1- DIMENSIONAL CONSOLIDATION BEHAVIOUR OF ORGANIC SOIL

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ABSTRAK

1-DIMENSIONAL CONSOLIDATION BEHAVIOUR OF ORGANIC SOIL

Kajian ini dijalankan untuk mengkaji sifat-sifat pengukuhan bagi tanah organik. Pengukuhan satu dimensi adalah proses perubahan ketebalan tanah apabila tanah tersebut dikenakan kepada peningkatan tekanan berkesan, di mana tiada perubahan lateral berlaku. Tanah organik adalah tanah bermasalah yang terkenal dengan kebolehmampatannya yang tinggi. Dengan itu, bangunan jarang dibina di atas tanah organik. Untuk kajian ini, tiga sample tanah dari lokasi yang berlainan di Kota Kinabalu telah diambil dan dikajikan ciri-ciri asas dan sifat-sifat pengukuhannya. Graf e-log p menunjukkan ketiga-tiga tanah organik mengalami proses pengukuhan normal. Kajian yang dijalankan menunjukkan pekali kebolehmampatan (terkukuh normal) bertambah apabila kandungan organik dalam tanah bertambah. Ini bermaksud kandungan organik dalam tanah menyumbang kepada kebolehmampatan tanah. Pekali kebolehmampatan isipadu adalah berkadar langsung dengan kandungan organik, tetapi berkadar songsang kepada penambahan tekanan. Nisbah lompang juga berkadar terus dengan kandungan organik dalam tanah, di mana peningkatan kandungan organik akan menyebabkan peningkatan dalam nisbah lompang. Walau bagaimanapun, penambahan beban akan mengurangkan nisbah lompang dalam tanah. Dua kaedah telah digunakan untuk menentukan pekali pengukuhan, iaitu kaedah Casagrande dan kaedah Taylor. Pekali pengukuhan menurun apabila tekanan dan kandungan organik bertambah. Nilai t_{50} and t_{90} bagi tanah organik bertambah apabila tekanan dan kandungan organik bertambah. Kajian juga menunjukan penambahan kandungan organik menyebabkan peningkatan dalam kandungan lembapan dan pengurangan dalam gravity spesifik tanah. Daripada ketiga-tiga sample, tanah organik yang diambil dekat kawasan di hadapan Universiti Malaysia Sabah menpunyai pekali kebolehmampatan (terkukuh normal) paling tinggi.

ABSTRACT

1-DIMENSIONAL CONSOLIDATION BEHAVIOUR OF ORGANIC SOIL

This paper presents a review of the consolidation behaviour of organic soil. One dimensional consolidation is the settlement due to changes of thickness occurs when soil sample is subjected to an increase in effective stress, where no changes in lateral dimension take place. Organic soil is considered as problematic soil for its high compressibility, where structures are seldom built on organic soil. For this paper, there are three organic soils collected from different locations in Kota Kinabalu were tested for their basic properties and the consolidation behaviours. The e-log p curves showed that the soil samples are normally consolidated organic soils. From the tests, the compression index is increasing with the increase in organic content. This means the organic content in soil contributes to the compressibility of soil. The coefficient of volume compressibility is directly proportional to the organic content in soil. But it will decrease when load is increasing. The influence of organic content in soil to void ratio is relative, where increase in organic content will cause increase in void ratio. However, the increase of load will reduce the void ratio in soil. There are two method used in determining the coefficient of consolidation, which are Casagrande method and Taylor method. The coefficient of consolidation will decline when the pressure or organic content is rising. The t_{50} *and* t_{90} *of organic soil samples are increase proportionally to the pressure and organic content in soil. The tests also showed that the increase in organic content will cause increase in water content, but decrease in specific gravity. From the three samples, the organic soil collected from the site located opposite University Malaysia Sabah has the highest compression index, which means it is most compressible.*

CONTENTS

LIST OF SYMBOLS AND ABBREVIATIONS

C_c	Compression index
C_v	Coefficient of consolidation
e	Void ratio
Δe	Changes of void ratio
G_s	Specific gravity of soil particle
H	Thickness of soil specimen
Δh	Changes of thickness
LL	Liquid limit
m	Mass
M_v	Coefficient of volume compressibility
p	Net pressure
Δp	changes of pressure
P_d	Dry density
PI	Plasticity index
PL	Plastic limit
S	Degree of saturation
t	Time
w	Moisture content

CHAPTER 1

INTRODUCTION

1.1 Overview

Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to drainage of some of the pore water. The process continuing until the excess pore water pressure set up by an increase in total stress has completely dissipated. The process of swelling, the reverse of consolidation, is the gradual increase in volume of a soil under negative excess pore water pressure.

Consolidation settlement is the vertical displacement of the surface corresponding to the volume change at any stage of the consolidation process. Consolidation settlement will result if a structure is built over a layer of saturated soil or if the water table is lowered permanently in a stratum overlying a soil layer. If it is assumed that the soil remained saturated throughout the consolidation process, it follows that the decrease in volume that takes place is equal to the volume of water squeezed out, and that is represented by the change in void ratio. A process will be considered as one-dimensional when there is no change in lateral dimension take place, but only change in thickness occurs when a soil sample is subjected to an increase in effective stress.

The compressibility characteristic of a soil relating to the amount and rate of settlement are usually determined from the consolidation test, using an apparatus called oedometer.

It may be assumed that the immediate settlement due to the elastic compression of the soil takes place instantaneously following an increase in stress. In the consolidation process, however, sometime must elapse while water seeps from the soil and the excess pore water pressure is dissipated. Therefore the rate at which consolidation occurs depends mainly on the permeability of the soil. To a lesser extent it also depends on deformational creep in the soil particle skeleton and on the compressibility of soil constituents such as air, water vapor, organic matter and the solid matter itself.

Therefore, it is convenient to considered primary consolidation as being that change in volume due to the seepage of pore water from a saturated soil, and secondary consolidation as that resulting from creep, slippage between particles, and others.

1.2 Problem background

Recently urban development has been brisk around large cities in Malaysia. Because of limited land area, some structures are built on highly organic soft ground, which used to be a marsh. Highly organic soils are well known for their high compressibility and long term settlement. In particular, the deformation of the surrounding ground surface of a partially loaded embankment construction consists of not only vertical settlement but also heave deformation due to lateral movement of foundation while embankment loading. These phenomena bring about many kinds of damages.

Furthermore, there is a continuous demand for minimizing the construction and maintenance costs and for reduction of the construction time, accompanied by a higher reliability. These show that the economical importance of construction is increasing. However, deformation and stability are also important issues.

When a structure is conducted on organic soil, stability and settlement problems often arise. The most troublesome problems are associated with the high compressibility, very low shear strength and large creep of the organic soils.

1.3 Objectives

The objectives of this research are:

a) To understand the consolidation characteristics of the organic soils.

b) To compare the consolidation characteristics of organic soils collected from different locations.

c) To understand how the soil characteristics influence the compressibility of organic soils.

1.4 Research Scope

To obtain the one-dimensional consolidation behaviour of organic soil, oedometer tests were being carried out for organic soil specimens. There were three organic soil specimens collected from different site around Kota Kinabalu for the experiments in this research. The research scope is only covers laboratory experiments of one-dimensional consolidation behavior of organic soil.

The organic soil samples were taken in-situ, where the ring of the oedometer was taken to the site and pressed to the ground to obtain the soil samples. Then immediately the soil sample was brought back to the lab for future testing.

Besides the oedometer test, some basic preliminary test such as specific gravity test, water content test, sieve analysis test, liquid limit and plastic limit test were also being conducted to obtain the basic properties of soil.

CHAPTER 2

LITERATURE REVIEW

2.1 Consolidation and settlement of peat under loading

Kogure (1998) stated that peat is a rather complex material containing fibrous organic material as major constituent. The high compressibility of peat stands out as a most significant engineering property. In applying the conventional method of one-dimensional settlement to the behavior of peat, there is the major deviation from the usual assumption, that is, the compressibility of fibrous organic matter, which forms the fibrous framework of peat. This anomaly is believed to account for the significant differences in consolidation behavior between peat and mineral soils. The consolidation of peat is considered as the total compression resulting from volume change under a vertical load. The settlement will be considered as the vertical deformation of the peat surface. The vertical deformation resulting from shear may be appreciable, particularly at high load intensities, and there should be considered separately when predicting field compression.

In applying the conventional theory to the consolidation behavior of peat, there are two major deviations, namely the compressibility of the solids, and the change in permeability under applied load.

A typical laboratory time settlement curve for peat is showed in Figure 2.1. It will be seen that an initial compression occurs over a very short time interval, followed by a long-term compression that is essentially straight with the logarithm of time. The dissipation of pore water pressure is associated with the rapid or initial stage. The initial stage is analogous to primary consolidation, and the long term stage to secondary compression.

Compression index, Cc in peat depends on the consolidation pressure and is not a constant value or a straight line relation in the normally consolidation region. If the e-log p curve is not a straight line, the maximum slope of e-log p curve is regarded Cc.

Figure 2.2 shows the relationship between the instantaneous compression index Cc ($=\Delta e/\Delta \log p$) and the consolidation pressure p of the peat. It will be seen that the peat value of Cc in peat appears in the range from about 0.5 to 0.7 kgf/cm^2 of the applied consolidation pressure.

2.2 Laboratory test of an Italian peaty soil

Laboratory test was carried out by Colleseli and Cortellazzo (1998) on an Italian peaty soil collected near the Northern Adriatic Coast. The soil is dark brown and according to Radforth's (1969) classification, mostly corresponds to class A, categories 3, 4 and 5. The water contents of peaty materials are generally between 250-400%, with unit weight between 4.1 and 6.0, and percentage of organic contents at 500° higher than 70%.

Large diameter (0.2 m) samples were collected in order to minimize disturbance. Long duration oedometric tests were conducted with load remaining constant for at least 1 month, or even more than 4 months. Loading steps in the ratio of $\Delta p/p = 1$ were applied to incremental tests, while constant loading steps of 50, 100 and 200 kPa were used for the long-duration tests.

The preconsolidation pressure was estimated at about 110 kPa. Compression index Cc fell between 2.7 and 3.5. Permeability was between 10^{-10} and 10^{-11} m/s for stress over 100 kPa. The ratio $\Delta e/\Delta \log k$ varied between 1.4 and 2.0.

They concluded that the long-duration oedometer tests showed that the compression curve in the creep stage is initially linear and then becomes steeper, the same result was found by Dhowian & Edil (1980). The Taylor construction method underestimates the time to the end of primary compression and overestimates the end of the primary void ratio, compared with those determined through pore pressure dissipation.

2.3 An overview of the physical and chemical properties of Japanese peat

Oikawa, Sasaki, Fujii, Komatu and Ohtake (1998) stated that the mean specific gravity of organic constituents, G_o for Japanese peat is 1.5, somewhat larger than that of Dutch peat with G_o = 1.365. They also stated that peat is not always fully saturated, even if it is submerged beneath the ground water table. The measured value of the degree of saturation, S_r is plotted against its natural water content, w_n in Figure 2.3. According to the figure, S_r tends to decrease lineally with increase in w_n. This relationship can be expressed by the equation: $S_r = 1-0.01\ w_n$.

Natural water content, w_n is plotted against ignition loss, L_{ig}, where increase in w_n will result increase in L_{ig}. w_n / L_{ig} has been considered as a index that shows the virginity of peat land.

The natural water content, w_n of Japanese peat is as high as about 1200%, and the void ratio, e_n is as high as about 20. Such high values have been also reported by Canadian peat (Hanrahan: 1954). But this relationship is differ from the peat reported near the central Netherlands (Den Haan: 1997)

Wet density and dry density of Japanese peat decrease hyperbolically with increase in its natural water content, in a good correlation. This relationship is almost the same as the peat near the central Netherlands (Den Haan: 1997).

2.4 Behavior of ground surface deformation due to embankment loading on highly organic soil

Kamao and Yamada (1997) have conducted a series of laboratory model loading tests in order to establish a method to predict the deformation of embankment and surrounding ground surface. They concluded that the deformation characteristics of the ground surface are expressed by the loading speed and thickness of the soft ground. The magnitude of deformation of embankment itself is close to one-dimensional settlement when the lower loading speed is adopted. The larger the loading speed is, the larger the lateral deformation and heave of the ground surface are. Their proposed prediction equations of the ground surface deformation give in good agreement with the field measurement data, as showed in Figure 2.4.

2.5 Measurement of long-term settlement of highly organic soil improved by preloading

Monoi and Fukazawa (1994) have been measuring the long-term settlement of highly organic soil at three construction sites for six to twelve years. These measurements show that long-term settlement can be minimized (e.g. less than 5cm in ten years) by preloading with an over-consolidation ratio of not less than 1.2 to 1.3. The details of the settlement measurement site are showed in Table 2.1.

They concluded that the resettlement of soil starts after about a year to two years after preload with an over-consolidation ratio (*OCR*) of not less than 1.0. The rebound occurs on removal of preload. The expansion strain εR is proportional to *OCR*. The period from unloading to resettlement start, t_s is approximately 0.4 times the preloading period, t_p. Coefficient of resettlement, $\varepsilon_{\alpha,}$ the rate of resettlement after removal of preload, increases with initial natural water content, w_n and decreases with *OCR*.

2.6 Problems in the construction of bridge foundation in peat and very soft organic clay

During the construction of the Kuala Lumpur International Airport, Hassandi (1998) stated that they faced problems in the construction of the foundation for a bridge in soft sensitive marine clay. They used jet grouting to stabilize the excavation in area which had been experienced ground movement. But in very soft soil, the use of jet grouting caused some displacement to the piles and affected the stability of the excavation when the soil parameters were not properly ascertained.

The organic content of the clay at the bridge site is between 4 to 21 percent with sensitivity ranges from 3.5 to 29. Typical subsoil profile of the area is showed in Figure 2.5.

Before sheet piles were driven and pilling work commenced, large cracks due to the ground movement developed within the earth fill platform after the site was flooded. These cracks extended to about 15m from the bank on the pier side. After the completion of pilling work, the excavations for piers were carried out. They observed that most piles were displaced beyond acceptance limit towards the direction of river. Some displacements were as large as 2m.

To solve the problem, they decided that the piles which moved excessively will be replaced with the same numbers and pile types. The excavations were backfilled with sand to facilitate the pilling work, and the jet grouting work was done for stabilization.

However, in the process of further excavation, the grouted sand fell into the excavated area breaking two piles. The removal of grouted columns was done in order to avoid the grouted sand falling into the work area again. But these made the situation became even worse. Cracks started appearing, ground depression also occurred. The piles in pier once again were displaced but in the opposite direction.

Pile integrity test was carried out on seventy one piles indicated that only two were sound while the rest were suspected to have cracks. Analysis to determine the factor of safety for the excavation showed if the removal of grouted column was not been carried out, there would not be any failure. The contractor has assumed the shear strength of the clay to be 10kN/m^2 which actually ranges between 1 to 3kN/m^2 also contributed to the failure.

They concluded that the failure to scrutinize the soil investigation report diligently and make generous assumption on the shear strength of the soil when sign of failure, piling and excavation caused considerable disturbance to the soil, and consequently caused first failure in piers. The removal of grouted column may have caused the ground movement for the second failure. Notwithstanding that, the excavation would still stand had the shear strength assumed was correct. The use of jet grouting within the piled area needed special attention in very soft sensitive soil because it may cause excessive displacement and upheaving of the piles.

2.7 Geotechnical parameters of Recife organic soils-peats

Coutinho and Oliveira (1998) have discussed some geotechnical and statistical correlations of the Recife soft / medium soil database, with emphasis on the subgroup organic soil – peats. They found the deposits of organic soil on the Northeastern coast of Brazil either on the surface or in a depth below a sandy layer.

From their research, the water content values of Recife soft soil is situated between 30% and 800%. The initial void ratios are between 0.5 and 10. The ranges of the compression index Cc are between 0.2 and 6, which is very high.

Table 2.1: Settlement measurement sites (Monoi and Fukazawa, 1994)

Site			Sapporo	Chiba	Kanagawa
Filling height			2 - 8m	6.5 - 9m	2 - 8m
Moisture content	w_n	%	863	680	470
Specific gravity	G_s	-	1.444	1.929	1.910
Unconfined compression strength	q_u	kN/m^2	14.7	20.6	23.5
Consolidation yield stress	P_c	kN/m^2	7.9	16.7	14.7
Compression index	C_c	-	12.55	10.80	4.95
Coefficient of consolidation	C_v	cm^2/s	$1.6x10^{-2}$	$3.5x10^{-3}$	$2.2x10^{-3}$
Coefficient of volume comp.	m_v	m^2/MN	16.9	12.7	4.6

Figure 2.1: Typical time-settlement curve of peat. (After Kogure, 1998)

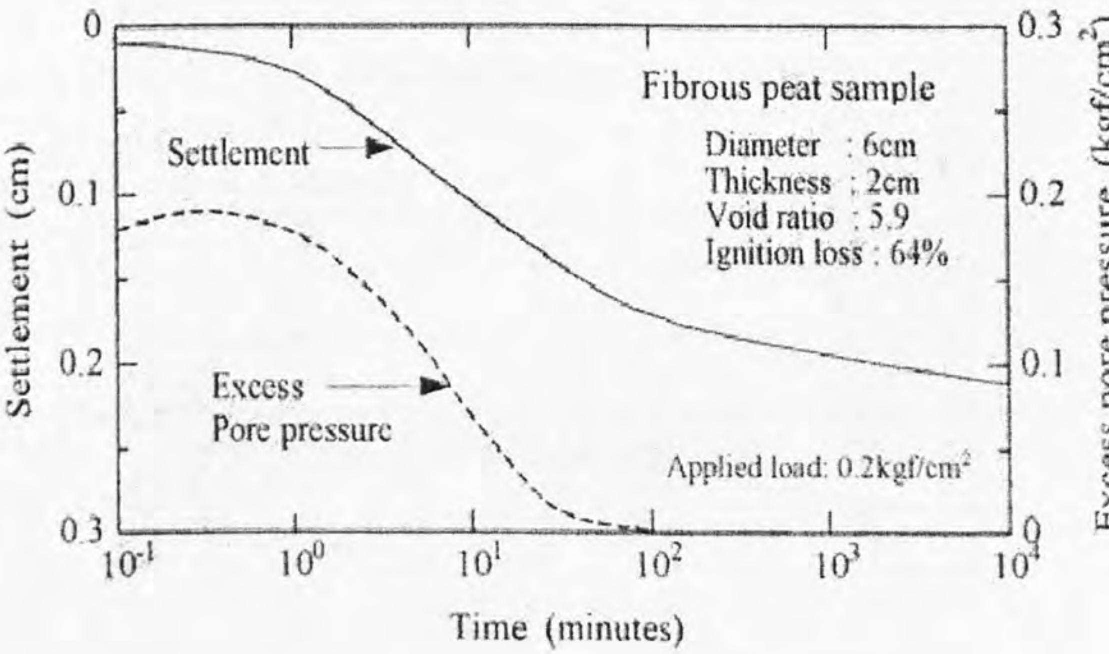


Figure 2.2: Relationship between C_c and consolidation pressure, p.
(After Kogure, 1998)

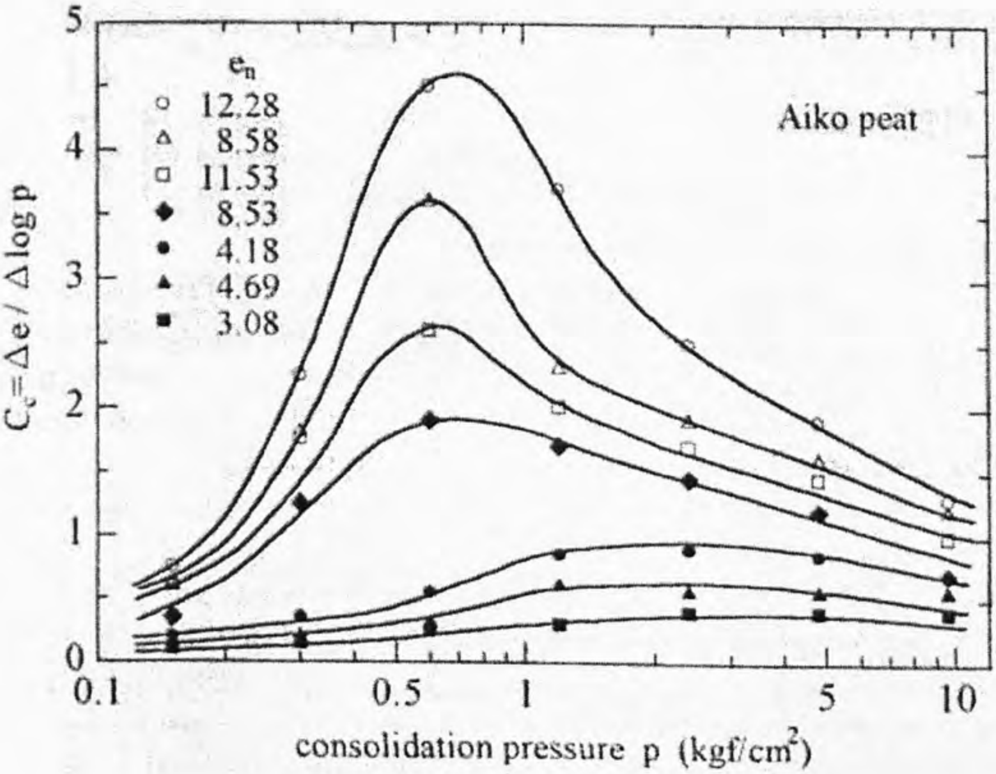


Figure 2.3: Degree of saturation.
(After Oikawa, Sasaki, Fujii. Komatu and Ohtake, 1998)

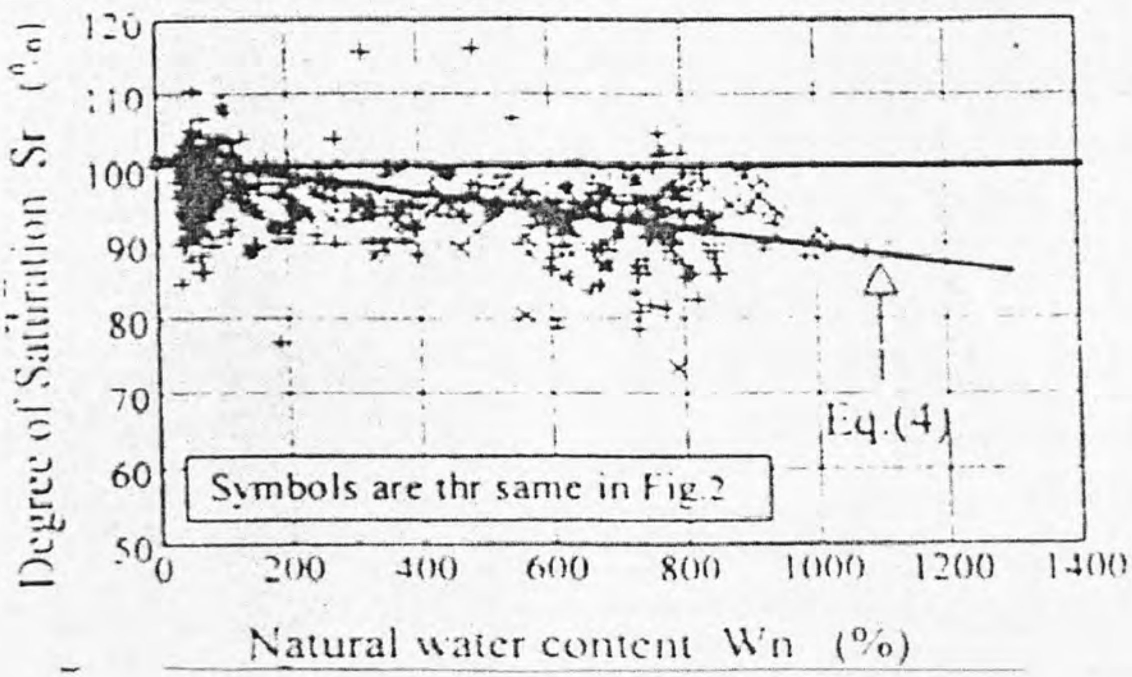


Figure 2.4: Comparison between prediction method and field measurement data.
(After Kamao and Yamada, 1997)

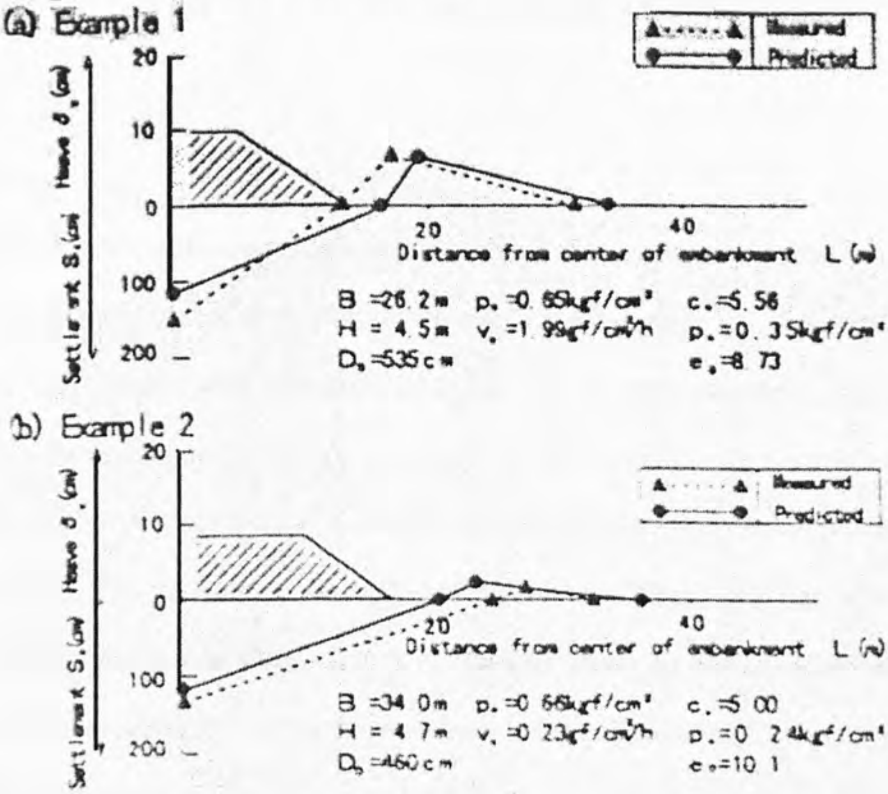
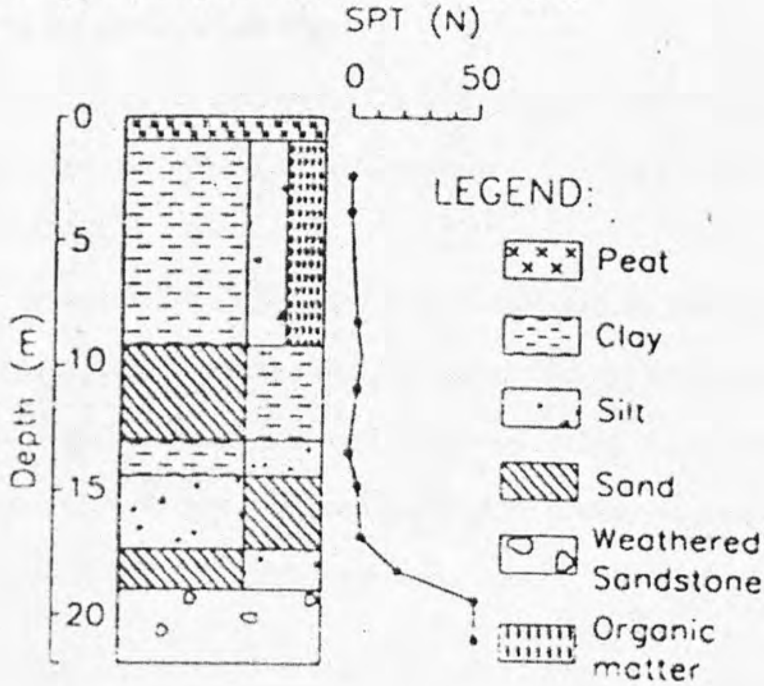


Figure 2.5: Typical soil profile. (After Hassandi, 1998)



CHAPTER 3

METHODOLOGY

3.1 Sampling

There are three (3) sets of sample obtained for testing. The first soil sample was obtained from a tourist spot, named as Bird Sanctuary, which is a swampy area. The second soil sample was obtained from the site located opposite University Malaysia Sabah. The third soil sample was obtained near the lake of School of Science and Technology, University Malaysia Sabah. During the sampling, the upper layer of soil was removed to obtain an undisturbed soil specimen. Next, the oedometer's ring was pressed into the soil ground. A block of soil was then dug out together with the ring and the soil surrounding it. The soil block was then wrapped and brought back to lab for trimming and testing.

3.2 Organic soil determination test

Before sampling, the first step is to determine whether the soil contains any organic material. The existence of organic material in soil can be determined by performing a variety of tests.

The presence of vegetable matter such as sticks, leaves, or grass, imparts to the soil a typical fibrous texture. The color of a moist soil will usually contain dark or drab shades of gray or brown, and may include colors that are almost dark. For comparison, the color of an inorganic soil contains brighter colors, including medium and light gray, olive green, brown, red, yellow and white.

Besides that, the odor of a fresh sample of an organic soil is distinctive, although it gradually diminishes on exposure to air. The original odor can be revived by heating a wet sample. The moisture content of an organic soil normally is very high. The specific gravity of an organic soil is normally lower than that of an inorganic soil due to the presence of vegetable matter and higher percentage of water and air.

This "Loss on ignition" method can also be used to determine the organic content in the soil sample (Hoyos et al, 2002). A container which can withstand high temperature was weighted and added in about 10g of soil sample. The container with soil sample was then dried in the oven overnight at temperature of 105°C. The sample was cooled in desiccator before weighting. The sample was dried in the furnace again with 550°C for 4 hours (APHA 1985). The weight of sample was obtained after cooled in desiccator. The difference of weight between two samples is the organic content which burned in high temperature.

3.3 Specific gravity test

Specific gravity is defined as the ratio of the density of the material to the density of water. The density of water is taken as 1g/cc at room temperature. To obtain the specific gravity of organic soil, specific gravity bottle method from Test 6(B) of BS1377 Part 2: 1990 was used. About 30g dried organic soil was filled into the 50ml specific gravity bottle until one third of the bottle. Mass of the dried sample was measured. Then, the bottle was filled with water until full. Mass of the bottle with wet soil was recorded. After cleaning the bottle, the mass of water filled bottle was measured.

3.4 Water content test

Water content test was performed based on BS 1377 Part 2: 1990 Test 3, to obtain the moisture content of the soil specimen. The mass of a dry container was measured. Then, wet soil was placed into the container before the mass of the container with wet soil was obtained. The specimen was dried overnight (110°C). The mass of container with dried soil was measured. The moisture content was obtained from the percentage for mass of moisture over mass of dry soil.

3.5 Atterberg Limits Test (Liquid Limit Test)

The liquid limit (LL) of soil is defined as the water content at which the soil would flow under its own weight. The liquid limit test was performed based on Test 4.3 of BS 1377 Part 2: 1990.

The moisture content container was weighted and recorded. The soil sample was poured onto a flat surface and mix with water thoroughly to a uniform consistency using a spatula. Small portion of soil was taken for water content determination. The soil was placed into the cup by using a spatula. The sample was pressed slightly to remove the air voids. The surface of the sample should be leveled to approximately horizontal.

The cup was placed on the base of cone penetration device. The cone was lowered until it reaches the surface of the soil sample. The dial gauge was then set to zero. Penetration started when switch was on. Cone was released to penetrate the soil for 5s and relocked into new position. Reading of dial gauge was taken. Penetration procedure was repeated 3 times on the same paste mix. Average of penetration was obtained. Penetration was repeated with paste mix of 4 different water contents.

A graph was drawn of cone penetration over water content. The liquid limit of soil was taken as the water content corresponding to a penetration of 20mm.

3.6 Atterberg Limits Test (Plastic Limit Test)

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