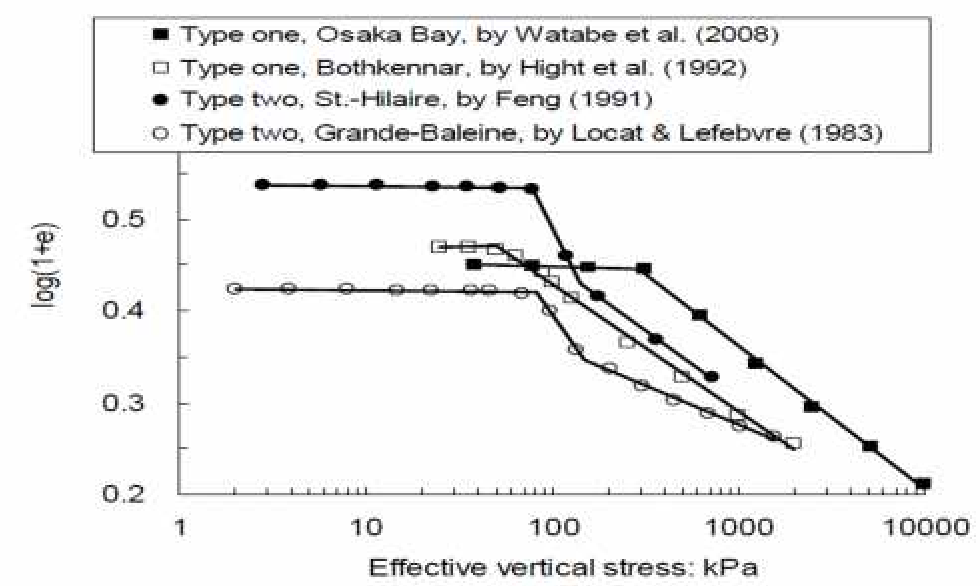

Figure 2.4: Compression curves of two types of natural clays (Hong et al., 2012)

2.6 Effect of Initial Void Ratio on Consolidation Test

Figure 2.6 shows the compression curves for two clay samples in a standard *e: log Pc'* space. The preconsolidation stress, *Pc* is the effective stress that separates the boundary line of soft and stiff deformation of soil with response to loading. The consolidation of the curves illustrated a slight concave upon reaching the virgin compression line. The same behavior was also mentioned by Taha et al. (2000) in his research works. The *Cc* is described by the change of the void ratio, e and the log scale. It can clearly observe that the initial void ratio of L1 was lower than L2 and L3 which indicate that the soil is relatively denser. It can be seen that the soil that has lower initial void ratio tend to has higher preconsolidation pressure.

Table 1: Value of Pre-consolidation and Initial void ratio (Z.A Rahman et al. 2013)

Sample No	L1	L2	L3
Pre-consolidation stress, P'c	65 – 58 kPa	14 – 53 kPa	60 – 59 kPa
Initial Void ratio	1.75	2.60	2.18

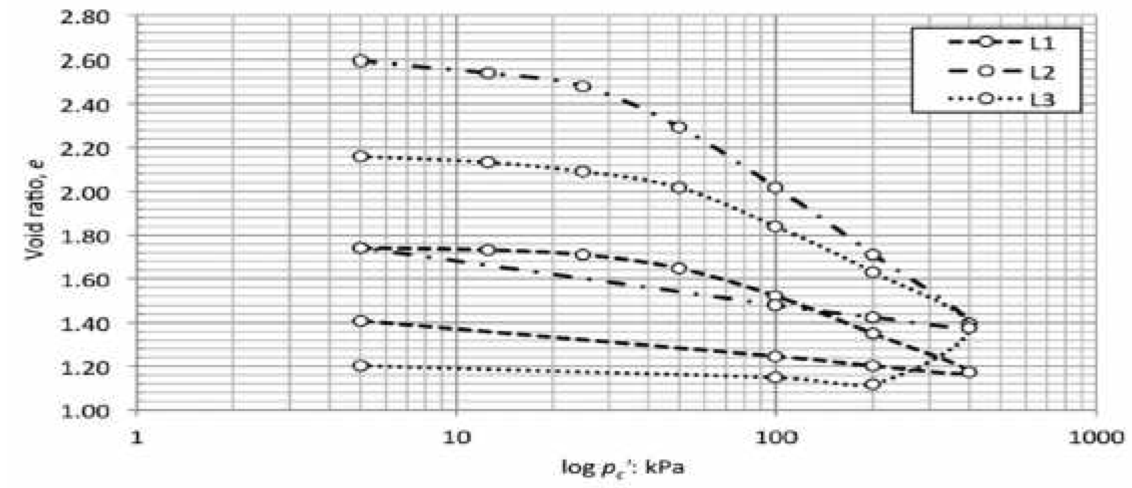

Figure 2.5: Effect of initial void ratio in e – log Pc' curves (Z.A Rahman et al, 2013)

3 Methodology

Several tests were carried out such as the basic soil test and oedometer in this project. The samples were taken from a few offshore areas located at the East Coast of Peninsular Malaysia and Sarawak region. The depth of the samples taken was varied in the range of 2 – 30 m. The basic tests involved were particle size distribution, moisture content, Atterberg limits, specific gravity and Scanning Electron Microscope (SEM). The summary of the test conducted is represented in the table 3.1. The flow chart of overall laboratory testing was described in Figure 3.1.

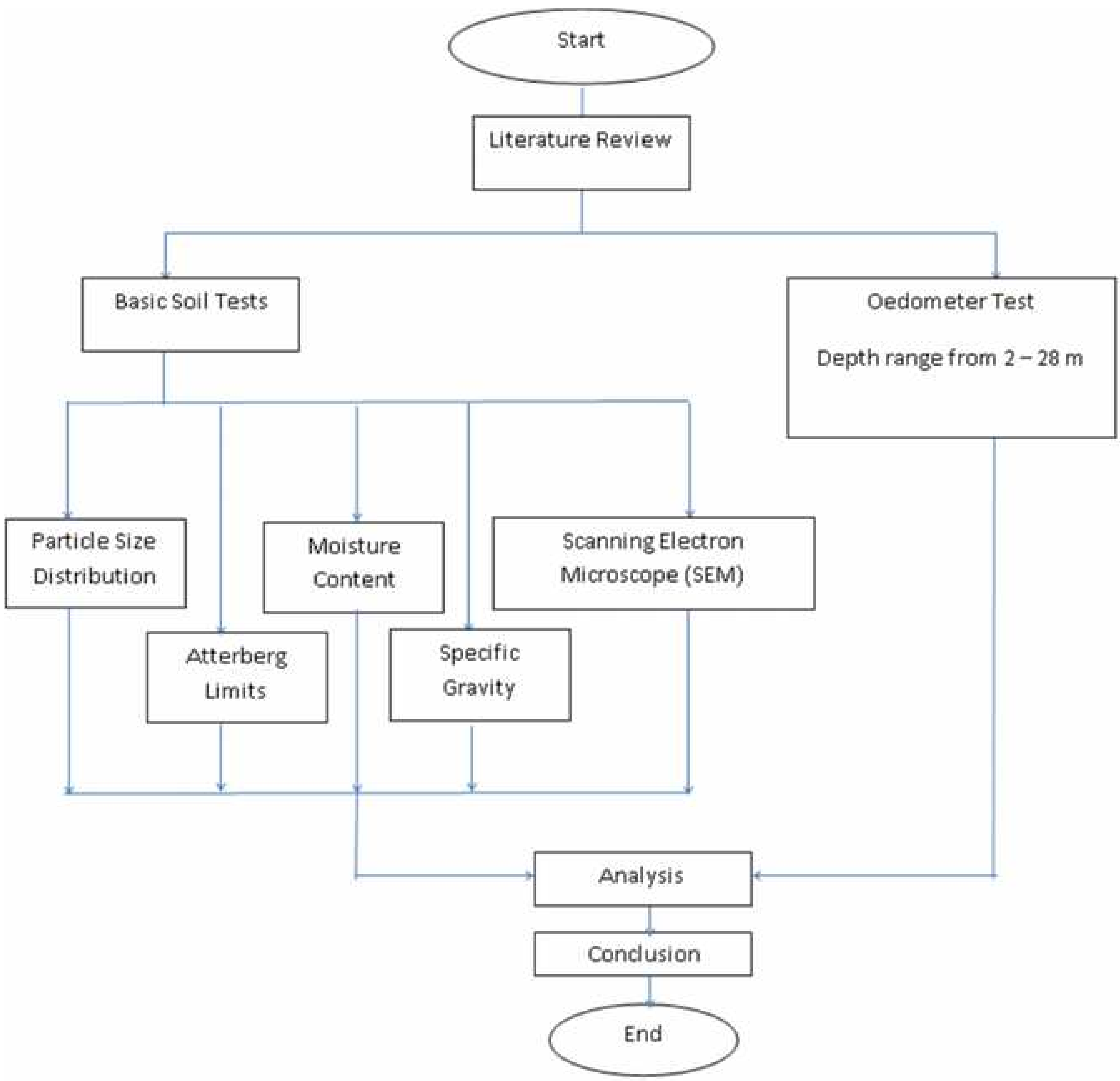

Figure 3.1: Flow chart of laboratory testing

3.1 Basic Soil Tests

The tests were performed in order to obtain the basic soil properties such as moisture content, particle size, plasticity value and others. There were 8 different samples of marine clay used for the entire test conducted. All the tests performed in this project were accordance to British Standard which is BS 1337 – 2: 1990 and the results are discussed in Chapter 4.

3.1.1 Particle Size Distribution (PSD) and Hydrometer Test

The test was performed on 8 different marine clay samples by using a set of sieves with different sizes. The samples were left for 24 hours at 105^0C in the oven to remove the moisture inside it before it being broken down to smaller sizes. The samples then were sieved using the shaker. Since the soil is a fine-grained soil, hydrometer test is required to complete the test for the soil passing through the 425 μm sieve to complete the distribution curve.

3.1.2 Moisture Content (MC)

Oven drying method was used to measure the initial water content inside the soil. A small portion of soil was taken from the trimming process and left for 24 hours at 105^0C in the oven to completely remove the water inside the soil. The amount of water loss will then be calculated for the initial moisture content value.

3.1.3 Atterberg Limits

The Atterberg limits of the soil are composed of 2 different tests which were Plastic Limit (PL) and Liquid Limit (LL). They were determined by the standard Cone Penetrometer method and Plastic Limit method as shown in Figure 3.2 and 3.3. The values for both plastic and liquid limit were used for correlation with several parameters obtained from oedometer test. The value of PL and LL were used to determine the Plasticity Index.

3.1.4 Specific Gravity (SG)

Specific gravity tests were conducted using the small pyknometer jar method as shown in Figure 3.3. This method is more accurate for fine grained soil compare to the large pyknomter jar method. The value for the SG will be used for the calculation of the soil parameters such as the moist density or the initial void ratio.

3.1.5 Scanning Electron Microscope (SEM)

SEM was performed in order to obtain the morphology of the soil. The samples were scanned for the existing structure to identify the arrangement of the particles. The test can identify the traced elements of the soil based on the selected spots. The samples were trimmed for 10 mm diameter before it will be scanned.

Figure 3.3: Sample of marine clay was placed inside the container before the penetration test for Liquid limit test

Figure 3.2: Sample was rolled into 3mm diameter for plastic limit test

Figure 3.4: Small pyknometer jars were used in SG test

Table 2: Summary of laboratory testing

Number	Sample	Symbol	Depth (m)	Laboratory Testing						
				PSD	PL	LL	SG	SEM	MC	OEDOMETER
1	SERENDAH	SR1	19.7	/	/	/	/	/	/	/
2	SEMANTAN	SE1	2.2	/	/	/	/	×	/	/
3	SEMANTAN	SE2	6.6	/	/	/	/	×	/	/
4	SEMANTAN	SE3	8.6	/	/	/	/	×	/	/
5	SEMANTAN	SE4	18.8	/	/	/	/	/	/	/
6	SEMANTAN	SE5	27.6	×	/	/	/	/	/	/
7	DUYUNG SHALLOW	D1	4.8	/	/	/	/	/	/	/
8	DUYUNG SHALLOW	D2	8.8	×	/	/	/	×	/	/

3.2 Oedometer Test

Eight oedometer tests were performed in order to determine the magnitude of soil volume decrement which was laterally confined when subjected to different vertical pressures in this project. The results of the experiments will give the compression curve (pressure-void relationship). This data is useful in determining the compression index (*Cc*), recompression index (*Cs*) and the preconsolidation pressure (*Pc*) of the soil. The oedometer tests were carried out as specified by British Standards (BSI, 1990) and using a standard fixed ring method. The trimmed specimens were placed inside the stainless steel ring withthe diameter of 50 mm and 20 mm height. The lever arm ratio of the oedometer machine is 10: 1 indicating that the load transfer from the weight should be multiplied by the given proportion. The porous stones were placed both on the top and bottom of the sample to allow water to penetrate inside the soil. The porous stones were kept in distilled water under vacuum condition in order to saturate the porous stone and to facilitate the permeability of the porous stone over a period of time. The test was conducted with room temperature maintained at 25^{o} +- 1^{o} C.

3.2.1 Sample Preparation

For this research project, intact samples were obtained directly from the soil contractor. The preparation of the sample started with the taking out the sample by cutting the plastic casing and separating the sample from wax layer as shown in Figure 3.5. Then, the sample was cut 1/3 from the original height before trimming. The soil was trimmed to a size of 20 mm diameter and 50 mm height following the stainless steel ring dimensions. Trimmer was used to hold the soil while a pre-tension steel wire was used to reduce the thickness of soil to the required dimensions. The soil was trimmed vertically against the trimmer wall in order to have a smooth cylindrical shape presented in Figure 3.6. The leftovers samples from the trimming process were then used for other tests such as the Atterberg limits and the moisture content. The trimmed sample then was placed inside the ring shown in Figure 3.7 and assembled as in oedometer apparatus.

Figure 3.5: The sample was trimmed against the trimmer wall

Figure 3.6: Plastic casing of the sample was cut

Figure 3.7: The sample was placed inside the metal ring

3.2.2 Saturation Stage

In consolidation test, the specimens were subjected to a saturation process first as shown in Figure 3.8. In this stage, the specimens were left in the presence of water for 24 hours in the metal casing. This is however an assumption made by Terzaghi (1925) that all pores inside the soil is fully saturated with the water. The initial reading of the dial gauge was taken as a reference before the saturation process started. It has been observed during the saturation process that the soil tended to expand after the water is poured inside the metal casing. In order to keep the soil from swelling further, several light weights were added one by one until the dial gauge reading achieved the initial value before the saturation process shown in Figure 3.9. These applied pressures were added carefully and just enough to prevent the soil from further expansion. The soils were left for 24 hours of saturation and the reading was checked again until it achieved a constant value before the consolidation started.

3.2.3 Loading and Unloading Stages

The consolidation tests conducted consisted of 2 stages of loading and unloading processes. In each test, the specimen was loaded up until 1600 kPa and in between of the loading, the sample was unloaded at 400 and 800 kPa respectively. At 400 kPa, the sample was unloaded to 200 kPa before loaded it back to 800 kPa. After the load was reached at this point, the soil was unloaded back to 600 kPa. After that, the applied pressure was increased from 600 kPa to 6400 kPa. The summary of loading and unloading stages was presented in Table 3. In this test, the duration to complete loading and unloading stages will not be completed without the observation of creep behaviour for 24 hours at the end of loading or unloading stage. The compression and expansion values were recorded by an observation from the dial gauge reading. Creep is a condition where the soil experiencing adjustment in the soil fabrics. It is important to monitor the creep to ensure that the next compression of the soil is not due to creeping influence but it was due to compression by the weight loaded to the soil. A plot of settlement (mm) versus time (hour) and the gradient was observed before any intention to load or unload the. If the gradient was less than 0.002, then the next loading or unloading stages can be proceeded as the changes is not so significant and the soil has reached equilibrium state.

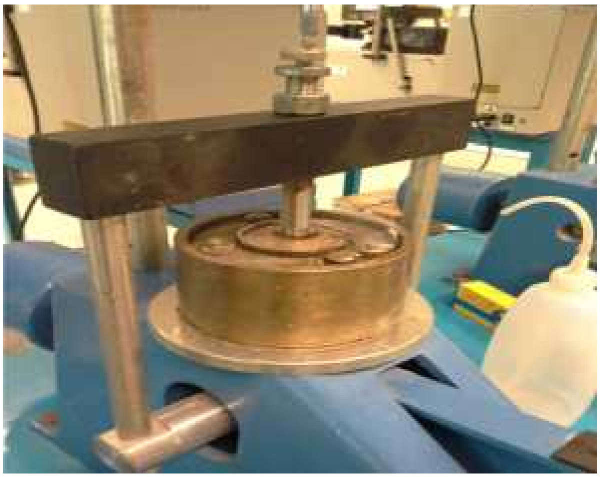

Figure 3.9: The sample was left for saturation

Figure 3.8: Small weights were added during saturation process

Table 3: Summary of loading and unloading stages

Pressure (kPa)	Stages	Pressure (kPa)	Stages	Pressure (kPa)	Stages
1	Loading	400	Loading	800	Loading
5	Loading	300	Unloading	1600	Loading
10	Loading	200	Unloading	3200	Loading
25	Loading	300	Loading	6400	Loading
50	Loading	400	Loading		
100	Loading	800	Loading		
200	Loading	600	Unloading		

4 Results and Discussions

4.1 Basic properties

The results for basic properties were presented in Table 4. These values were used in correlation with the depth of the samples taken. Since the samples were obtained from the different location, the trend in each characteristic can be observed through the correlation.

Table 4: Basic properties of the samples

Sample	Symbol	Depth (m)	Moisture Content (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index
Semantan 2.2 m	S1	2.2	50.24	33.16	55.75	22.59
Semantan 6.6 m	S2	6.6	49.37	29.66	55.25	25.58
Semantan 8.6 m	S3	8.6	51.21	30.77	58.5	27.73
Semantan 18.8 m	S4	18.8	50.29	36.27	54.8	18.53
Semantan 27.6 m	S5	27.6	24.26	27.08	54.5	27.41
Duyung Shallow 4.8 m	D1	4.8	36.97	28.28	50.80	22.52
Duyung Shallow 8.8 m	D2	8.8	42.22	28.42	52.46	23.58
Serendah 19.7 m	SR1	19.7	65.25	31.50	66.5	35

4.2 Result of moisture content

The values of moisture content were plotted against depth of the samples taken according to their location as shown in Figure 4.1. These values were then used in calculation of initial void ratio for the consolidation test. Based on the graph, the samples from Semantan shows the decrement in the moisture content as it go deeper compare to Duyung shallow. Since the samples used were in natural state, the preservation of moisture content is important to the accuracy of result obtained. The highest values of moisture content come from Serendah with the value of 65 %. The variation of moisture content in the samples may result from different layer of formation during the depositional of the soil. The scattered values in moisture content of all samples can be related to the different of fines content in the soil.

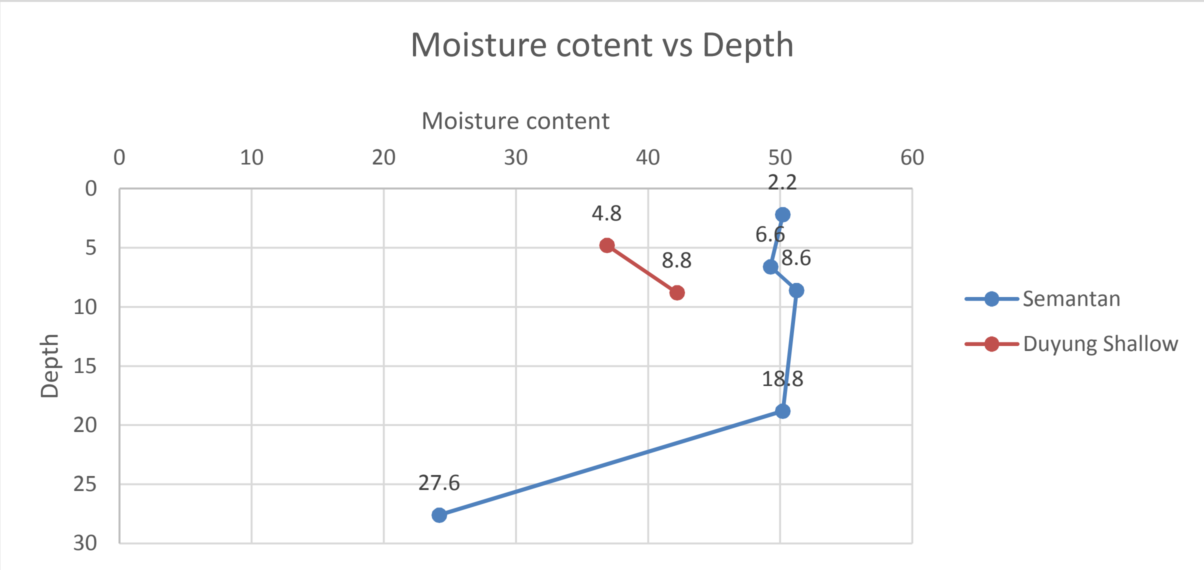

Figure 4.1: Moisture content vs Depth

4.3 Result of Atterberg Limit

Another basic properties obtained from the test performed were Atterberg Limit. The graphs of the Plastic Limit, Liquid Limit and Plasticity Index against depth were shown respectively in Figure 4.2, 4.3 and 4.5. Atterberg limit can be used as an indication of the soil transformation upon receiving certain amount of water. Based on the graph shown, the Plastic limit value for Semantan location range from 27 to 36 %. The Plastic Limit values for Duyung Shallow shows and equal value of 28%. The Liquid Limit values for Semantan samples ranged from 54 – 55% whereas the Duyung samples have value of 50-52%. A similar result was also obtained from the same location conducted by Thian and Lee in 2014 for offshore soil from Terengganu, Malaysia. The liquid Limit value for soil tested is about 54% whereas the Plastic Limit value is 27%. These values were close to Semantan (27.6 m) and Duyung shallow as both of them were obtained from east coast of Malaysia water region. Benson and Trast (1995) suggested that the liquid and plastic limit values are directly related to mineralogy and clay content.

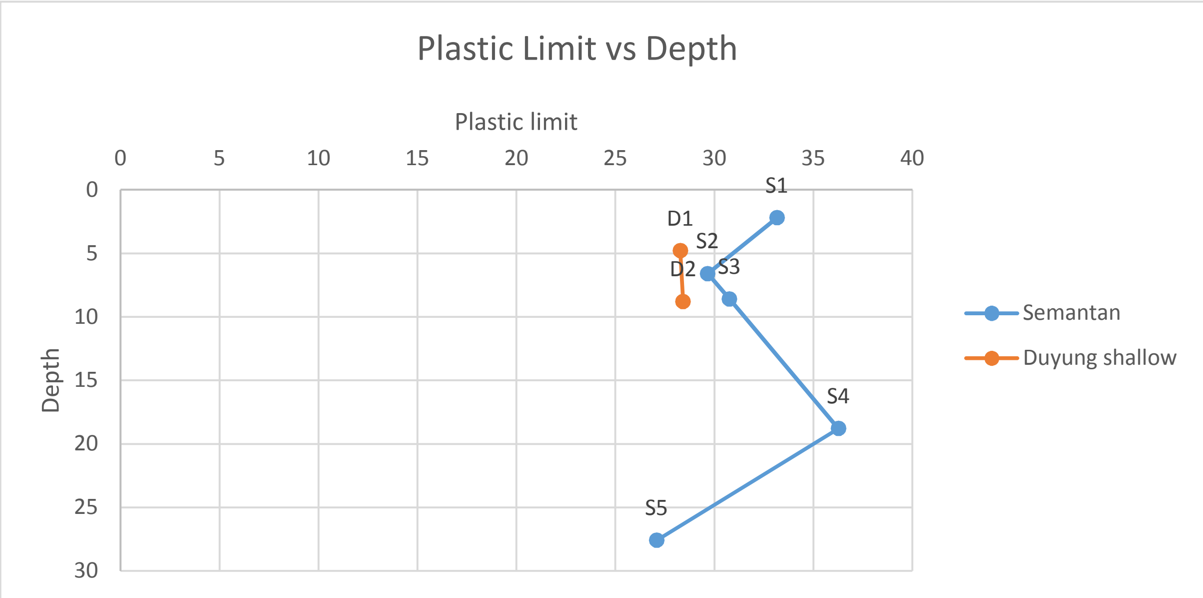

Figure 4.2: Plastic Limit vs depth

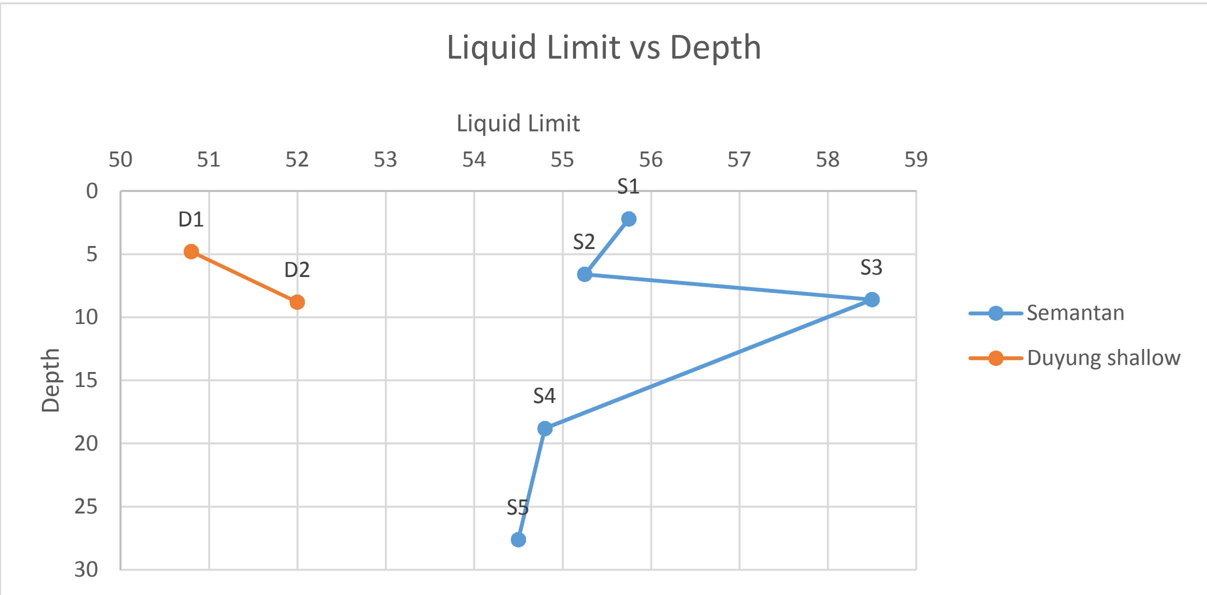

Figure 4.3: Liquid Limit vs depth

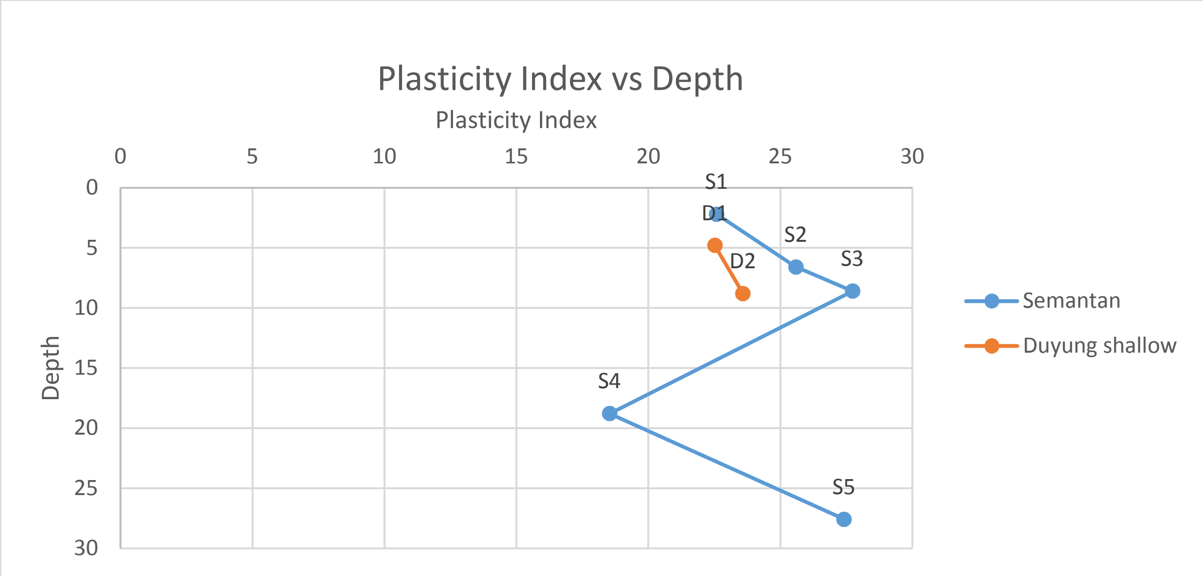

Figure 4.4: Plasticity Index vs Depth

4.4 Hydrometer

The particle size distribution curves for 6 clay samples tested were plotted as shown in Figure 4.5. These curves were used to classify the soil by their percentage finer present. The summary of soil constituent was tabulated in Table 5. From the table, it shows that the sample from Semantan has an increased in clay percentage as it goes deeper compare to silt and sand fraction. Sample from Serendah has maximum value for the clay percentage compare to other samples. The behaviour of the samples tested governed by the amount percentage finer found in the soil.

Table 5: Summary of soil constituent

Sample	Depth (m)	Clay (%)	Silt (%)	Sand (%)
Semantan 2.2 m	2.2	25	63	12
Semantan 6.6 m	6.6	40	52	8
Semantan 8.6 m	8.6	48	47	5
Semantan 18.8 m	18.8	60	35	5
Duyung Shallow 4.8 m	4.8	47	46	7
Serendah 19.7 m	19.7	53	42	5

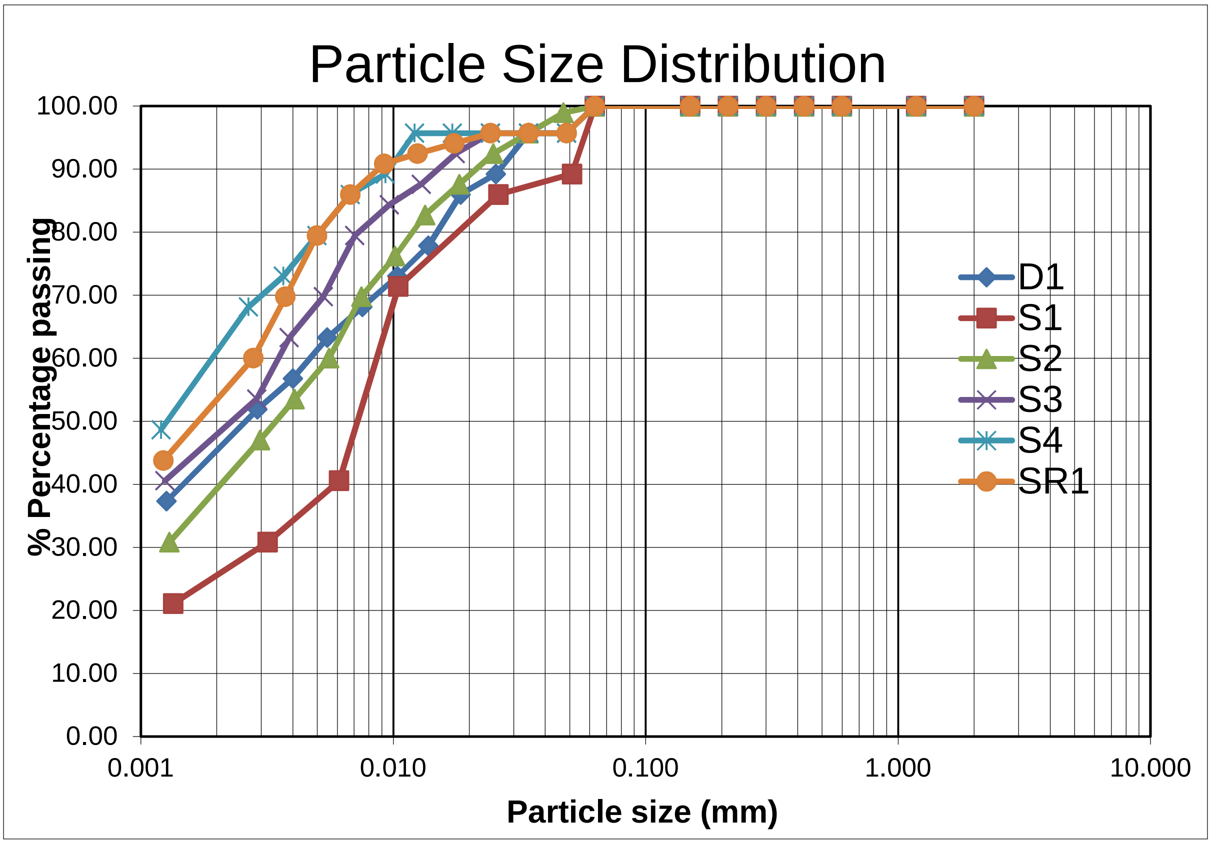

Figure 4.5: Particle size distribution curves for the marine clay samples

4.5 Specific Gravity, *Gs*

The mean value for specific gravity, Gs of the marine clay was 2.6 – 2.65. The results obtained from the experiments show the value Gs ranged from 2.4 – 2.66 as tabulated in Table 4.3 and no significant difference in Gs was observed in the results.

Table 6: Specific gravity of soils

Sample	Specific Gravity, Gs	Sample	Specific Gravity, Gs
Semantan 2.2 m	2.42	Semantan 27.6 m	2.42
Semantan 6.6 m	2.66	Duyung Shallow 4.8 m	2.68
Semantan 8.6 m	2.61	Duyung Shallow 8.8 m	2.68
Semantan 18.8 m	2.62	Serendah 19.7 m	2.48

4.6 Scanning Electron Microscope (SEM)

The studies of structure in this project were presented by the result of SEM. Microstructural studies of Semantan clay at 2.2 m shows the presence of flocculated structure with large particle represented by the silt particle shown in Figure 4.6. Sample from Serendah at 19.7 m has flocculated structure with the particles are closely packed together shown in Figure 4.7. The overall structures of Duyung (8.8 m) reveal flocculated structure shown in Figure 4.8. These fabric studies are important to the behaviour of soil during deformation because of the structure of the soil itself can affect the compressibility and swelling ability during compression and expansion. Flocculated structure has a great change in fabric compare to deflocculated structure once it reached the maximum yielding stress. Other characteristic associated with flocculated structure is the soil tend to have a high initial void ratio.

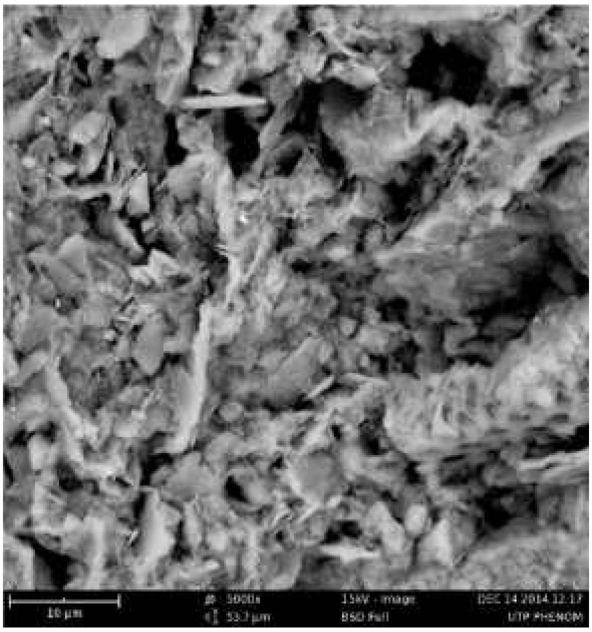

Figure 4.6: Scanning electron micrograph of Semantan 2.2 m

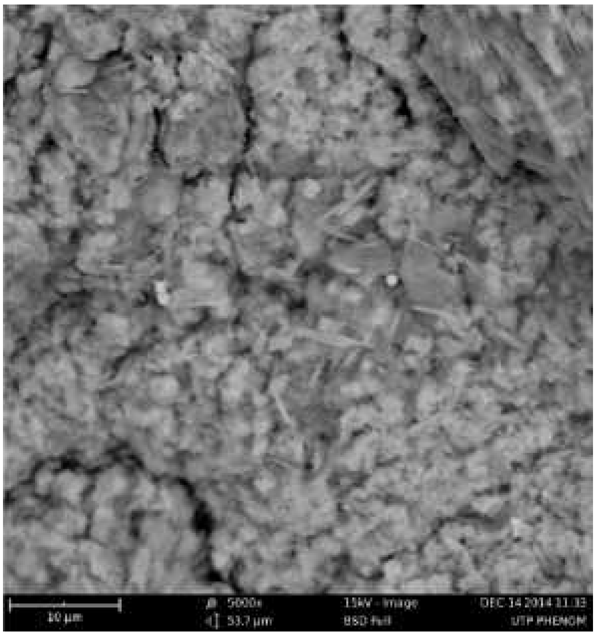

Figure 4.7: Scanning electron micrograph of Serendah 19.7 m

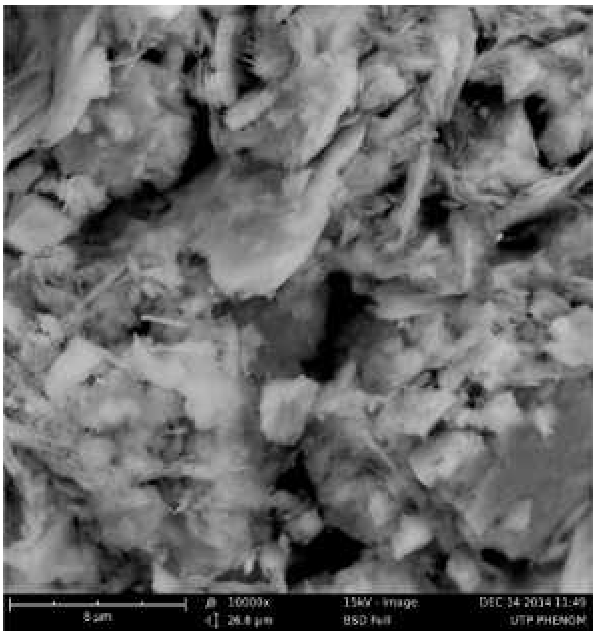

Figure 4.8: Scanning electron micrograph of Duyung Shallow 8.8 m

4.7 Compression curves

Typical *e: log p* curves were plotted for the Semantan, Duyung and Serendah clays as shown in Figure 4.9. The curves plotted as result from the loading and unloading process from the consolidation test. Since the samples used were in natural state, the moisture content has great influence in the initial void ratio. The preconsolidation pressure were then obtained from the graph as a comparison from the past overburden stress. Based on the curves, the specimens were subjected to different loading sequences in order to investigate for the incremental loading on compression behaviour. 4 specimens were tested up to 1600 kPa while the other 3 samples were loaded up to 3200 kPa. There is an exception for the Duyung 8.8 m where it was loaded to 6400 kPa. The variation in loading sequence shows the behaviour of all samples that tend to converge to a similar line provided that they were having similar loading and unloading sequence. The results of preconsolidation pressure are listed in Table 7.

Table 7: Preconsolidation pressure of samples

Samples	Depth (m)	Symbol	Initial void ratio	Preconsolidation pressure, *Pc* (kPa)	Maximum pressure tested (kPa)
Semantan	2.2	S1	1.59	400	1600
	6.6	S2	1.39	200	1600
	8.6	S3	1.32	300	1600
	18.8	S4	1.37	480	3200
	27.6	S5	0.51	500	3200
Duyung Shallow	4.8	D1	1.08	300	3200
	8.8	D2	1.17	350	6400
Serendah	19.7	SR1	1.81	300	1600

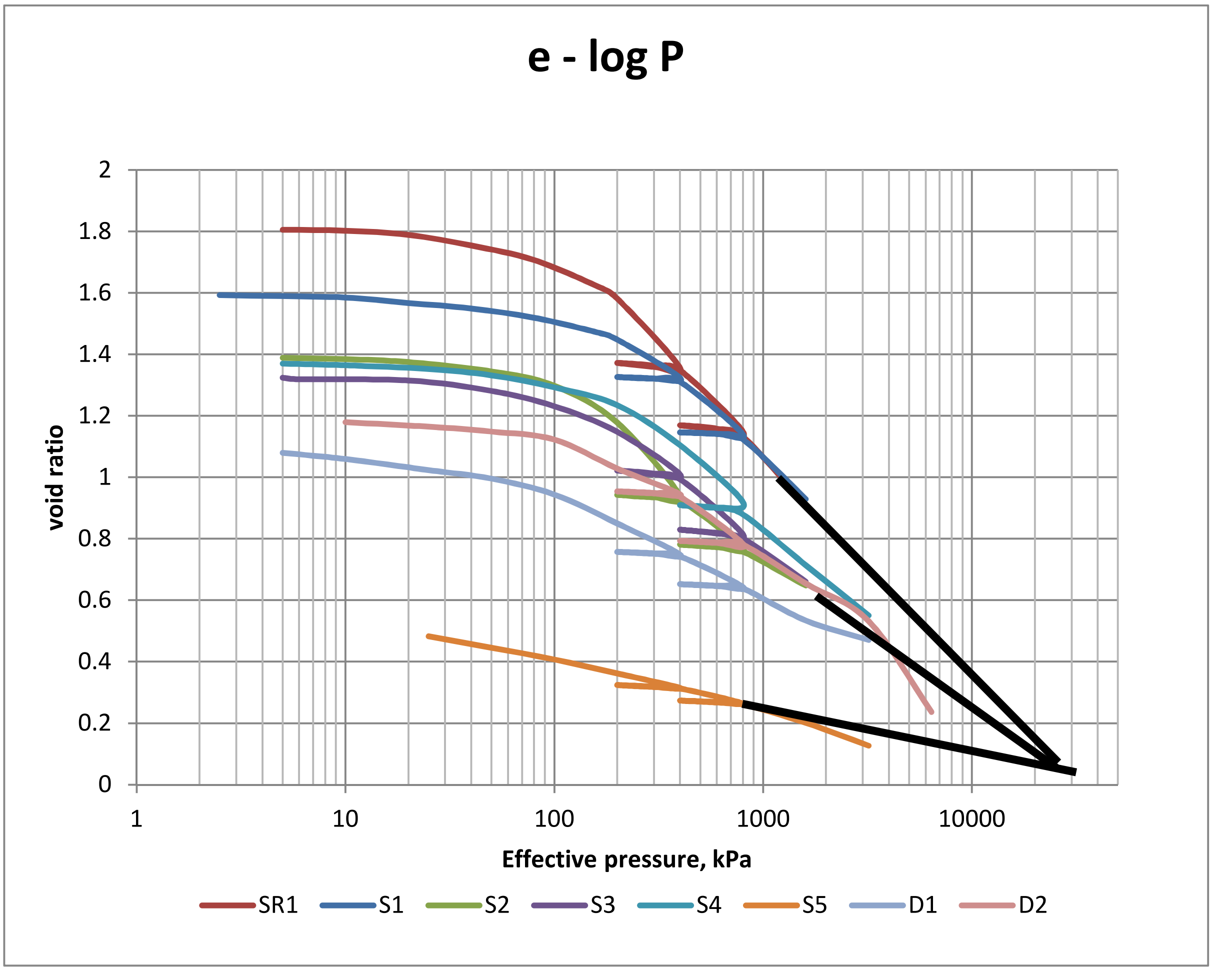

Figure 4.9: Compression curves for 8 different samples

4.8 Compression curves

One of the essential parameters obtained from *e: log p* curve is compression Index, Cc. This parameter will be used in calculation of primary settlement. The compression index is described as the linear portion of e log p curve during the loading stages. The swelling index is determined by the slope of linear portion during the unloading stages. The values for both compression and swelling index were tabulated in Table 8. According to Mitchell (1993), soils having compression index less than 0.2 represented soils of slight to low compressibility. A compression index of 0.2 to 0.4 are for soils of moderate to intermediate compressibility and the soils are considered to be highly compressible if the compression values greater than 0.4. Some correlations are made between the compression index and Liquid Limit (LL) as shown in Figure 4.10. The relationship between the Cc and LL produced a straight line for both Semantan and Serendah area. The equations are described as below:-

Cc = 0.0372LL – 1.58 (Semantan) Equation 4.1

Cc = 0.0483LL – 2.11 (Duyung Shallow) Equation 4.2

The *Cc* value has been correlated with index properties by various researcher in numerous ways since Terzaghi proposed the well-known relationship of Cc = 0.009(LL – 10). According to Leroueil et al. (1983), the compression Index is influenced by the sensitivity of natural clays in which it can be generally related to the void ratio of the soil. Figure 4.11 shows the relationship between void ratio and compression index for both specimens of Semantan and Serendah. The linear relationship was given in a form equation described as below: -

Cc = 0.398e_o – 0.0073 (Semantan) Equation 4.3

Cc = 0.672e_o – 0.387 (Duyung Shallow) Equation 4.4

The swelling index, *Cs* is also an important compressibility parameter to be known. Cs is defined in the same way as *Cc* except that it is applied to the linear portion during unloading phase of the oedometer test. In this project, the specimens were subjected to twice unloading phases thus the Cs of both phases were then been compared. The values for all *Cs* during the unloading stages are recorded in Table 8. Figure 4.12 shows the relationship between the ratio of compression and swelling index to depth according to specific samples. Semantan samples show a decrement in ratio of *Cc/Cr* until it reach 18.8 m. Duyung samples however show an increase in the ratio as it goes deeper.

Table 8: Swelling and Compression Index

Sample	Depth (m)	Compression Index, Cc	Swelling Index, Cs	
			400 - 200 kPa	800 - 600 kPa
Semantan 2.2 m	2.2	0.564	0.066	0.066
Semantan 6.6 m	6.6	0.564	0.066	0.099
Semantan 8.6 m	8.6	0.542	0.099	0.133
Semantan 18.8 m	18.8	0.611		0.033
Semantan 27.6 m	27.6	0.177	0.066	0.066
Duyung Shallow 4.8 m	4.8	0.341	0.066	0.066
Duyung Shallow 8.8 m	8.8	0.399	0.033	0.033
Serendah 19.7 m	19.7	0.797	0.033	0.066

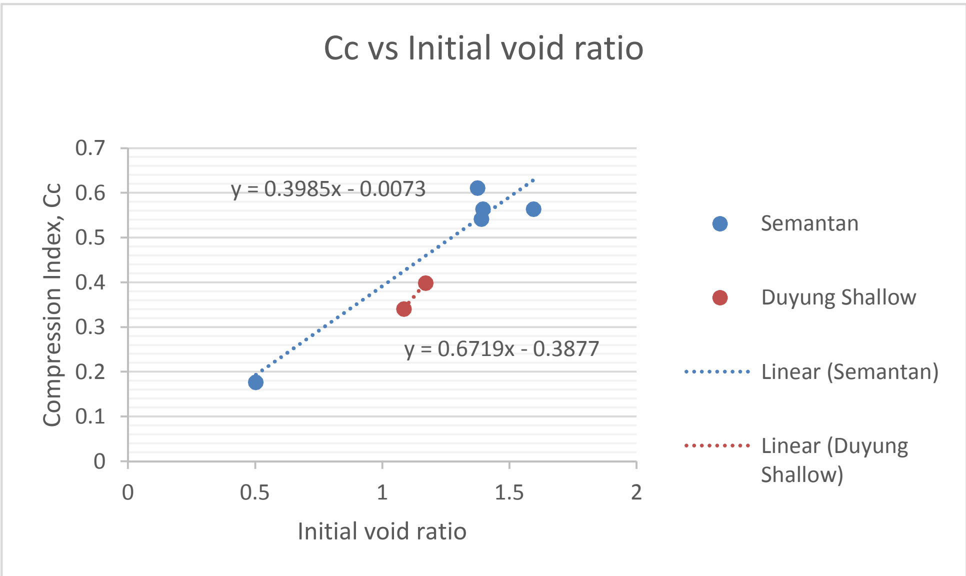

Figure 4.10: Relationship between Compression index and Liquid Limit

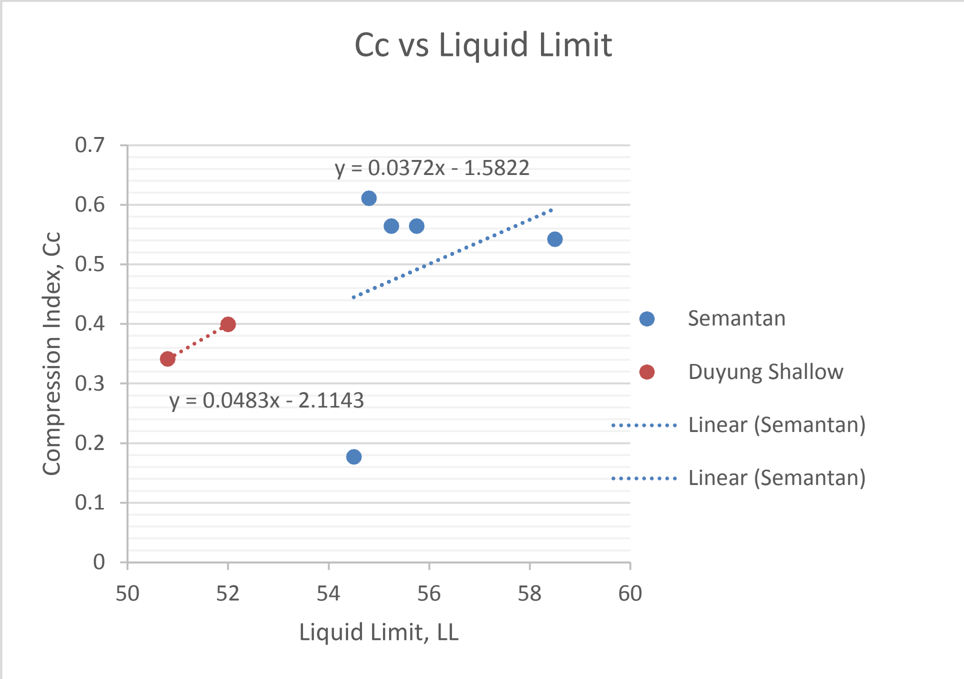

Figure 4.11: Relationship between Compression index and Initial void ratio

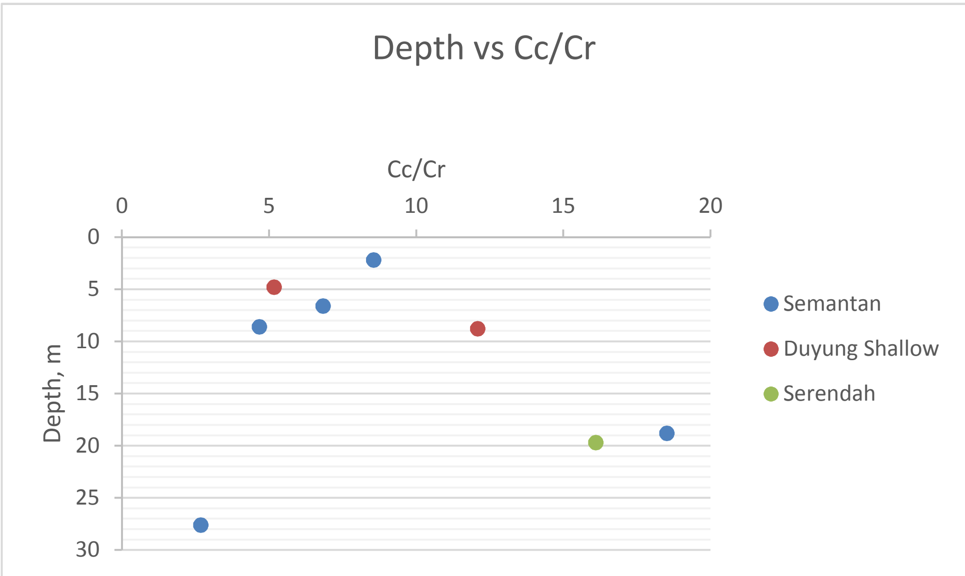

Figure 4.12: Ratio of Cc/Cr vs depth

4.9 Effect of percentage finer on compression curves

The behaviour of soil is well related to the particle composition presence inside it. The influence of fines content and type can be known through the compression curves of consolidation test. Lupogo (2012) claimed that the soil containing fines is more compressible and increases tendency to creep. The hydrometer test done in this project shows the amount of percentage finer of specific soils as shown in Table 5. S1 and S2 soils are dominated by more number of silt percentage compare to S3 and S4. The effect of more number in silt percentage can be seen through the initial void ratio. S1 and S2 and has more number of initial void ratio compare to S3, 4 and 5. As it goes deeper, more clay soil was found as shown in Table 5. When the clay content is higher, the soil is said to fill with the clay particle, which leads to considerable decrease of void ratio.

In the case of Semantan samples shown in Figure 4.13, the compression curves show the high compressibility of soil taken from 18.8 m. The soil contains more percentage finer besides high in initial void ratio. S1 which was taken from 2.2 m below seabed showed a stiff behaviour from the more number of silt content. S5 however is said to be a denser soil because of its less compressible compare to other samples. This might due to the soil may experience great amount of overburden stress during the deposition. The soil structures were compacted that leads to low initial void ratio. The curves can also be observed through the preconsolidation pressure obtained. S5 has the highest Pc compare to the other samples tested. The SEM performed in Semantan 2.2 m shows the effect microstructures on the compression behaviour. The flocculated structure of soil resulted in higher void ratio besides having higher moisture content.

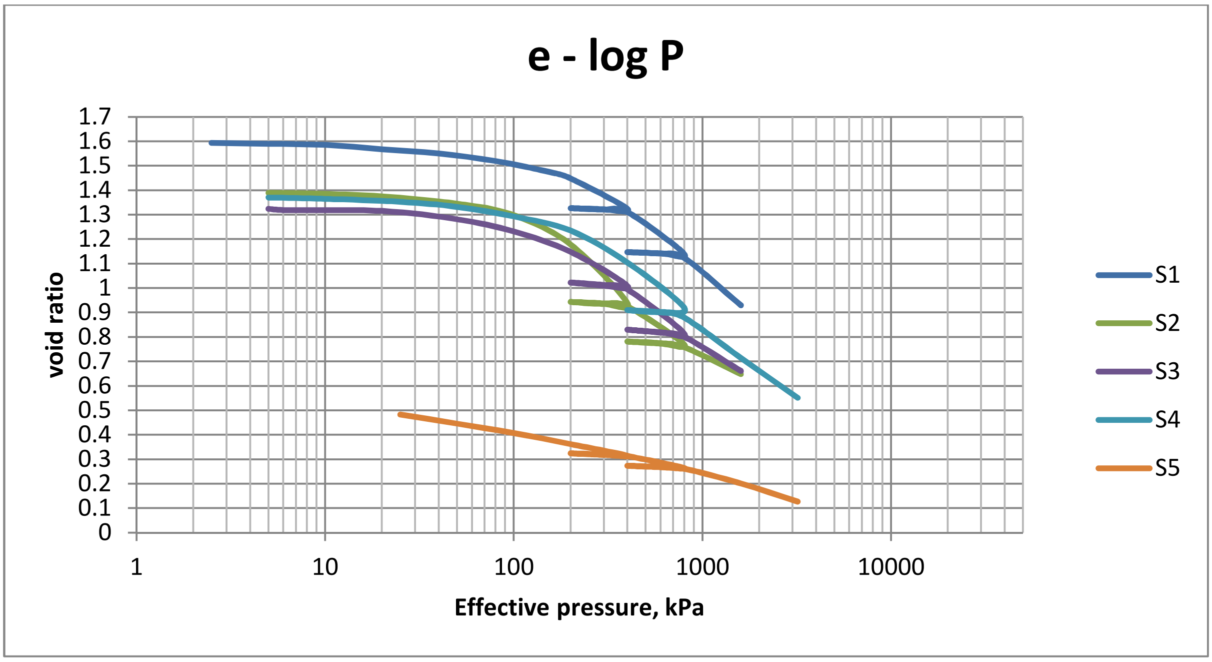

Figure 4.13: Compression curves for Semantan samples

Duyung Shallow samples shown in Figure 4.14 showed significant increase in the initial ratio as the depth taken increased. The percentage finer shown in the hydrometer result gives more amount of clay content in D2 compare to D1. The behaviour of soil is controlled by the percentage fine presence inside the soil. As the pressure increase, the structure of soil is said to resist the pressure until yielding was achieved. During the stage the soil is less compressible before the yielding was passed. The fines content then moved and filled up the void hence governed the compressibility of soil. The microstructures of soil that is flocculated shown in SEM result indicate that the soil is highly compressible.

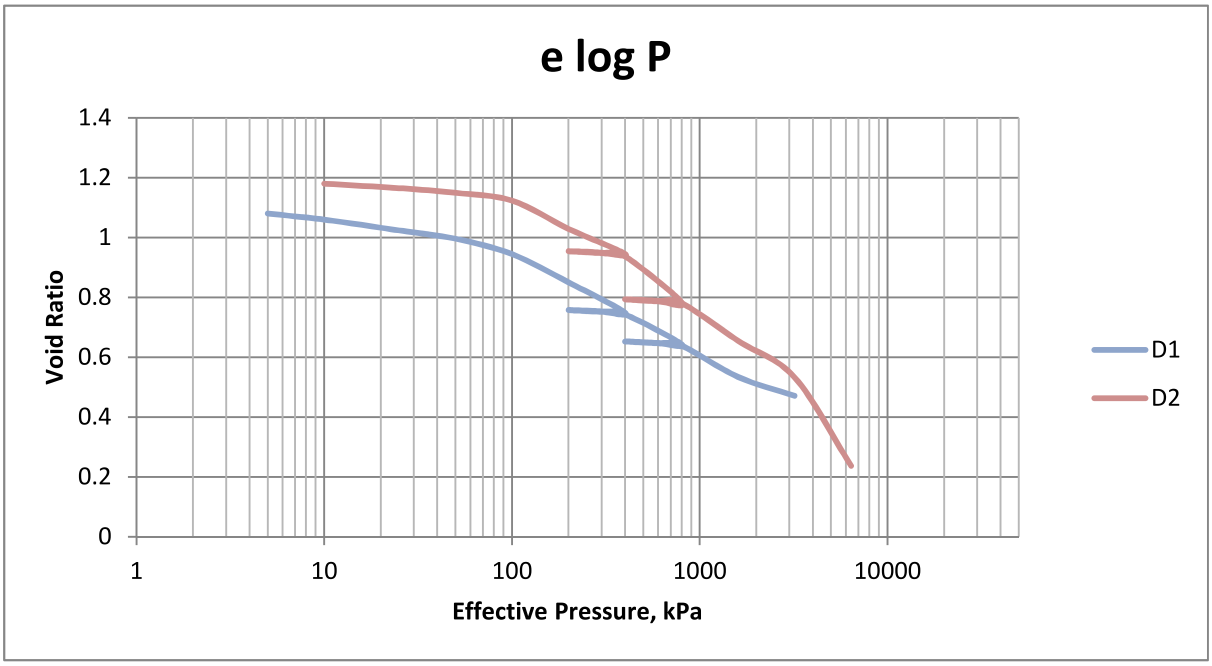

Figure 4.14: Compression curves for Duyung Shallow samples

5 Conclusions

Oedometer test have been performed on natural marine clays in order to investigate the compression behaviour of different depth. The influence of fines content and type can also be determined from the consolidation test. The increased in clay content of Semantan samples from 25 – 60 % led to the increment of Cc from 0.564 – 0.611. The natural state of soil implies that there is no alteration in soil structure which can govern the compression behaviour.

Based on the result obtained, the soils were varied in term of Initial void ratio. The compressibility of soil is mainly controlled by the structure and percentage finer. Soil that has high in clay content exhibit high compressibility characteristic provided the soil is normally consolidated.

As the soil depth is deeper, the behaviour of soil changes due to moisture content. Soil with higher moisture content has low initial void content thus shows a stiff behaviour towards the graph plotted. The highest value of moisture content recorded from this project comes from Serendah which is 65.25 %, thus the soil is having highest initial void ratio of 1.81. The steeper the graph indicated the soil is more compressible within small range of effective pressure. Several geotechnical parameters were then determined from the graph for settlement calculation.

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