CERTIFICATION OF APPROVAL

The Mechanical Behaviour of Reconstituted Marine Clay in Semantan Field

by

Muhamad Noor Ariff Bin Abd Rashid
16860

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Approved by,

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Ms Niraku Rosmawati Ahmad

(Project Supervisor)

UNIVERSITI TEKNOLOGI PETRONAS

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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.

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(MUHAMAD NOOR ARIFF BIN ABD RASHID)

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ABSTRACT

Throughout the years, the positive growth in Malaysia economy has increased the usage of new technology in area consists of marine clay. The condition of the soil which very soft creates complexities in design due to low bearing capacity and high compression behaviour.5 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 0.341 – 0.409 for reconstituted samples as the percentage finer increased, thus the compressibility of soil is mainly controlled by the fines particles. The preconsolidation pressure for two reconstituted samples are 3 kPa. In conclusion, the compression behaviour of reconstituted soil is close related with the basic soil properties and non-structure effect of soil.

1.0 Introduction

1.1 Background study:

According to Basack (2009), the soil is classified as marine soil if it is found in deep ocean bed. The properties of soil depend significantly on its initial conditions. The properties of saturated marine soil differ significantly from moist soil and dry soil. Marine clay contains a high proportion of calcium, silica, aluminum, iron and other traces material. Besides that, Basack (2009) added that the marine clay is made of microcrystalline in nature and clay mineral such as chlorite, kaolinite and illite and non-clay mineral like quartz and feldspar which are present in the soil.

Leroueil et al. (1985) classified the compressive behaviour of natural clays into two states: the intact state as it occurs in natural deposits; and the destructured state referring to the breakdown of the original clay structure when submitted to volumetric or shear deformations. The former is related to elastic deformation whereas the latter is related to elasto-plastic deformation. It is a common practice to refer clay as reconstituted clays when assessing the effect of soil structure on the mechanical behaviour of natural sedimentary clays. According to Burland (1990), the properties of reconstituted clays are termed 'intrinsic properties' since the properties itself are inherent and independent of natural state. However, Burland (1990) further added that the properties of natural clay are differ from intrinsic properties due to influence of soil structure which categorizes in fabric and bonding. The compression curve of reconstituted marine clays is intrinsic properties (no bonding) and used as a frame of reference for natural clays and in-situ properties. The term 'intrinsic' is used to describe the properties of clays which have been reconstituted at a water content amount between $1.0w_L$ and $1.5w_L$ (prefer $1.25w_L$) without oven drying or air drying then consolidated. Besides that, Burland (1990) proposed a concept of void index and demonstrated its ability to interpret the compression behavior of reconstituted clays and influence of natural clays properties.

Tavenas (1979) mentioned that the term "soil structure" is defined by both particle arrangement (fabric), association and inter-particle forces (bonding). The resistance of soil structure is importance in defining the difference of engineering behaviour of natural soils between structured and destructured (remoulded). The difference of void ratio between natural clays and reconstituted clays at the same stress level often increases with increasing consolidation stress up to the consolidation yield stress, but decreases when the applied stress level is larger than the consolidation yield stress, (Liu & Carter, 1999).

1.2 Problem Statement

Throughout the years, the positive growth in Malaysia economy has caused the increase of number of projects in construction site, offshore and onshore platform. In terms of geotechnical engineering, the foundation of the structure plays an important role for every structure. The role of geotechnical engineer is important in enhancing construction progress. Dealing with soil is one of the major aspects need to be focused for any construction. The solutions of many geotechnical issues in the construction site are very much directly or indirectly related to the understanding of problematic soil. Marine clay has proven to be one of the issues in the west coastal of Peninsular Malaysia. Thus, it is very important to know the basic properties of marine clay. Besides that, marine clays also can be found in deep ocean floor. This will lead to problematic issues such as installation of jacket platform which deal with marine clay. The problematic issue with marine clay is that it has a low bearing capacity and high deformation behavior. The condition of the soil which is very soft creates complexities in design, construction and maintenance of structures.

1.3 Objectives:

The objectives of this project is to observe the behavior of reconstituted marine clay towards different stress loading applied on the soil. The knowledge of this behavior is important as to predict the compression behavior of marine clay. Thus, the objectives are listed as below.

1. To study the basic properties of reconstituted marine clay.
2. To study the effect of loading to the compression behavior of reconstituted marine clay.
3. To study the effect of void ratio on compression behavior of reconstituted marine clay.

2.0 Literature Review

2.1 Origin of Marine clay

Marine clay is a type of clay found in coastal regions around the world. In the South East Asian, Malaysia is one of the countries which have marine clay deposited along the coastal areas. For example, Rahman et al (2013) studied marine clay which are deposited along coastal area of Kuala Muda, Kedah. Clay particles can self-assemble into various configurations, each with totally different properties. A clay is formed by a group of crystalline minerals of kaolinite, illite and montmorillonite. The particle that made up the clay structure 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 structure of particle in clay playing a role in differentiating the intact state and the reconstituted state. Burland (1990) mentioned that the properties of natural clay (intact state) differ from intrinsic properties (reconstituted state) due to the influence of soil structure (fabric and bonding). Following by Mitchell (1976) stated that the term structure means the combination of fabric (arrangement of particles) and interparticle bonding. There are 2 types of soil fabric commonly seen in the soil and they are flocculated and dispersed. Figure 2.1 shows the particles arranged in perpendicular to each other in flocculated structure while the dispersed particle are arranged in parallel arrangement in a layer. This arrangement will affect the void ratio if the spaces inside the soil varies. When clay particles initially dispersed in water, the particle will close to each other and flocs with face to face contact. If the particle are exposed in high salt environment, the flocculation will be more towards face to face contact.

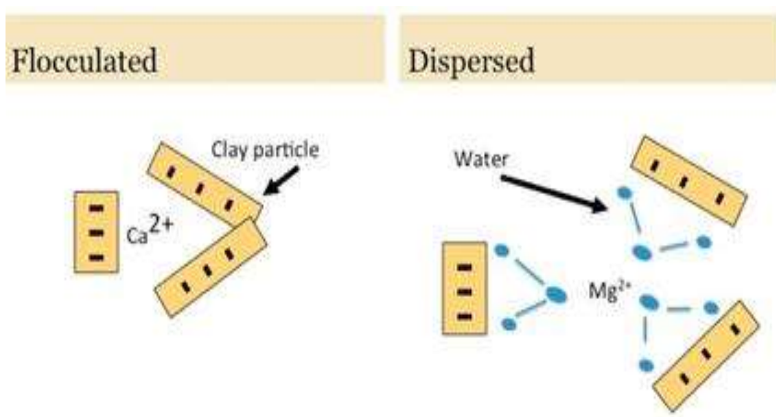

Figure 2.1: Marine Clay structure based on Lambe (1958)

2.2 Reconstituted marine clay properties

Soil structures that which include particles, particle groups and their associations, usually develop during the depositional and postpositional processes of the sediment. These structures are bond together with the interparticle forces and applied stresses. A considerable number of experimental and theoretical studies on the soil structure have been carried out from both macroscopic and microscopic viewpoints by Burland (1990). Most studies were performed by comparing the mechanical behaviors between undisturbed and reconstituted samples. The mechanical behavior of reconstituted clays is termed as intrinsic properties, is the basic or inherent properties of given clays prepared in a specific manner, whereas the natural clays can be regarded as a combination of the behavior of the corresponding reconstituted clays and the structural effects of the soils. Many experimental studies show that the mechanical behavior of natural clays is substantially different from that of the corresponding reconstituted clays due to the soil structure. 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. The strength of undisturbed samples is affected by several factors, including void ratio, soil structure and stress history. Therefore, when the strengths are compared between undisturbed and corresponding reconstituted samples at the same confining pressures, combined effects of void ratio and soil structure should be considered.

2.3 Compression Behaviour of Reconstituted Marine Clay

Hong et al. (2010) performed 42 oedometer tests to investigate the compression behaviour of three reconstituted clays (Lianyungang clay; Baimahu clay; and Kemen clay) with a wide range of initial water contents. The test results illustrate an inverse "S" shape of the $e - \log \sigma_v'$ compression curves, as typically shown in Figure 2.2, similar to those of many natural clays. This similarity in shape of the $e - \log \sigma_v'$ compression curves between natural clays and reconstituted clays indicates that there is a particular stress for reconstituted clays which is similar to the consolidation yield stress for natural clays. As for the compression behaviour of natural clays, the compression curves of reconstituted clays can be divided into two lines which intersect at a particular stress. Small compressibility occurs when the effective stress is lower than this particular stress, which acts in a similar role to the consolidation yield stress of natural clays. Hong et al. (2010) termed the particular stress as suction pressure. He termed as the remoulded yield stress to avoid confusion with descriptions of the behaviour of unsaturated soils.

As shown in Figure 2.2, the compressibility of a reconstituted clay increases dramatically when the effective stress is higher than the remoulded yield stress. For a given reconstituted clay, the compressibility of a reconstituted clay is affected by the starting point of the compression line whenever the effective vertical stress is higher than the remoulded yield stress. The e_{yr} is the void ratio at the remoulded yield stress. Burland (1990) defined compression index as:

$$C_c^* = (e^*_{100} - e^*_{1000}) \qquad (1)$$

Where e^*_{100} and e^*_{1000} are the void ratio of the reconstituted clays at effective vertical stress σ_{yr}' of 100 kPa and 1000 kPa. The compression index Cc* depends on the values of the remoulded yield stress and the void ratio at the remoulded yield stress.

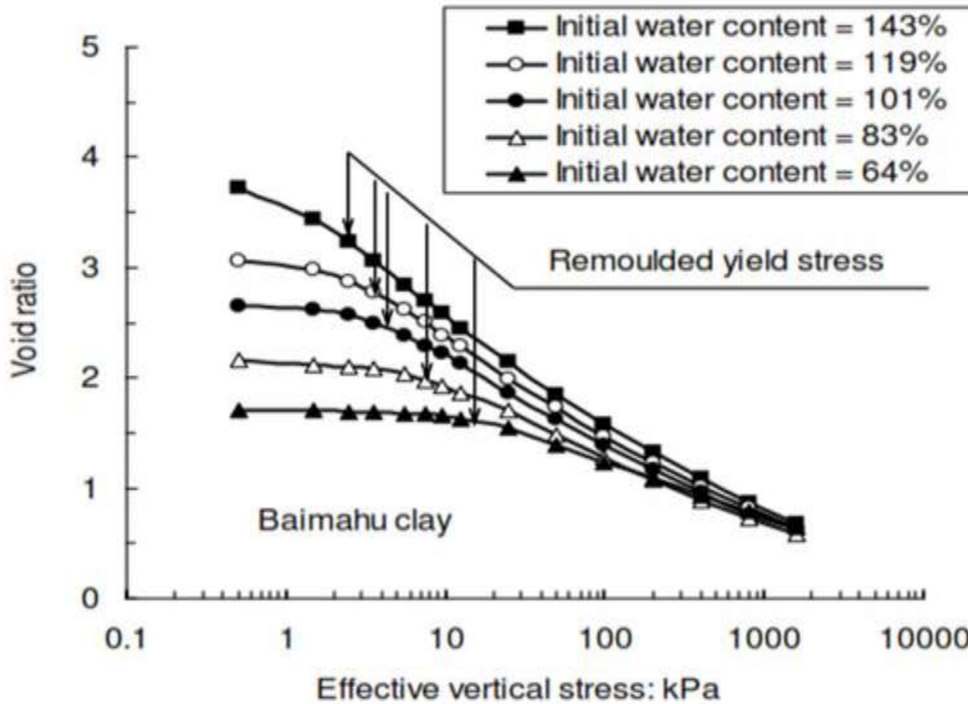

Figure 2.2: Compression curves of reconstituted Baimahu clay at different initial water content based on Hong et al (2010).

Dean (2013) carried out several oedometer test of 7 samples of marine clay deposit of soft soil collected from Yangtze Rver Delta, China. Figure 2.3 depicted the relationship of compression index, C_c and the void ratio e_{10} based on the oedometer tests on the seven different marine soft clays. The test results of reconstituted samples as shown in Figure 2.3(a) indicated that there are a unique linear relationship between the compression indices C_c and the void ratio e, though the data is slightly scattered. However, the relationships between compression index, C_c and the void ratio, e for undisturbed samples depended on the sample quality which are shown in Figure 2.3(b). For the undisturbed samples with very poor quality, this relationship is almost the same as that of reconstituted samples. But for the poor or fair samples, the fitting line has a larger slope than that of very poor quality samples. The higher the sample quality, the larger the slope of the fitting line.

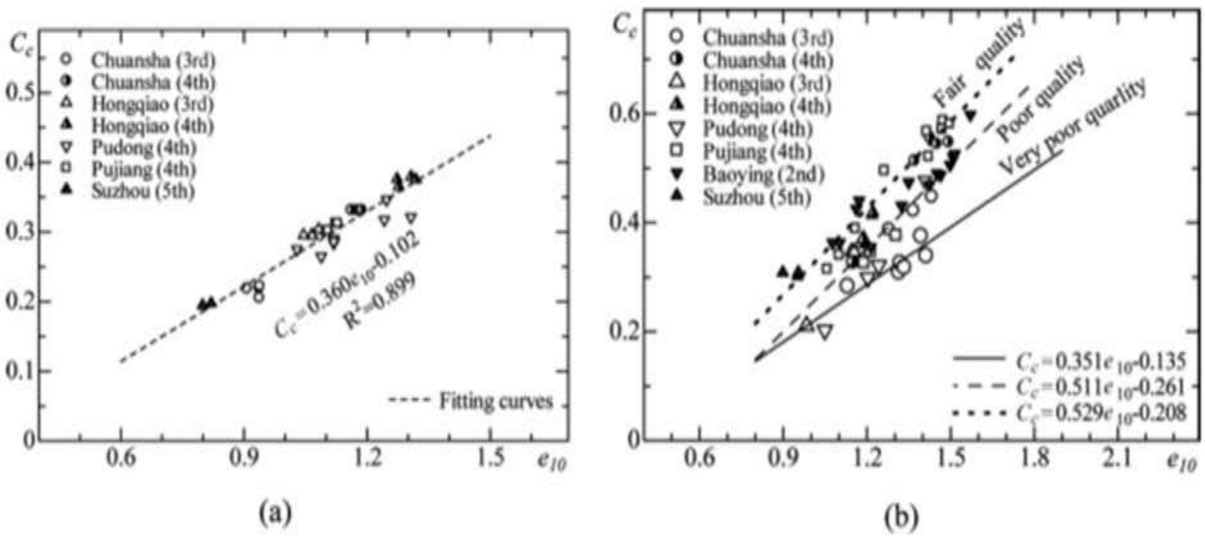

Figure 2.3: Compression index C_c versus Void Ratio e_{10} from oedometer test (a) Reconstituted (b) Undisturbed Sample Dean (2013)

Dean (2013) proposed the reference void ratio e_{10}^* is defined by a void ratio at the effective stress of 10 kPa by extending the linear section of the compression curves during the post-yielding which is shown in Figure 2.4. The value of e_{10}^* is introduced as a soil fabric index to analyze the results of compression and shear tests. The void ratio (e_{10}) of the samples at effective stress of 10 kPa is related to the soil structure (i.e., fabric and bonding) and stress history. For the reconstituted samples used in the tests, $e_{10}*$ can be used as an index to measure the soil fabric, as shown in Figure 2.3(a), because the samples are experienced the same pre-consolidated pressure of 70 kPa during their preparation processes and no bonding exists. For the undisturbed samples used in the tests, $e_{10}*$ cannot be used as a fabric index, as shown in Figure 2.3(b), because the samples have different structural yield stresses and the different bonding exists. In order to eliminate the effects of the stress history and the bonding, a reference void ratio e_{10}^* is proposed, which can be used to solely measure the soil fabric of different clays.

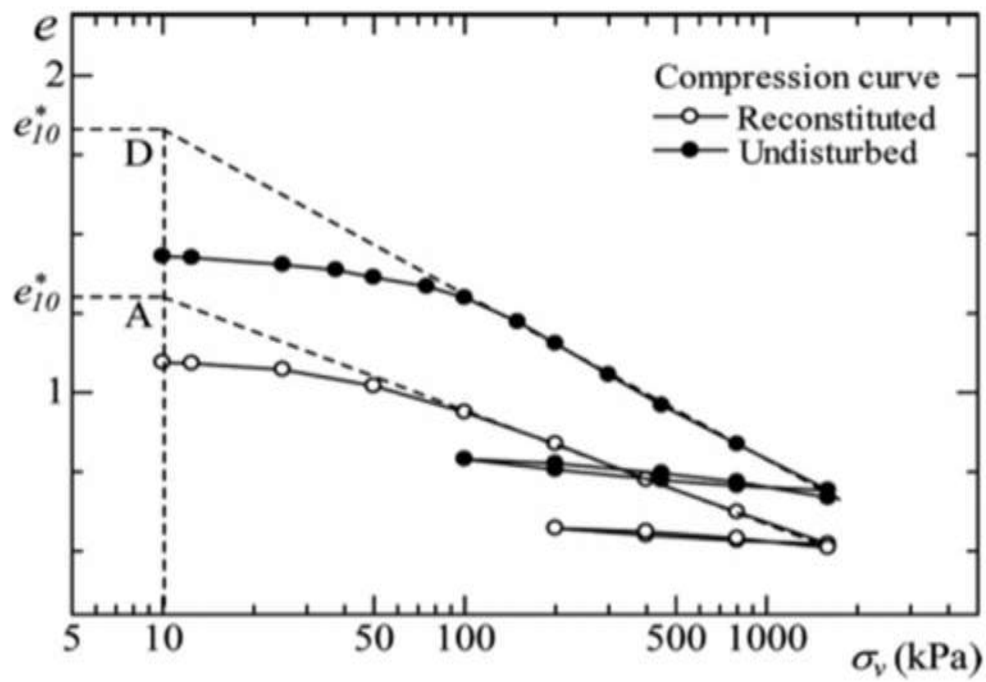

Figure 2.4: Vertical effective stress against void ratio Dean (2013)

2.4 Compression ratio, Cc/Cs of Reconstituted Marine Clay

Dean (2013) mentioned the value of Cc/Cs for reconstituted samples is significantly larger than undisturbed samples. This means that the value of swelling index, Cs for undisturbed samples are much larger than reconstituted samples since the value of compression index, Cc of reconstituted samples is lower than undisturbed samples. The swelling index of undisturbed samples are related to the unloading stresses and the unloading stress for all the undisturbed samples is about 1600 kPa, which is greater than that of the corresponding reconstituted samples. The larger the unloading stress, the more the soil structures are destroyed. Hence the undisturbed sample unloaded at larger stress should have larger swell indices Picarelli (1991).

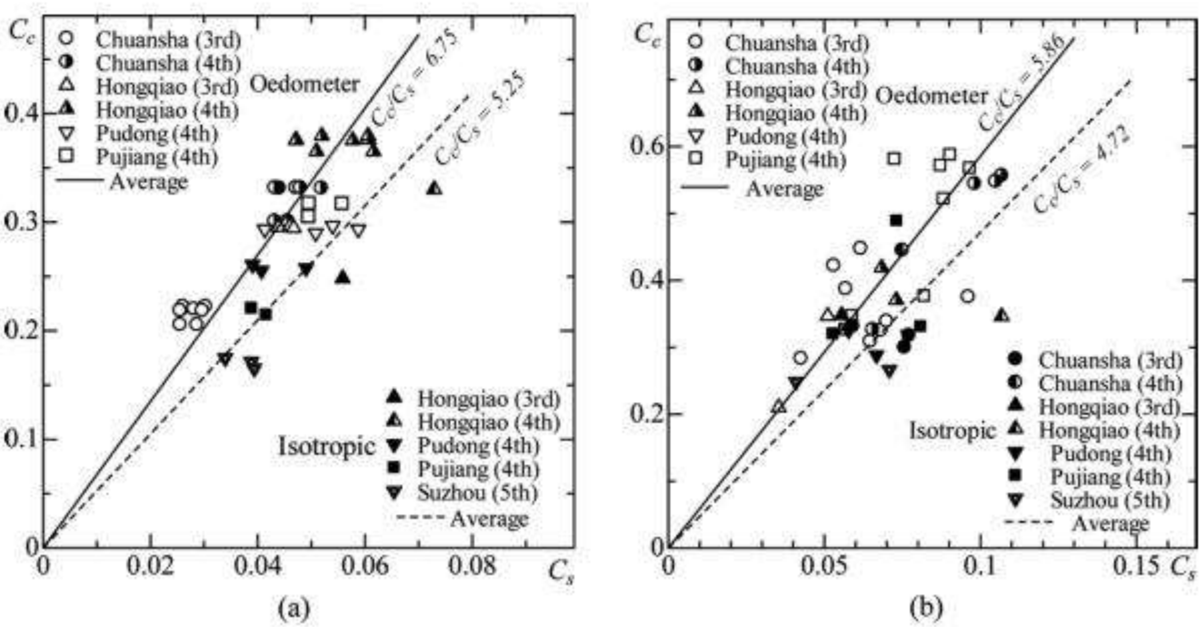

Figure 2.5: Compression Index Cc versus Swelling Index Cs for (a) reconstituted samples (b) intact samples based Dean (2013)

2.5 Effect of water content

Figure 2.6 shows an illustration of the concept of e_1 for a given reconstituted clay with different initial water contents. e_1 is the void ratio at effective vertical stress is 1 kPa. As reported by Hong et al. (2010), the starting point ($\sigma yr´$, e_{yr}) varies with the initial water content. The higher the initial water content, the greater is the void ratio at the remoulded yield stress and the lower is the remoulded yield stress. It can be observed in Figure 2.6 that a unique value of e_1 can be expected for a given position of (σyr´, e_{yr}). That is, for a given clay with a certain liquid limit, the difference in initial water content will result in different positions of ($\sigma yr´$, eyr), and in turn different values of e_{yr}. In the same manner, reconstituted clays with different liquid limits but at the same initial water content still have different starting points of the compression lines with the effective vertical stress higher than the remoulded yield stress, consequently resulting in different values of e_1.

The reconstituted clays having different liquid limits or different initial water contents certainly have different positions ($\sigma_{yr}´$, e_{yr}) and thus different values of e_1. Hence, the void ratio at 1 kPa (e_1)

can reflect the combined effects of e_{yr}, $\sigma_{yr}´$ and e_1 on the compression behaviour of reconstituted clays. In this study, the index e_1 is used for comparison of the compressibility of various reconstituted clays at different initial water contents.

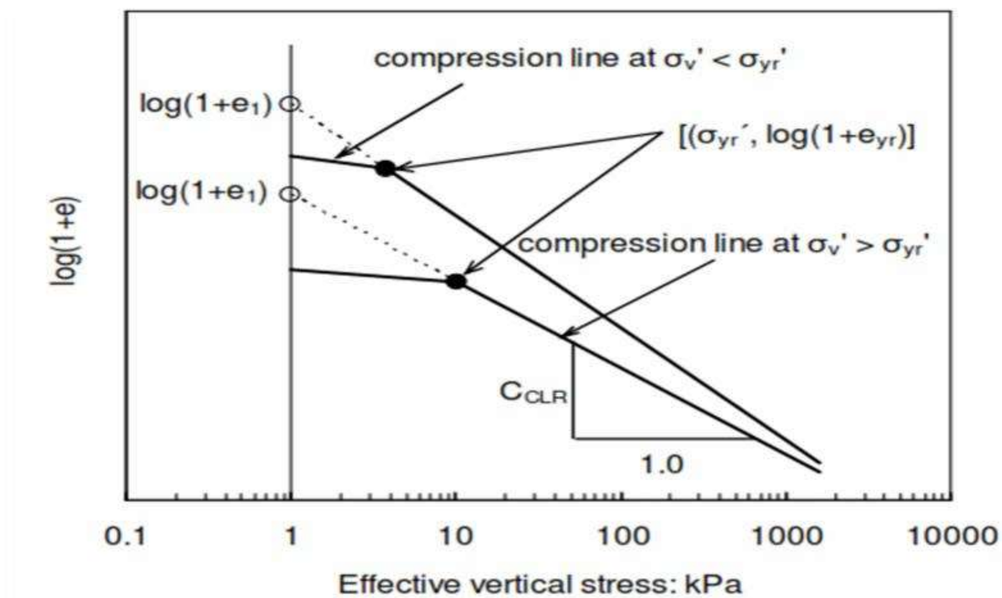

Figure 2.6: Effect of starting point on compressibility for a given clay at different initial point based on Hong et al (2010)

2.6 Effect of void content

On the other hand, Hong et al. (2010) reported that the *e*-*log* σ_v' compression curves of various reconstituted clays with different initial water contents can be normalised well with respect to the void index, I_v when the effective vertical stress is higher than the remoulded yield stress; whereas it is not the case at the effective vertical stress being smaller than the remoulded yield stress, in which the compression curves lie on the left of the normalised line, and the I_v - log σ_v' curve with a higher initial water content lies above that with a lower initial water content, as shown in Figure 2.7.

The normalised compression curve for reconstituted clays at different initial water contents and with different liquid limits is designated as the extended intrinsic compression line (EICL) by Hong et al. (2010). The EICL extends the intrinsic compression line (ICL) of Burland (1990) to wider ranges for both of the initial water content and the effective vertical stress: 0.7 - 2.0 times the liquid limit and as low as 1.5 kPa, respectively. It was demonstrated that when the effective vertical stress is higher than 25 kPa, the EICL coincides with the ICL suggested by Burland (1990). Within the range of effective stress from 1.5 kPa to about 10 kPa, the EICL lies a little above the ICL (Hong et al., 2010). Note that Burland (1990) suggested that ICL should be used only for effective vertical stress ranging from 10 kPa to 4000 kPa. All the 90 reconstituted clays have a normalized behaviour of compression curves with respect to the void index only if the effective vertical stress is higher than the remoulded yield stress by Hong et al (2010).

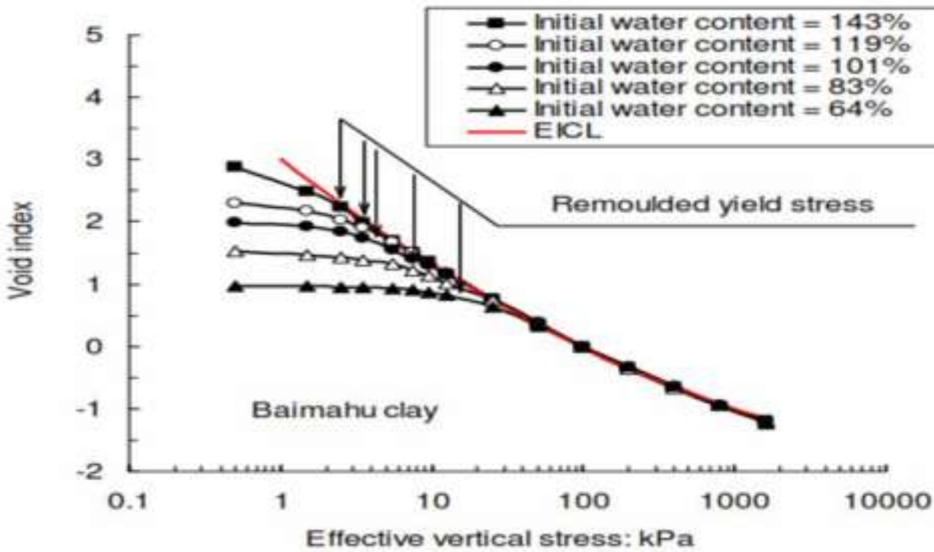

Figure 2.7: Effect of void ratio to consolidation test based on Hong et al (2010)

3.0 Methodology

A total of 5 reconstituted marine clay samples were used to carry out tests such as basic soil test and oedometer tests. The samples were taken from a Semantan field area in Terengganu region. The basic tests involved moisture content, Atterberg's limit, specific gravity, particle size distribution using hydrometer test and Scanning Electron Microscope (SEM). In total, 28 tests were carried out in this project. The summary of the tests was shown in table 3.1. The flow chart for overall laboratory testing is shown in Figure 3.1.

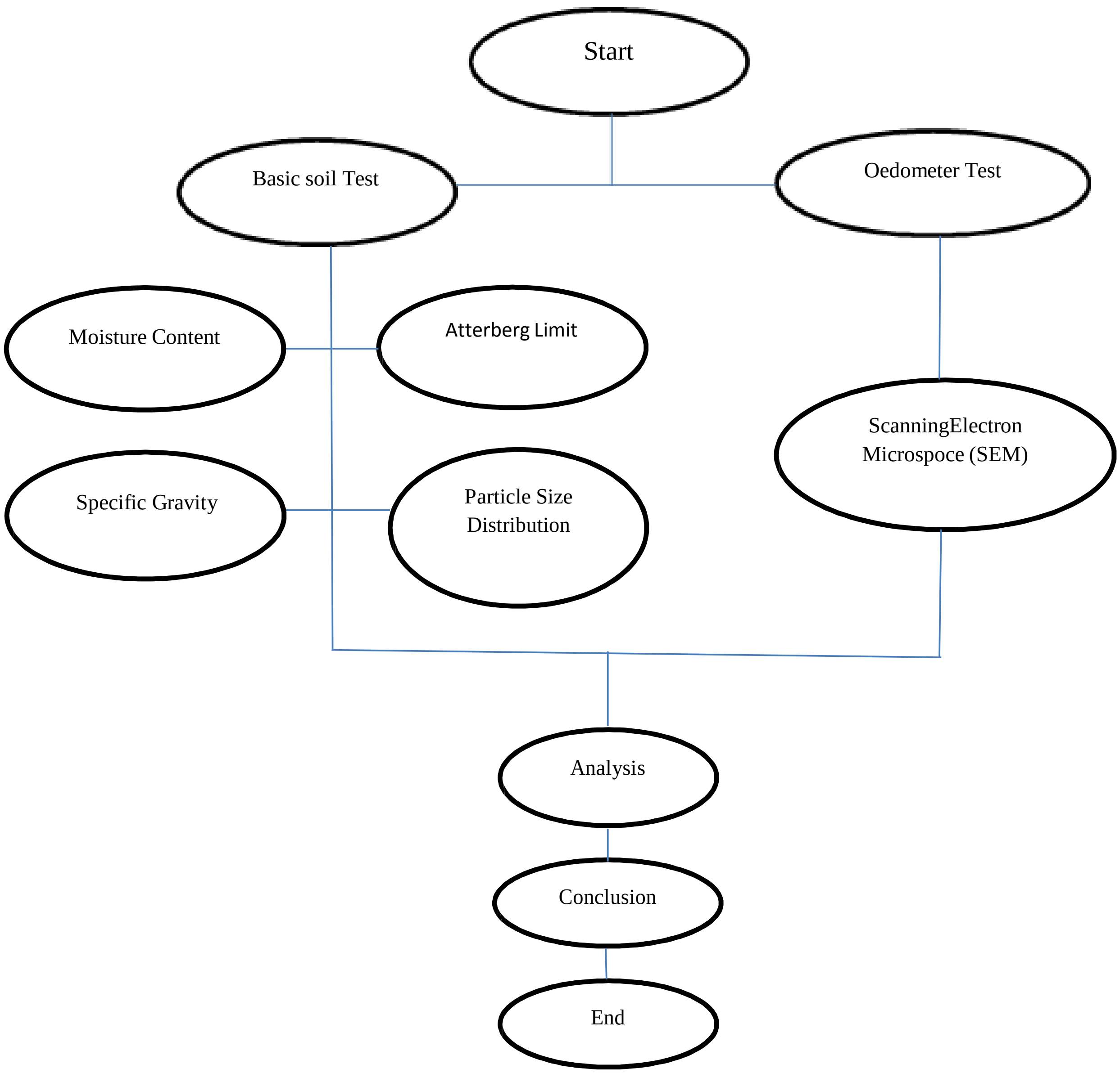

Figure 3.1: Laboratory testing

3.1 Basic Soil Tests

The total of 5 tests were performed in order to obtain the basic soil properties such as moisture content, particle size, plasticity value and others. All the tests performed in this project were accordance to British Standard which is BS 1337 – 2: 1990.

3.1.1 Particle Size Distribution (PSD) and Hydrometer Test

The test was performed on the clay samples to determine the distribution curve. The samples were broken down to smaller size (powder size) after being dried for 24 hours at 1000C in the oven. The samples then were sieved using the shaker. Since there is a part of clay in the sample, hydrometer test is required to complete the test by passing the sieve shaker 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 1000C 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 two different tests which were Plastic Limit (PL) and Liquid Limit (LL). They were determined by the standard Cone Penetrometer method and Plastic Limit method. 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 (PI).

3.1.4 **Specific Gravity (SG)**

Specific gravity (SG) tests were conducted using the small pyknometer jar method. This method is more accurate for fine grained soil compare to the large pyknometer 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.

Figure 3.2: Small pyknometer jar will be used in specific gravity test

Figure 3.3: one litre of cylinder will be used in hydromter test

Table 3.1: List of samples for laboratory testing

Number	Sample	Symbol	Depth (m)	Laboratory Testing						
				Basic Soil Test						
				PSD	PL	LL	SG	MC	SEM	OEDOMETER
1	SEMANTAN	SE1	10.6	/	/	/	/	/	/	/
2	SEMANTAN	SE2	16.7	/	/	/	/	/	/	/
3	SEMANTAN	SE3	17.8	/	/	/	/	/	×	/
4	SEMANTAN	SE4	23.8	/	/	/	/	/	×	/
5	SEMANTAN	SE5	24.6	/	/	/	/	/	/	/

3.2 **Oedometer Test**

Five oedometer test were performed to determine the magnitude and rate of consolidation of a saturated or near-saturated specimen of soil in form of a disc confined laterally when subjected to different vertical axial pressure. The objective of the test is to determine compression index, C_c, recompression index, C_s and the preconsolidation pressure, P_c of the soil. The end result of the experimental will give the compression curve (pressure – void ratio relationship). The test will be carried out using fixed ring method. The trimmed specimen was placed inside the stainless steel ring with the diameter of 50 mm and 20 mm height. The water was added to the samples with 1.25 w_L for reconstituted sample with oven drying method. The lever arm ratio of eodometer machine is 10:1indictating the load transfer from weight should be multiplied with 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 27°C.

3.2.1 **Sample Preparation**

For this research project, undisturbed samples were obtained directly from the soil contractor. The preparation of the sample started by taking the sample out from the case by cutting the plastic casing and separating the sample from wax layer. Then, the sample was cut 1/3 from the original height before drying in the oven. The dried samples were broken into smaller pieces and sieved with passing through 425µm. After sieving, the dry sample were reconstituted at 1.25 w_{LL} as mention by Burland (1990). The samples was mixed and mixed thoroughly 1.25 times liquid limit. After mixing slurry, the samples were left in the vacuum for 4 hours to remove the air bubbles from the soils. The sample, then was left for 24 hours for it to settle as shown in figure 3.4. A 50 mm diameter of ring was used for the oedometer test and the slurry sample was placed inside the ring as shown in figure 3.5. The remaining intact sample left in the plastic casing was used for other tests such as Atterbergs limit and moisture content.

Figure 3.4: Sample after curing process

Figure 3.5: The sample was placed inside the ring.

3.2.2 **Saturation Stages**

In consolidation test, the specimens were subjected to a saturation process first as shown in Figure 3.6. In this stage, the specimens were left in the presence of water for 24 hours in the metal casing. This is for 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 was 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 3.6. These applied pressures were added carefully and just enough to prevent the soil from further expansion. The soils were left for 24 hours for saturation and the reading was checked again until it achieved a constant value before the consolidation test started.

3.2.3 **Loading and unloading Stages**

A loading sequence is adopted to give a range of compression stresses suitable for the soil type and also for the effective pressure which will occur in situ due to the overburden and the proposed construction. The initial pressure should be large enough to ensure that the sample in the consolidation cell does not swell. The test involved two stages which is loading and unloading. A loading and unloading sequence of stages selected from the following range of pressures is tabulated in table 3.2. The loading will start at 1, 5, 10, 25, 50, 100, 200, 400, 800, 1600, 3200, and 6400 kPa. The unloaded will be at 200,300 and 400 kPa. 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.

Table 3.2: Loading and unloading stages of oedometer

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		

Figure 3.6: Sample were left for saturation stage for 24 hours

Figure 3.7: Small weight were added during saturation process

3.3 Scanning Electron Microscope

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. Four samples were powdered before it will be scanned.

4.0 **Result and Discussion**

4.1 **Basic Properties**

The result of basic soil test for was shown in table 4. These values were used in correlation with the depth of the samples taken. Since the sample were obtained in Semantan Field, the variation of each sample are the depth location. thus the trend in each characteristic can be observed through the correlation. From this result, the are variation of percentage of moisture content and Atterbergs Limit.

Table 4: Basic properties of the samples

Sample	Depth (m)	Symbol	Moisture Content (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index
Semantan	10.6	SER1	56.00	49.90	75.5	25.6
	16.7	SER2	54.78	52.54	73.27	20.83
	17.8	SER3	45.98	36.20	66.11	29.91
	23.8	SER4	47.53	37.77	64.78	27.00
	24.6	SER5	44.78	39.14	64.34	25.20

4.1.1 **Result for Moisture Content**

The values of moisture content were plotted against the depth of the samples based on Semantan location as shown in Figure 4.1. From the graph, the values for both reconstituted and intact samples shows the decreasing in the moisture content as it go deeper. The samples were used in natural state based on its originality. Thus, it is important to preserve the moisture content in the samples. The highest value obtain from 10.6m depth with the value of 56%. The decline of moisture content in the samples may be due to percentage of clay which in the sand.

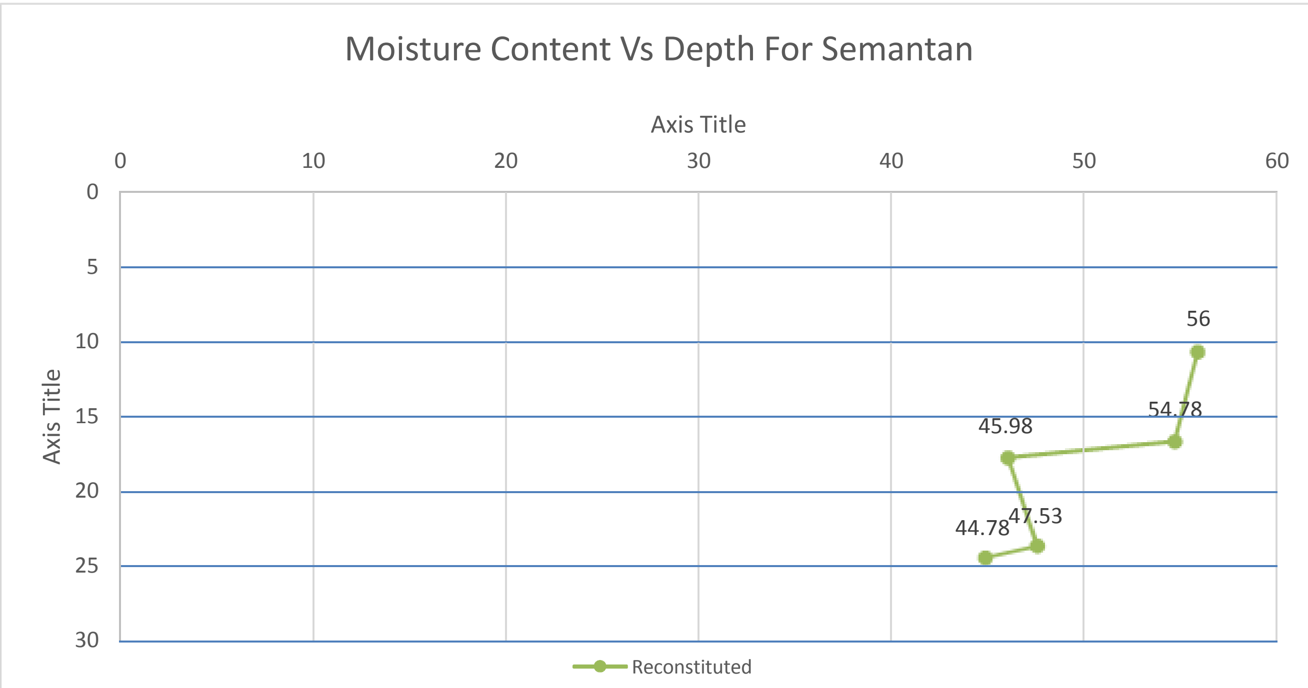

Figure 4.1: Moisture Content Vs Depth

4.1.2 **Result for Atterberg Limit**

The graphs of Plastic Limit, Liquid Limit and Plasticity Index againts depth were shwon respectively in Figure 4.2, 4.3 and 4.4. The Atterberg limit can be used as an indication of the soil transformation upon receiving certain amount of water. Based on the graph, the Plastic Limit range from 36% - 53% for reconstituted. The Liquid Limit values for reconstituted ranged from 64% - 76%. The Plasticity Index values shown a range from 20% - 29%. Based on Burmister (1949) classified the plasticity index ranged between 20% - 40% are high plasticity. Thus, all the samples in Semantan Field are high plasticity.

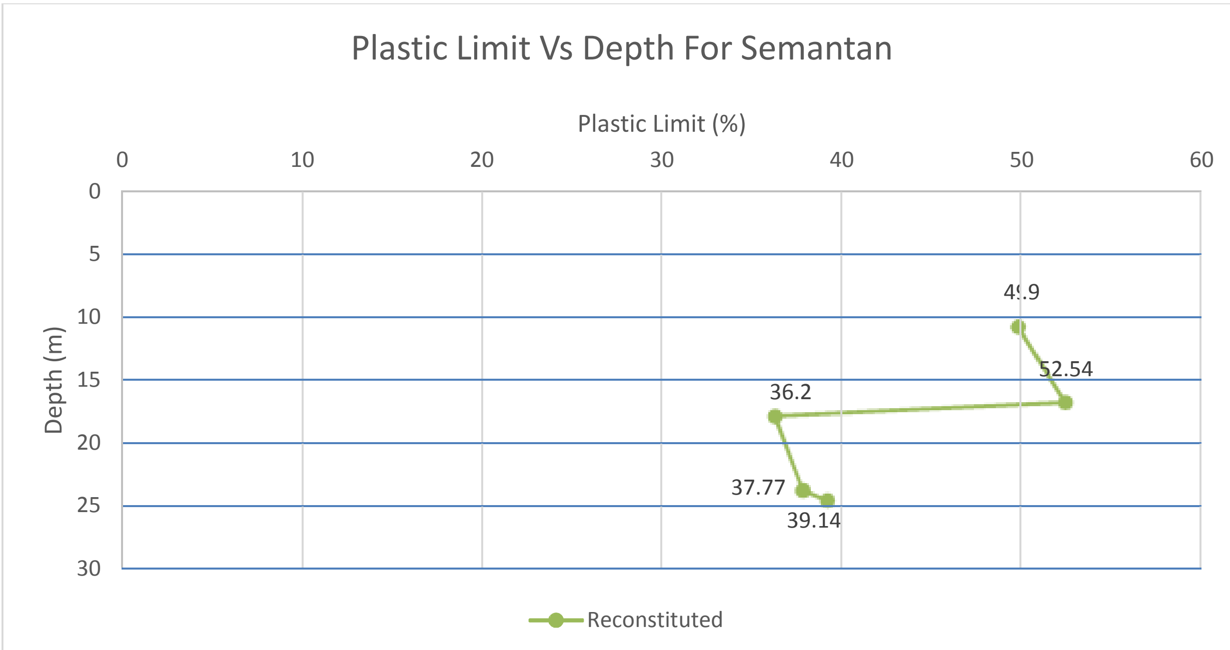

Figure 4.2: Plastic Limit Vs Depth

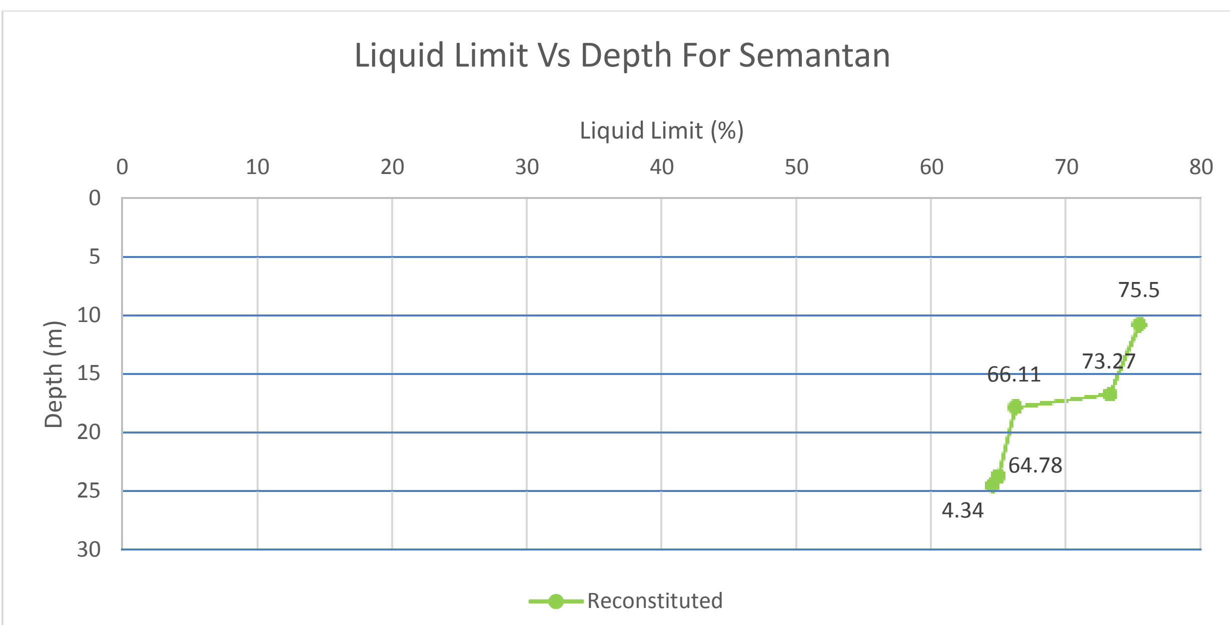

Figure 4.3: Liquid Limit Vs Depth

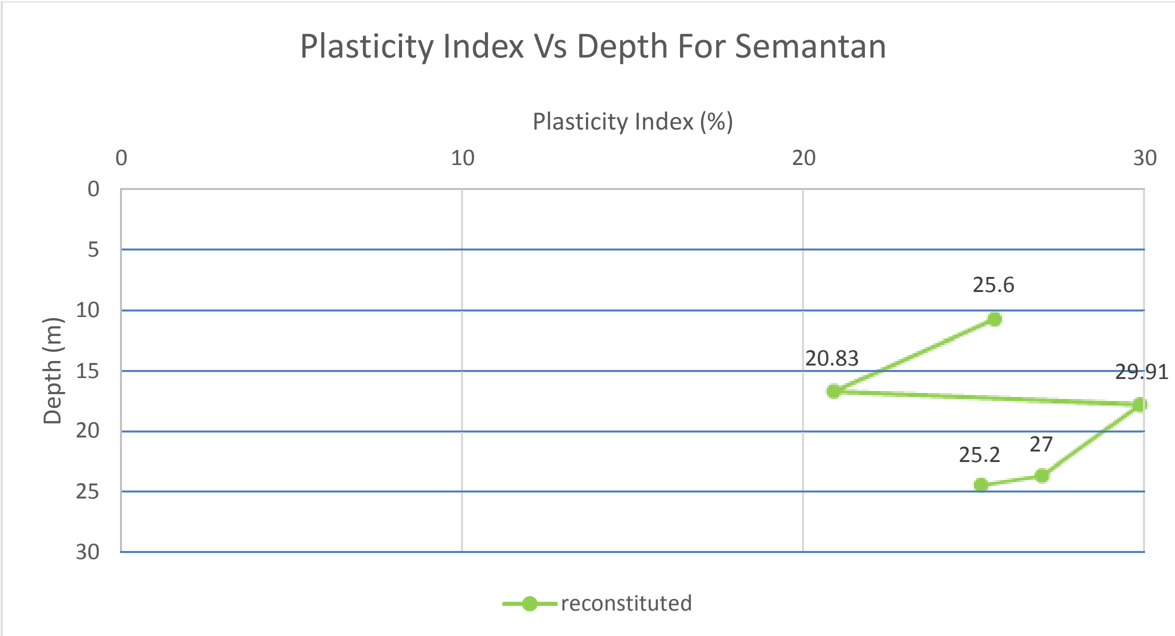

Figure 4.4: Plasticity Index Vs Depth

4.1.3 **Result for Specific Gravity**

The Values of Specific Gravity ranges from 2.2 – 2 .5 as tabulated in Table 4.1.

Table 4.2: The specific gravity of soil

Sample	Depth (m)	Symbol	Specific Gravity	Depth (m)	Symbol	Specific Gravity
Semantan	10.6	SER1	2.23	2.2	S1	2.42
	16.7	SER2	2.48	6.6	S2	2.66
	17.8	SER3	2.65	8.6	S3	2.61
	23.8	SER4	2.55	18.8	S4	2.62
	24.6	SER5	2.48	27.6	S5	2.55

4.2 **Result for Hydrometer test**

The Particle Size Distribution curves were plotted in Figure 4.5. These curves was used to classify the soil percentage finer present. The summary of soil constituent was tabulated in Table 4.3. From the table, the sample at the depth of 10.6m shows the highest percentage of clay. The percentage of clay founded shows decreasing from depth as it goes deeper. The constituent of clay and sand plays a major role in determine whether the soil have a structure or not. Based on U.S. Department of Agriculture (USDA) textural classification classified range of soil from 10.6m – 23.8m are clay while soil in 24.6m is silty clay.

Table 4.3: The summary of soil constituent

Sample	Symbol	Depth (m)	Clay (%)	Silt (%)	Sand (%)
Semantan	SER1	10.6	55	35	10
	SER2	16.7	52	35	13
	SER3	17.8	51	42	7
	SER4	23.8	50	45	5
	SER5	24.6	40	52	8

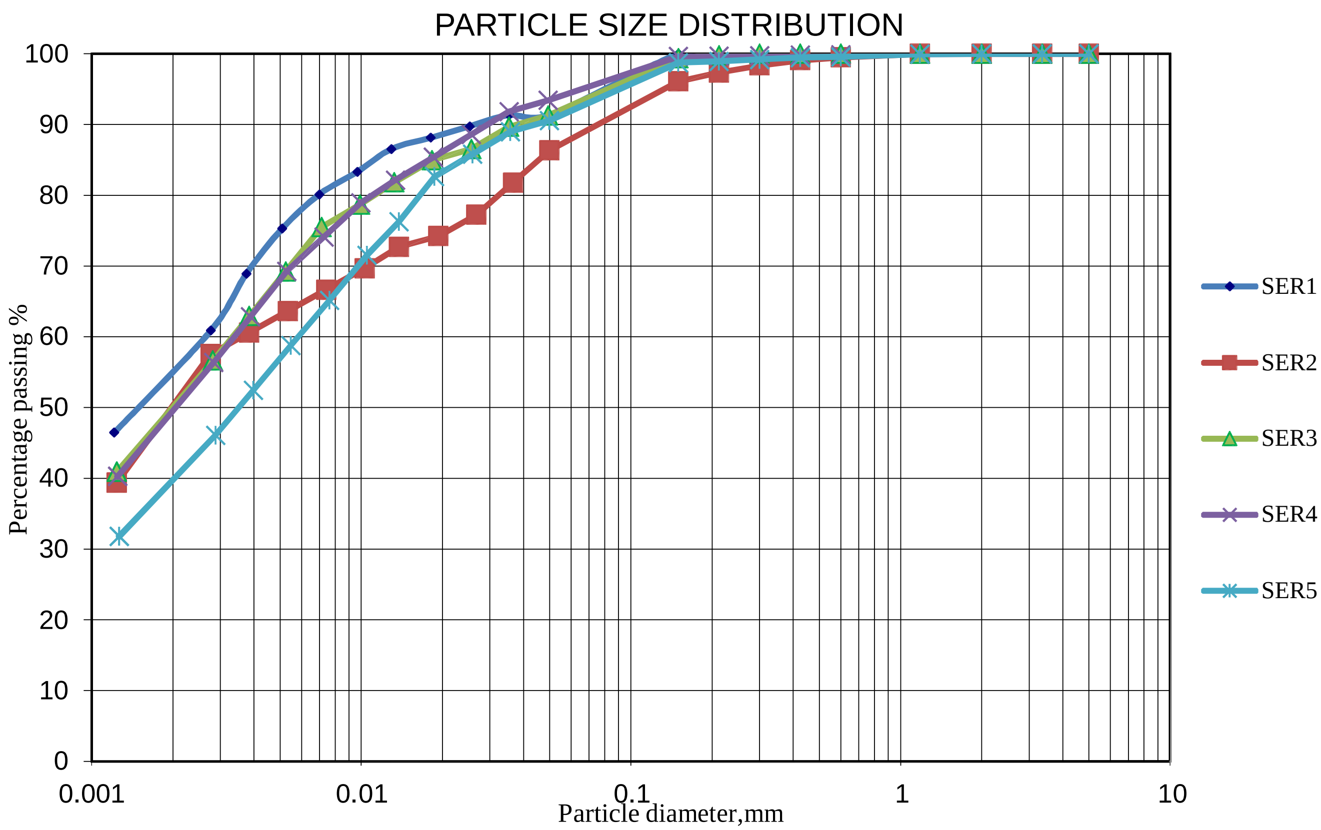

Figure 4.5: Particle Size Distribution

4.3 Scanning Electron Microscope (SEM)

The studies of structure in this project were presented by the result of SEM. The intact sample was powdered and sieve passing through 425µm. Microstructural studies of Semantan clay at 10.6m, 16.7m and 24.6m shows the presence of flocculated structure with the particles are closely packed together shown in Figure 4.6, 4.7 and 5.8. 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.

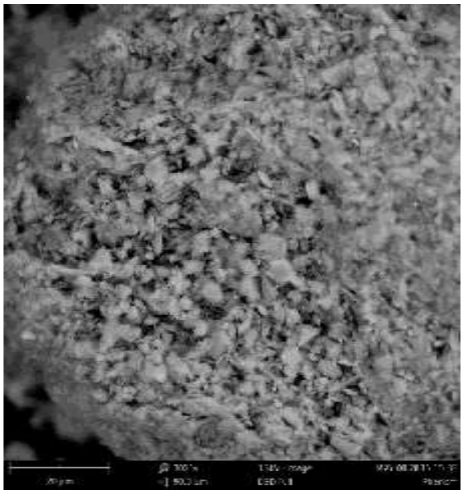

Figure 4.6: Scanning Electron Microscope of Semantan 10.6m

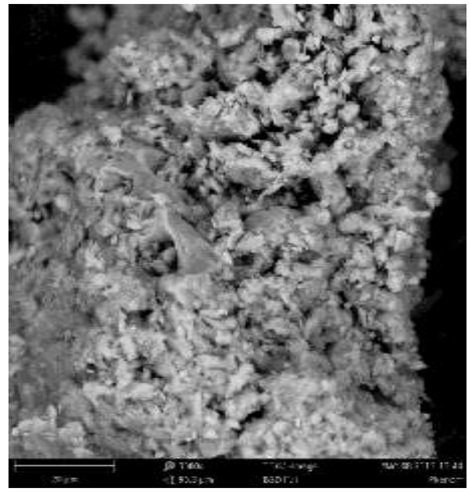

Figure 4.7: Scanning Electron Microscope of Semantan 16.7m

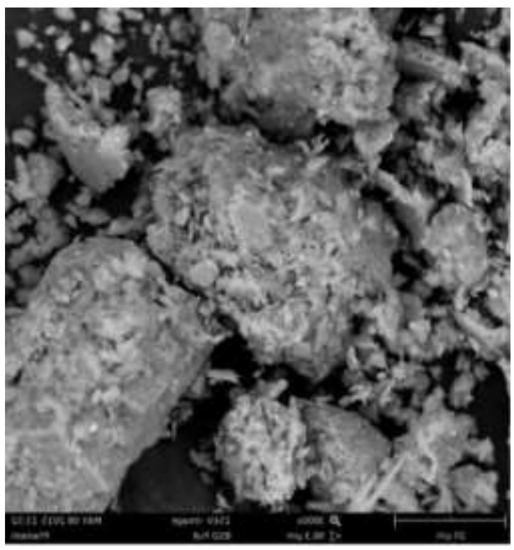

Figure 4.8: Scanning Electron Microscope of Semantan 24.6m.

4.4 **Compression Curves**

Compression curves were plotted for the Semantan clays was 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 and reconstituted with 1.25 w_L, 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. 3 specimens with 24 hours consolidation were tested up to 1600 kPa while the other 2 samples with consolidation for one week were loaded up to 3200 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.

Based on the Figure 4.9, the reconstituted samples, it does not have structure in the soil and thus, the pre-consolidation pressure is less for it to reach breakage point. For sample 10.6m, 16.7m and 17.8m the graph shows no pre-consolidation pressure. The constituent of clay as shown in table 4.1 makes the structure in the soil. For reconstituted samples, the percentage of clay are higher and it shows a very high plasticity. This proves that, the structure will have less breakage point due to clay constituent in the soil. The results of preconsolidation pressure are listed in Table 4.3.

Table 4.3: Pre-consolidation pressures of all the samples

Sample	Depth (m)	Symbol	Initial void ratio	Preconsolidation pressure, *Pc* (kPa)	Maximum pressure tested (kPa)
Semantan	10.6	SER1	1.74	-	1600
	16.7	SER2	1.90	-	1600
	17.8	SER3	2.01	-	1600
	23.8	SER4	1.84	3	6400
	24.6	SER5	1.91	3	6400

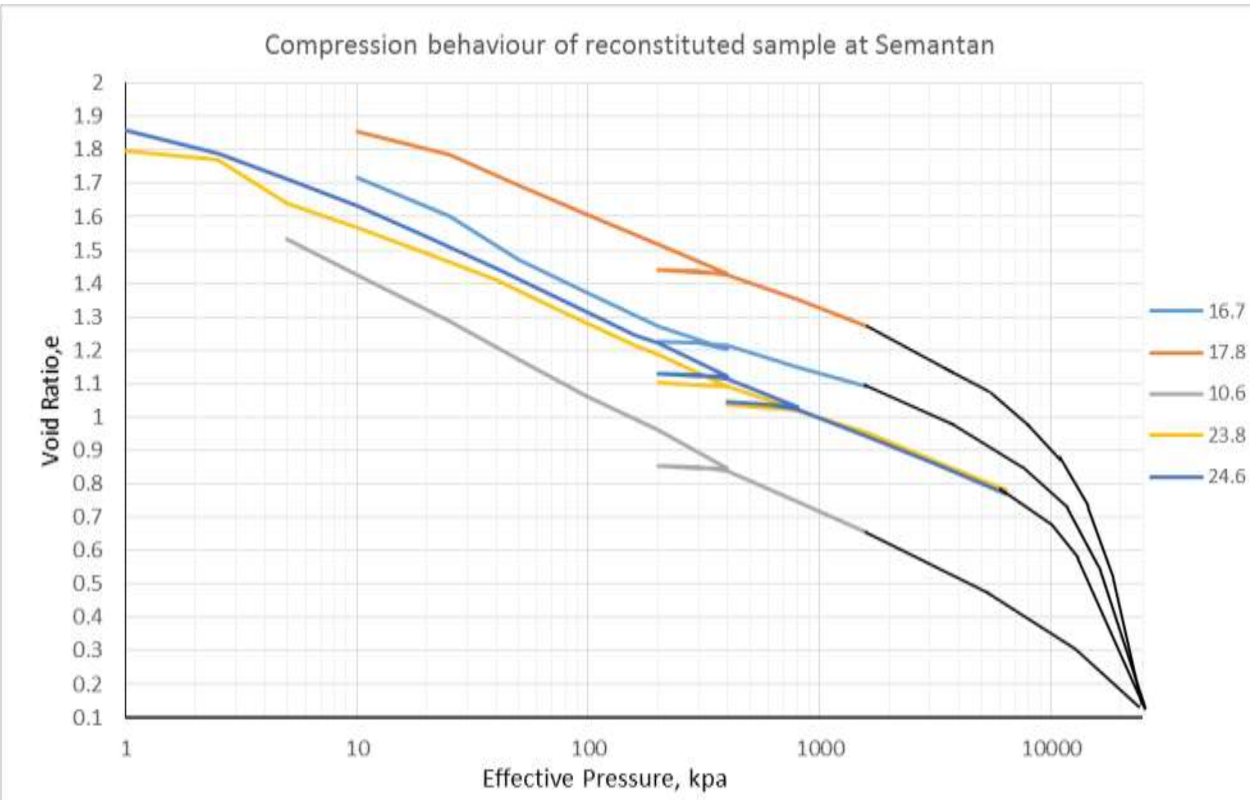

Figure 4.9 : Compression curves of the samples

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 4.4. 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. The correlations are made between the compression index and Liquid Limit (LL) for intact and reconstituted samples as shown in Figure 4.10.

The relationship between the Cc and LL produced a straight line for Semantan area. The equations are described as below:-

$Cc = 0.003LL + 0.3478$ Equation 4.1

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 specimens in Semantan.

The linear relationship was given in a form equation described as below: -

$Cc = 0.166eo + 0.0591$ Equation 4.2

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 4.4. The value of Cc/Cr for reconstituted was shown in figure 4.12. Semantan 17.8 has a larger value of Cc/Cs which is 11.5.

Table 4.4: Compression and swelling Index

Sample	Symbol	Depth (m)	Compression Index, C_c	Swelling Index, Cs	
				400 - 200 kpa	800 - 600 kpa
Semantan	SER1	10.6	0.341	0.056	-
	SER2	16.7	0.409	0.104	-
	SER3	17.8	0.391	0.034	-
	SER4	23.8	0.374	0.045	0.057
	SER5	24.6	0.341	0.057	0.057

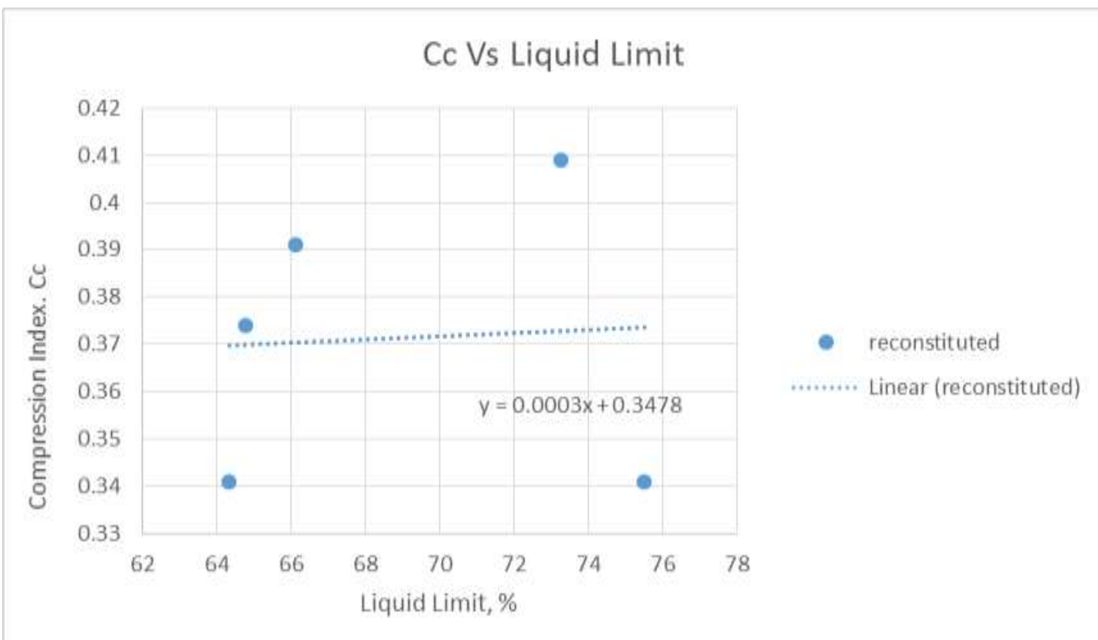

Figure 4.10: Relationship between Compression Index and Liquid Limit

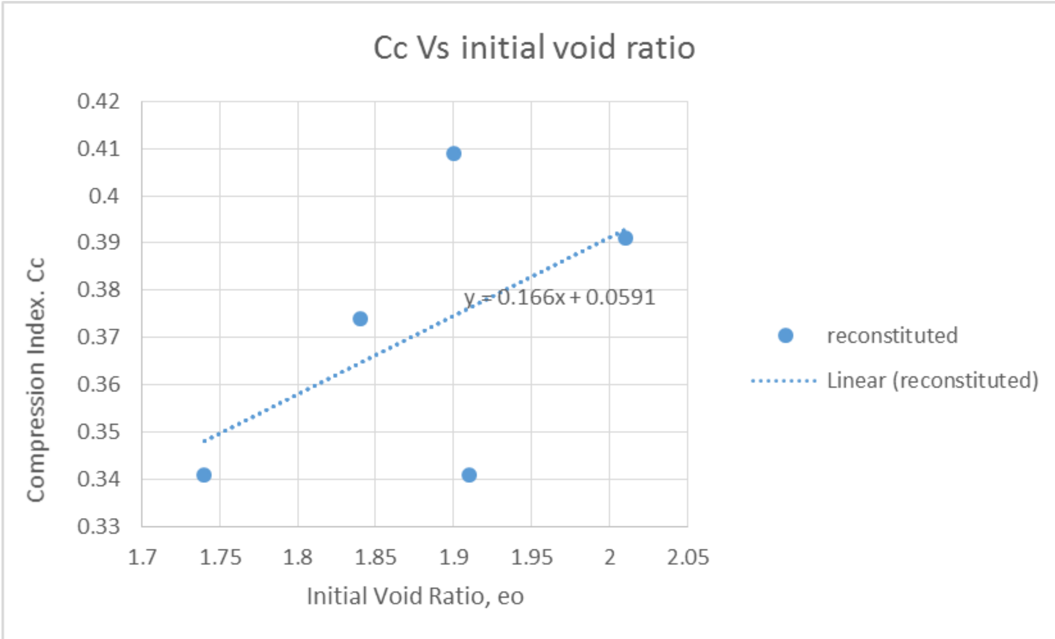

Figure 4.11: Relationship between Compression Index and Initial Void Ratio

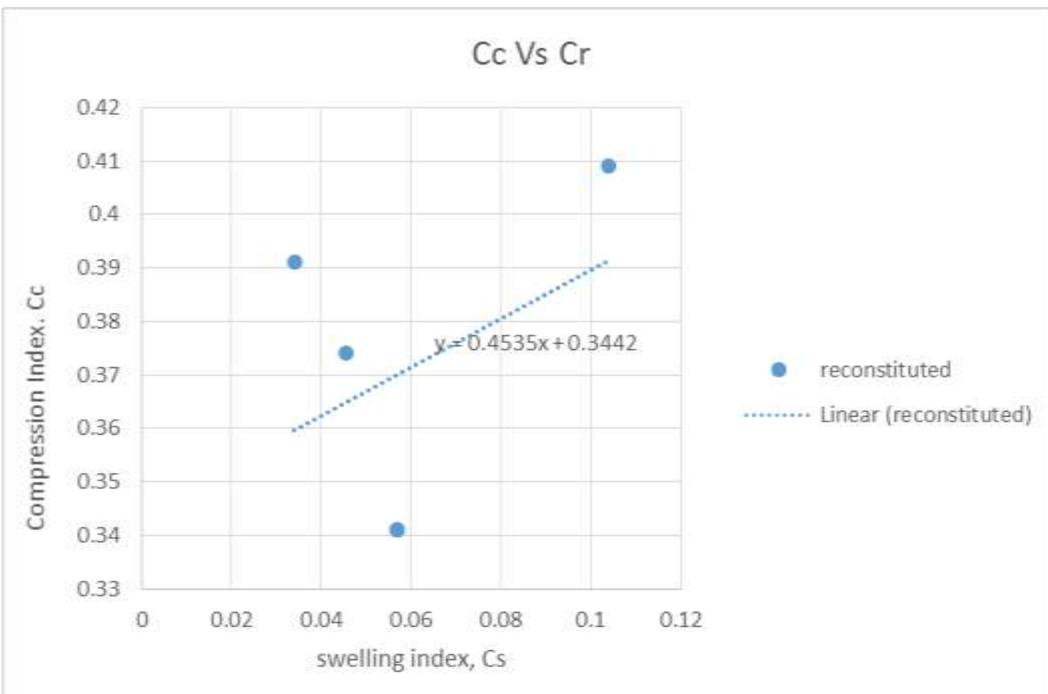

Figure 4.12: Relationship between Compression Index and Swelling Index

5.0 **Conclusion**

The oedometer test have been performed on reconstituted marine clays in order to investigate the compression behaviour of different depth in Semantan field. The influence of fines content and type can also be determined from the consolidation test. The increased in clay content of Semantan samples from 40 – 60 % led to the increment of Cc from 0.341 – 0.409 for reconstituted sample.

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 high initial void content thus shows a compressible behaviour towards the graph plotted. The highest value of moisture content recorded from this project comes from 10.6m which is 56.0 %, thus the soil is having initial void ratio of 1.74. The steeper the graph indicated the soil is more compressible within small range of effective pressure. Tests result for all the sample with different initial water contents suggest that the compression behavior of reconstituted clays is influenced by both the liquid limit and initial water content

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