THE FORMATION, PROPERTIES AND BEHAVIOUR OF COASTAL SOFT SOIL DEPOSITS AT PERLIS AND OTHER SITES IN PENINSULAR MALAYSIA

by

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VOLUME I

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PREFACE

بسم الله الرحمن الرحيم

In the Name of Allah, the Most Gracious and Most Merciful

ABSTRACT

Peninsular Malaysia coastal soft soil deposits have their own unique characteristics and properties which are quite different from other tropical and temperate soft soil deposits. This research study reviews the formation of coastal soft soil deposits with emphasis on the formation of Peninsular Malaysia coastal soft soil deposits. Factors influencing the formation and deposition of Peninsular Malaysia are described. Previous research work carried out on the properties of Peninsular Malaysia coastal soft soil deposits is reviewed.

A detailed study of the properties was carried out on the coastal soft soil deposit at the trial embankment site in Kuala Perlis. The properties of the trial embankment site were determined from field testing, including field vane and piezocone which were carried out prior to construction of the trial embankment. Laboratory testing, including consolidation tests, strength tests, chemical tests and fabric analysis were carried out on undisturbed samples obtained from the trial site.

Field behaviour of the trial embankment was monitored using several types of instrumentation, including pneumatic piezometers, inclinometers and settlements plates. Although two trial embankments were constructed at the trial site, the present research study only deals with the data obtained from the South embankment, which was constructed at a rapid rate of construction and completed in 36 days. The other embankment, North embankment, which was constructed at a slow rate of construction continues to be monitored.

Computer analyses using two standard programs were carried out to compare predicted results with actual field behaviour. Comparison of the field behaviour of the trial embankment were also made with two other trial embankment sites at Muar and Juru in order to study the similarity and differences in the behaviour of the two trial embankments with that at Perlis.

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LIST OF SYMBOLS AND ABBREVIATIONS

c_v : Coefficient of Consolidation

C_α : Coefficient of Secondary Consolidation

C_c : Compression Index

C_s : Swelling Index

C_u : Undisturbed Undrained Shear Strength

C_r : Remoulded Undrained Shear Strength

C_k : Permeability Index

e_o : Initial Void Ratio

e_L : Void Ratio At Liquid Limit

e : Void Ratio At Natural State

f_s : Local Friction

m_v : Coefficient of Volume Compressibility

p_c : Preconsolidation Pressure

q_c : Cone Resistance

R_f : Friction Ratio

u_{max} : Maximum Pore Pressure

w : Natural Moisture Content

m : metres

ASCE : American Society of Civil Engineers

ASTM : American Standards of Testing Materials

B.P. : Before Present

BS : British Standard

CR : Compression Ratio

ISSMFE : International Society of Soil Mechanics and Foundation Engineering

LL : Liquid Limit

OCR : Overconsolidation Ratio

PL : Plastic Limit

PI : Plasticity Index

1D : One Dimensional

2D : Two Dimensional

3D : Three Dimensional

CHAPTER ONE

INTRODUCTION

1.0. Introduction

Due to the rapid growth and extensive development occurring in Peninsular Malaysia and also to cater for the future needs of the Malaysian government for more expertise in certain geotechnical fields such as soft soils and land reclamation, the Geotechnical Research Unit of the Public Works Institute (IKRAM) decided to embark on a research program with the Department of Civil Engineering of the University of Strathclyde, Glasgow in early 1990. As a result of the joint research program, two research engineers from IKRAM were sent to the University of Strathclyde, Glasgow in September 1991 to do their postgraduate degree and research on Peninsular Malaysia coastal soft soil deposits.

The initial research study on Peninsular Malaysian coastal soft soil deposits was reported on by Aziz (1993) who studied the general engineering characteristics and behaviour of Peninsular Malaysia soft soils deposits. Many valuable and interesting data were identified in this study including differences in the nature and properties of Peninsular Malaysia and many South East Asian soft soil deposits with Temperate Region soft soil deposits.

This thesis presents further research work carried out on Peninsular Malaysia coastal soft soil deposits with a detailed study on a specific site. The site chosen for the study was in Kuala Perlis in the state of Perlis where a trial embankment was constructed to study the behaviour of the coastal soft soil deposits.

Detailed field testing was carried out on the trial site together with laboratory testing on the undisturbed samples obtained from the trial site. Instrumentation works were also installed in order to obtain the field behaviour of the trial embankment.

1.1. Objectives of the Research Study

The main objectives of this research study on Peninsular Malaysia coastal soft soil deposits with particular reference to the coastal soft soil deposit at the Kuala Perlis trial embankment site are as follows :

i. To review and study the formation of these coastal soft soil deposits.

ii. To determine and study the properties and characteristics of these coastal soft soil deposits through field and laboratory testing.

iii. To study the behaviour of these soft soil deposits under embankment loading.

1.2. Outline of Thesis

This thesis is divided into nine chapters including this Introduction. The outline of the thesis is briefly summarised as follows :

Chapter Two includes the review of previous work carried out on tropical soft soil deposits. This includes the formation of tropical soft soil deposits, types of field and laboratory testing, types of instrumentation works that are often installed in soft soil deposits and the theoretical analyses of the behaviour of soft soil deposits.

Chapter Three consists of the previous research work carried out on Peninsular Malaysian coastal soft soil deposits which includes factors affecting the formation of Peninsular Malaysia soft soil deposits, their engineering properties and characteristics.

Chapter Four consists of the location and layout of the Kuala Perlis trial embankment site and the type of field and laboratory testing carried out as well as the details of the instruments installed.

Chapter Five includes the analysis of results that were obtained from field testing carried out on the trial site and from laboratory testing of undisturbed samples obtained from the Kuala Perlis trial embankment.

Chapter Six consists of the analysis of data obtained from instrumentation work installed in the trial embankment. This includes data obtained from settlement plates, piezometers, extensometers and inclinometers.

Chapter Seven describes the computer analysis carried out by two consolidation programs and their comparison with field data obtained from the Kuala Perlis trial embankment.

Chapter Eight consists of the review of previous study carried out on the behaviour of embankments on Peninsular Malaysia coastal soft soil deposits. Data obtained from the present study in Kuala Perlis are compared with two other trial embankment sites at Muar and Juru.

Chapter Nine contains the discussion of the findings of the research and the main conclusions.

CHAPTER TWO

REVIEW OF THE FORMATION, PROPERTIES AND TESTING OF TROPICAL SOFT SOIL DEPOSITS

2.0. Introduction

Although much has been written regarding various soft soil deposits throughout the world, this chapter will concentrate only on aspects relating to tropical soft soil deposits similar to the coastal soft soil deposits of Peninsular Malaysia. This can be summarised in three main sections viz. the origin and formation, the engineering properties and the testing of tropical soft soil deposits.

2.1. Origin and Formation of Tropical Soft Soil Deposits

2.1.1. Weathering and Transportation of Weathering Products

Tropical soils are formed from the weathering of parent rocks and the breakdown of various silicate minerals. These are transported by a number of transporting agents like wind or water and are then redeposited. Weathering forms part of the geological cycle of rock genesis. degradation, alteration and reformation, McGown and Cook (1994), and consists of physical and chemical processes which breakdown existing rock and soil masses in-situ. The processes involved in weathering are physical, chemical and biological. Physical processes reduce the particle sizes and increase the surface area and bulk volume while chemical and biological processes may cause a complete change in both physical and chemical properties. The five physical weathering processes are unloading, thermal expansion and contraction, crystal growth including frost action, colloidal plucking and organic activity. Chemical weathering includes hydrolysis, chelation, cation exchange, oxidation and carbonation. Two important factors controlling the rate of weathering are climate and the parent material while topography determines the rate of erosion and controls the depth of soil accumulation and the time available for weathering prior to removal of the material from site.

Two main groups of soil can be classified based on their mode of formation viz. residual soils and sedimentary soils, Ollier (1969) or residual soils and transported soils, Legget and Hathaway (1988) (cited by McGown and Cook (1994)). Residual soils are formed by the direct in-situ weathering of bedrock and have not been transported far enough to change their original structure. Sedimentary or transported soils are formed from weathered materials which have undergone significant transportation.

2.1.2. Redeposition of Tropical Soft Soil Deposits

Tropical residuals soils may be completely different in composition from temperate residual soils while tropical sedimentary or transported soils can have similar properties to those in the temperate regions. McGown and Cook (1994) divided tropical soils into three main groups namely tropical weathered in-situ materials (TWIM's), Tropically Weathered Transported Materials (TWTM's) and Tropically Weathered Transported and Redeposited Materials (TWTRM's).

Tropical coastal soft soil deposits which are relevant to the present study, can be classified as Tropically Weathered Transported and Redeposited Materials (TWTRM's). TWTRM's are soils that have been tropically weathered and transported by gravity, wind or water and have been significantly restructured during redeposition. The soils can be deposited in a variety of depositional environments from freshwater to marine.

The possible types of re-depositional environments of tropical coastal soft soil deposits are shown in Fig.2.1, Kukal (1971). They can be divided into five main types namely deltaic, fluvial, tidal flats and marshes, lacustrine, bay and lagoonal.

This section briefly summarises the main re-depositional environments which are relevant to the present study. These are the deltaic, fluvial, tidal flats and marshes. Lacustrine, bay and lagoonal re-depositional environments are also found in Peninsular Malaysia but are not reviewed as they do not relate to the present study.

2.1.2.1. Deltaic Environment

Deltas are formed from sediments that are carried by rivers to the sea. Deltas are often formed between the boundary of the continental and marine environments. Coarser materials are deposited close to the river mouth while finer grained materials are deposited offshore where the bottom currents are weak. Kukal (1971) states that the factors affecting the distribution of deltaic sediments are :

i. The manner of influx of fresh water into the sea or lake.
ii. Presence of currents e.g. tidal, turbidity or longshore currents.
iii. Climatic factors e.g. wind.
iv. Wave movements.

Deltas may be classified into many types such as birdfoot, lobate, cuspate, arcuate and estuarine. Large deltas are very complex sedimentary structures but they can basically be divided into three components known as topset, foreset and bottomset structures, Fig.2.2. Deltas can be divided into two parts which are known as subaqueous and subaerial deltas. The subaqueous delta is the portion of the delta which lies below the lowest water level and is the foundation on which propagation of the subaerial delta must proceed. The subaqueous delta is characterised by seaward or lakeward fining of sediments with the sand being deposited nearest the river mouths and fine and silty clays settling further offshore. The subaerial delta is the portion of delta above low water level and is normally thinner vertically than the subaqueous delta. The subaerial delta consists of a lower and upper delta plain. The lower delta plain lies within the realm of riverine-marine interaction and extends landward to the limit of tidal or other water level change influences while the upper delta is the older portion above significant tidal or water level change influences. The upper delta is a seaward continuation of the alluvial valley and is dominated by riverine depositional processes. Salt or brackish water vegetation and fauna are not present within the upper delta plain.

The seaward part of the subaqueous delta is known as the pro-delta and is composed of fine materials. Pro-delta clays have lateral continuity and low lithologic variation, Wright (1978). In large deltas the topset and foreset structures are difficult to differentiate, thus terms like pro-delta, intra-delta and inter-distributary environment have been developed. The term pro-delta is synonymous with bottomset, Kolb (1978).

Pro-delta deposits are the first terrigenous sediments introduced into a depositional area by an advancing delta. The majority of the materials are deposited in a broad fan about the delta front. Pro-delta deposits are usually fine grained, plastic clays. The presence of shells is common throughout the material. Pro-delta clays appear to be devoid of structure but in reality, when taken by X-ray radiographs, they show a wide variety of laminae, fractures, contorted beddings and complex displacements, Kolb (1978). The fractures and distortion of the layers are thought to be related to the rate of deposition of the layers. The denser coarse grained materials, which cover the low density pro-delta clays when the delta advances, generate different loading which leads to peripheral slumping, radial tension faulting, diapirism and deep seated clay flowage, Wright (1978).

Topset beds or the interdistributary zone of the delta, are partly subaerial and partly subaqueous and the sediments are heterogeneous. The subaqueous part consist of shallow bays or intertidal flats depending on the range of tides. Muddy sediments are deposited between the distributaries and tidal channels but are frequently interbedded with crevasse splay deposits , Wright (1978). The subaerial part are characterised by marshes and swamps which form highly organic deposits consisting of organic clays or peats intercalated with fine clastics.

Large deltas contain a number of subenvironments, both marine and terrestrial and deposition takes place with a salinity which varies from freshwater to seawater. Deltas with large coastal plains are often major locations for coastal soft soil deposits.

2.1.2.2. **Fluvial Environment**

Fluvial sediments are sediments which are transported by rivers and consist not only of sediments of river channels but include those laid within river valleys. River sedimentation can take place in three types of environment. The three types are the river channels, natural levees and flood plains environments, Kukal (1971). Fluvial sediments or river deposits are usually divided into channel, bank, flood plains and alluvial plains deposits.

Channel deposits usually appear as a unit and are not easily divided into sub environments. The grain size of the channel deposit usually decreases from the central part to the margins of the channel. Channel deposits of mountain rivers are usually distinguished by the existence of bimodal grain-size composition where the medium and coarse sandy fractions are missing. This is due to the reworking and redeposition of sediments while channel sediments of lower rivers usually have less than 5% of gravel, medium or fine grained sand which constitutes the prevailing part of their material. The fundamental form of channel deposition is the formation of sand and gravel bars and mega-ripples.

Bank deposits are usually mistaken for channel or flood plain deposits due to the indefinite character of their petrography, that is when coarse they resemble channel deposits and when clayey-silt, they look like overbank deposits. Bank deposits normally have considerable amounts of carbonates and organic matter. Due to their homogeneity, bank deposits do not show a great variety of structures.

Flood plain sediments usually form the bulk of stream valley deposits. They constitute a sedimentary cover of both channel and bank deposits. The shape of the flood plains are controlled by the shape of stream valley where it attains great width in the valleys of lowland rivers and is narrow, almost undeveloped, in deep cut valleys of streams with steep gradients. The altitude and thickness of flood plains are influenced by the difference between the stream level under flooding and normal conditions. Flood plain deposits are mostly fine grained materials having variable amounts of carbonates and organic matters. They are usually represented by bedding planes and laminations of varying thickness, depending on the amount of sediments deposited during flooding. Homogeneous structures occasionally occur through the greater part of the profile due to the deposition of homogeneous clays or silts. Flood plain deposits can contain a large amount of plant debris which is non uniform in distribution of organic matter and is responsible for the later mottled appearance of the sediments. Mud cracks are often developed on their surfaces. The most reliable indicator of individual environments is the grain size. Structure and chemical composition are secondary diagnostic features and the geological position and shape of the body and lateral transition are auxiliary parameters, Kukal (1971). For flood plain deposits, stream sediments are differentiated from marine sediments on the basis of carbonate content, with the carbonate content increasing with decrease in particle size for stream sediments while for marine sediments the effect is the reverse.

Sediments of alluvial plains are the part of the river deposits which have the greatest thickness and extent. Alluvial sediments are usually deposited near river mouths or on coastal plains and are generally in the lower reaches of major streams. Great thicknesses of these deposits can occur, often due to tectonic subsidence of the plains or sea level rise. The bulk of the sediments of alluvial plains are usually overbank deposits laid down during high flooding. These sediments usually pass into the upper part of the river delta where the transition is gradual. Thus it is sometimes difficult to determine whether or not sediments belong to the delta or the continental alluvial plain, Kukal (1971).

2.1.3. Mineralogy

Tropical soft soil deposits usually consists of sand, silt and clay particles with the percentage in the clay fraction being large. The clay minerals are obtained from the breakdown of various silicate minerals and are essentially hydrous aluminum silicates which contain some other minerals e.g. magnesium, potassium, calcium, sodium and ferrous ions. Clay particles are basically composed of clay minerals which are usually less than 2μ in size, Millot (1978). Clay minerals are classified into a variety of types or groups in which the three main types are kaolinite, montmorillonite (smectite), and illite (hydrous mica). The amounts and types of clay minerals formed at any location depends on climate, parent material, topography and vegetation. Transportation of weathering products can take place in the form of particles or in the form of ions which have been leached from rocks by percolating water. The clay particles in a sediment can have three kinds of origin, Millot (1978) :

i. Neoformation or Authigenesis which is the in-situ crystallisation of clay minerals from ions present in the environment.

ii. Inheritance which is the detrital accumulation of previously formed clay minerals without any modification.

iii. Transformation which is the alteration of previously formed clay minerals due to geochemical changes in the environment. There are two types of transformation. The first is known as degradation which is the removal of ions from the clay structure and the second is aggradation which is the addition of ions to the clay structure.

All of the three processes above are governed by two environmental types. The first environment is a leached environment which destroys the primary silicates, degrades the layer silicates and transforms clay minerals, while the second is a confined environment which is responsible for the aggradation and degradation of clay minerals and for the formation of new clay minerals from concentrated and confined solutions, Millot (1978).

Clay minerals have small colloidal crystals which look like plates or flakes. Details of the clay structure have been studied by Grim (1962) and Mitchell (1976). Most clay minerals are made up of either the tetrahedral (silica) sheet or the octahedral (alumina) sheet. The tetrahedral sheet is a combination of silica tetrahedral units which consists of four oxygen or hydroxyl atoms at the corners surrounding a single silicon atom. The octahedral units consists of two sheets of closely packed oxygens or hydroxyls in which aluminum, magnesium or iron are embedded in an octahedral combination so that they are equidistant from six oxygen or hydroxyl atoms, Grim (1962). Details of the structure of clay minerals are shown in Fig.2.3.

Montmorillonite (smectite) consists of an octahedral sheet sandwiched between two silica sheets. In montmorillonite, extensive substitution of aluminum and silicon occurs within the lattice by other cations. Aluminum in the octahedral sheet may be replaced by magnesium, zinc, nickel, lithium or other cations while aluminum will replace up to 15% of the silicon in the tetrahedral sheet. Some of the silicon positions may also be occupied by phosphorous, Grim (1962). Montmorillonite is also known as a 2:1 clay mineral and is considered to be very unstable and having a strong affinity for water. The absorption of water by montmorillonite can result in swelling. Montmorillonite may therefore be stated to have a very high activity as well as a high liquid limit.

Illite consists of a layer composed of two silica tetrahedral sheets with a central octahedral sheet. Illite has a 2:1 structure similar to montmorillonite but some of the silicons are usually replaced by aluminums and the resultant deficiency is balanced by potassium atoms, Grim (1962). Some illite contains magnesium and iron in the octahedral sheet as well as aluminum, Marshall (1964) (cited by Mitchell (1976)). The properties of illite are generally between the properties of montmorillonite and kaolinite.

Kaolinite consists of a single tetrahedral sheet and a single alumina octahedral sheet which combine to form a common layer, Grim (1962). Kaolinite is considered a 1:1 clay mineral and is found in soils which have undergone considerable weathering. Kaolinite is considered to be a very stable clay mineral having a low liquid limit and a low activity.

Other types of clay minerals also found in tropical soft soil deposits include halloysite, vermiculite and chlorite. Halloysite is a member of the kaolinite subgroup. Halloysite has two distinct forms, one is a non hydrated form having the same structural composition as kaolinite while the other, a hydrated form consists of kaolinite layers which are separated by a single layer of water molecules.

Vermiculite consists of an interstratification of biotite mica layers and double molecular layers of water. Vermiculite may occur in nature as large crystalline masses having a sheet structure similar to mica. In soils, vermiculite occurs as small particles mixed with other clay minerals, Mitchell (1976).

Chlorite consists of alternating mica-like and brucite-like layers. The structure of chlorite is similar to vermiculite except that an octahedral sheet replaces the double water layers between the mica sheets. Chlorite minerals are found as microscopic grains of platy morphology and poorly defined crystals edges in altered igneous and metamorphic rocks. In soils, chlorites appears in mixtures with other clay minerals.

2.1.4. Sea Level Changes and Isostatic Uplift

The Quaternary period was characterised by the ice ages and resultant drastic climatic changes and worldwide sea level fluctuations. The Quaternary period has been divided into the Pleistocene Epoch, in which ice ages occurred and the Holocene (or Recent or Post-Glacial) Epoch which began at the end of last ice age up to the present time. The last ice age in Europe was the Weischel or Wurm stage which reached its maximum 20000 years to 17000 years B.P.. In America, the Post-Glacial

Epoch began only 6000 years B.P. after the retreat of the Wisconsin ice sheet. Most geologists assume that the Holocene/Pleistocene boundary is around 10500 years to 10000 years B.P.. Nearshore soft clays are usually of the late Pleistocene or Holocene age.

The study of sea level changes and isostatic uplift can be divided into three sections viz the fluctuations in sea levels, glacial-isostatic deformation and shoreline migration. The effect of glacial-isostatic deformation is not considered here as the present study is undertaken in Peninsular Malaysia which lies in the tropical region and was not covered by glaciers during the last glaciation. Further no geological data exists to show that during the late Pleistocene or Holocene ages any movement of the land had occurred, Tjia (1977; 1980).

2.1.4.1. **Fluctuations in Sea Levels**

Worldwide, during the last glaciation stage (Winsconsin or Wurm stage), the maximum sea level was thought to be about 137m below the present sea level and that there was a sea level rise during an inter-glacial of approximately 60m to 75m above present mean sea level, Kraft (1978). However Kenney (1964) gave 119m as the lowest sea level elevation.

The Flandrian transgression started to occur about 20000 years B.P. after the last stage of glaciation causing a rise in the sea level. The rise or fluctuation in sea level can also be inferred from palaeoclimatic records, the sea level rising during warmer periods. Evidence of changes in climate and temperature can be found from analysis of former lake levels in arid areas and from biological studies of pollen profiles, Kenney (1964).

Many researchers have established time-depth curves to predict the rise in sea level changes. This can be accomplished by several methods namely :

i. Carbon dating of marine fossils of the remains of plants and animals using the C_{14} dating method.

ii. From geomorphological features such as strandlines or submerged terrestrial features on the continental shelf e.g. raised beaches, submerged beaches etc.

The maximum rise in sea level is still being debated. Some researchers think that the maximum rise in sea level in the Holocene period was about 1m to 3m above present mean sea level, Fairbridge (1966). Other researchers thought that the maximum rise in sea level did not go beyond the present sea level, Jelgersma (1961); Curray (1965) (From Belknap and Kraft (1977)). Published curves of varying sea level hypothesis from various researchers are shown in Fig.2.4.

From the curves produced by Kenney (1964) and Fairbridge (1966), Fig.2.5, it can be seen that the rise in sea level prior to 10000 years B.P. was marked by two major regressions which are explained by glacial readvances or substages. The sea level curves also exhibit periods of standstill which have been deduced from the formation of marine terraces. Kenney (1964) suggests that the sea level then gradually rose until about 5000 years B.P where it remains more or less constant about the present sea level. However, as stated above this varies with the Fairbridge (1966), whose curve shows that the sea level started to fluctuate about 9000 years B.P. and then rose about 3m above present sea level about 6000 years B.P. before gradually subsiding to the present sea level.

Models regarding changes in sea level changes have also been introduced e.g. Clark et al (1978) who modelled relative sea level changes by taking into account of the interaction between ice loads, water loads and a deformable earth (cited by Brenner et al (1981)). Results of such models have shown that the sea level rise due to ice melting was not uniform everywhere and that zones of emerged beaches and regions where submergence was dominant can be predicted. Clark et al (1978) also assumed that there were no stable regions where eustatic sea level changes can be measured because deglaciation and the addition of water to the ocean basin deforms the earth and changes the observer reference point.

2.1.4.2. Shoreline Migration

The depositional environment of sediments in coastal areas are closely connected to the movement of the shoreline. The migration of the shoreline depends on two factors namely the rate of deposition and the rate and direction of sea level changes. A rising level will result in landward migration (transgression) of the shoreline while a seaward migration (regression) will result from a falling sea level. This tendency can be reversed by net deposition and erosion. The interaction of these two processes is given by Curray (1964) and is shown in Fig.2.6.

2.2. Engineering Properties of Tropical Soft Soil Deposits

The engineering properties of tropical soft soil deposits can be obtained from field and laboratory testing. Proper sampling and storage of the samples are needed prior to laboratory testing in order to obtain good samples for testing.

2.2.1. Sampling and Storage Techniques

2.2.1.1. Sampling Techniques

The type of sampling techniques used on soft soil deposits can affect the sample quality obtained. This section briefly discusses some of the factors affecting sample quality and the appropriate type of sampling techniques to obtain good quality samples.

2.2.1.1.1. Sample Quality

The quality of samples taken from site investigation works can be divided into five classes, BS5930 (1981), as shown in Table 2.1. The classes used by BS5930 (1981) are a simplified version of those proposed by Idel et al (1969). The classes of sample quality were based on the particle size distribution, moisture content, dry density and the shear strength or compression index. The lowest class of sample is 5 which can be used for the determination of geological stratum while the highest is 1 which can be used for determining compressibility or shear strength values.

Whyte (1986) mentioned that Hvorslev (1949) classified sample quality as non representative, representative and undisturbed or disturbed samples. High quality samples need to be both representative and undisturbed. Representative samples should be of a sufficient size to reflect grading, fabric and structure of the soil and can be large both in weight and size. Disturbance originates from two main sources that is from soil sensitivity and volume changes. Rowe (1972) gave some guidance on the size of sample required to represent soil materials for a range of parameters.

There are a few methods of assessing sample quality such as radiography, determination of effective stress after sampling and the use of oedometer curves. A detailed review of these methods is given by Jamiolkowski et al (1985).

2.2.1.1.2. Sample Disturbance

The quality of samples obtained from site investigation works are influenced by the amount of sample disturbance during sampling. The processes in which sample disturbance can occur during sampling or testing are described by Hvorslev (1949) (cited by Andresen (1981)) as :

i. Changes in stress conditions.

ii. Changes in water content and void ratio.

iii. Disturbance of the soil structure.

iv. Chemical changes.

v. Mixing and segregation of the soil constituents.

These disturbances are related to sampler design, method of sampling, handling and storage. For sampler design, the main factors that can cause disturbance during sampling are wall thickness and edge taper, inside and outside clearances, sampler dimensions and the friction between the soil and sampler. For wall thickness and edge taper, this is kept to a minimum by using thin walled tubes, while Hvorslev (1949) recommended a maximum of 10° for angle of cutting edge tapers for composite samplers. For inside and outside clearances, Hvorslev (1949) defined that an inside clearance results when the inside diameter steps up not far from the tapered end while an outside clearance exists when the outside diameter steps down not far from the end, Fig.2.7. Hvorslev (1949) mentioned that although outside clearances increased the area ratio, a clearance of 2% to 3% can be advantageous in clay (cited by Andresen (1981)).

Clayton (1986) reviewed the BS5930 (1981) for site investigation works and described three main factors controlling sample disturbance namely design of the equipment, maintenance of the equipment and the technique of sampling. Clayton (1976) mentioned that the ISSMFE subcommittee (1965) noted that the combination of area ratio and cutting edge taper are important in order to obtain high quality samples and suggested the combinations shown in Table 2.2 for 75mm diameter tubes. For clays the extreme edge of the cutting shoe can be given a thickness of 0.3m. For granular soils this thickness can be up to 10% of the grain size of the soil. Other factors influencing the design of rotary and driven samplers is the combination of the length/diameter ratio, inside clearance and the adhesive or frictional properties of the inside of the sampler barrel. Hvorslev (1949) suggested the use of 0.75% to 1.5% inside clearances for loose-dense cohesionless soils of L/D $\ngtr$ 5 to 10 and for subcommittee of ISSMFE (1965) suggested the clearances shown in Table 2.3, Clayton (1986).

2.2.1.1.3. **Sampling Techniques**

Proper sampling techniques are necessary to ensure that good quality samples can be taken. Sampling techniques can be divided into four main types, BS5930 (1981). The four types of sampling techniques are disturbed sampling, drive sampling, rotary sampling and block sampling. The sampling techniques relevant to this study are as follows :

(a) **Disturbed Sampling**

Disturbed sampling are taken during the drilling of boreholes. The quality of samples depends on the method of drilling as well as whether the ground is dry or wet. For disturbed samples taken underwater there can be a danger that they are not representative of the deposit especially for non cohesive soils containing fines, BS5930 (1981).

(b) **Drive Sampling**

Drive sampling is where a sampling tube is being forced into the ground by either a static or dynamic thrust. The type of samplers used for drive sampling can be divided into four main types, BS5930 (1981), which are open tube samplers, piston samplers, continuous soil samplers and sand samplers. Sand samplers will not be discussed here as they are not relevant to the present study on coastal soft soil deposits.

(i) Open Tube Samplers

Open tube samplers consist of a tube open at one end and fitted at the other end to the drill rods. Samples are taken as the tube is pushed into the ground. A typical open tube sampler is shown in Fig.2.8. Normally for sensitive soils, thin walled samplers are used to minimize sampling disturbance. 100mm open tube samplers are often used with cable percussion boring for different types of cohesive soils and in weak rock, BS5930 (1981). For non sensitive fine cohesive soils of stiff or lower consistency, class 1 or 2 samples can be obtained while for sensitive clays only class 2 samples can be obtained. For brittle or closely fissured materials such as weak rock and stiff clay, class 3 or 4 samples can be obtained depending on whether water has been added to the borehole, BS5930 (1981).

(ii) Piston Samplers

Piston samplers consist of a thin walled sampling tube with a close fitting sliding piston which is slightly coned at its lower face. The sampling tube is fitted to the drive head and connected to the drill rods. The piston is fixed to separate rods which pass through a sliding joint in the drive head and up inside the hollow rods. The diameter of piston samplers are normally 75mm or 100mm but sometimes a diameter of 250mm is used for special soil conditions. The piston is locked to the lower end of the sampling tube to prevent water and slurry from entering. In soft clay when the sampling tube is pushed in the ground at a specified depth, the piston is held stationary and the tube is driven by a static thrust until the drive encounters the upper face of the piston. An automatic clamp in the drive head prevents the piston from dropping down and extruding the sample as it is being withdrawn. Samples of class 1 quality are usually obtained with piston sampling. This is normally used for low strength fine cohesive soils and sensitive clays. Specially designed piston samplers are also used in stiff clays, BS5930 (1981). A typical piston sampler is shown in Fig.2.9.

(iii) **Continuous Soil Samplers**

Continuous soil samplers is often used in sensitive soils and for identifying soil fabric. It usually obtains superior quality samples to those obtained by consecutive drive samplers. The Swedish system uses 68mm diameter tubes using steel foils to eliminate side friction between the sample and the tube wall while the Delft system uses lighter equipment and offer two sizes of samplers. A typical Delft continuous soil sampler is shown in Fig.2.10, BS5930 (1981).

2.2.1.2. **Handling and Storage of Samples**

Proper handling and storage of samples is important in order to maintain the quality of samples prior to testing. This is specially true for sensitive clays, Bozozuk (1976). Samples should be carefully waxed, labelled and stored properly free from shocks or vibrations in a humid environment at a temperature close to the initial ground temperature during sampling, Bozozuk (1976).

Bozozuk (1976) also mentioned that the storing of soils in a warm humid environment can lead to high rates of oxidation which then cause chemical changes in the soil, deterioration of the sealing wax and formation of water blisters and can also breed bacteria that feed upon the soil or wax. High temperatures also cause dissolved gases in saturated clays to come out of solution thus causing expansion and deterioration of the soil skeleton. This allows soil moisture to escape and causes shrinkage. For soil samples stored submerged in water, the soil sample may absorb water and swell thus no longer representing the actual soil characteristics on site.

Bozozuk (1976) stated that grain size analysis and Atterberg limits of sensitive marine clays appear not to be affected by long storage periods even though the colour change frequently indicating some chemical changes has occurred. However, other researchers have found that the shear strength of clays reduces after a long storage period, Arman and McManis (1976). Preconsolidation pressure also appears to produce the same trend, however hand cut blocks did not show a decrease in strength or preconsolidation pressure with time, Arman and McManis (1976). Generally the degree of saturation and wet density decreases and the water content increases somewhat by extending storage time. There is also a reduction in shear strength and a small decrease in strain at failure, Bozozuk (1976).

Arman and McManis (1976) described that samples stored by wrapping in plastic film and aluminum foil were found to be preserve as well as those coated with paraffin wax. Arman and McManis (1976) proposed that tests should be carried out on samples in less than fifteen days after sampling to prevent errors due to deteriorating effects of long term storage.

2.2.2. Field Testing

There are many types of field testing that can be carried out on soft soils. This section will review some of the field testing methods used for Peninsular Malaysia soft soil deposits and which are relevant to the present study. These include field vane, penetration and permeability testing.

2.2.2.1. Field Vane Tests

Field vane tests are usually carried out in the field to give an estimate of the in-situ undisturbed and remoulded shear strength of the soft soil deposits. Field vane tests on soft soil deposits are briefly described in three sections. These include the principle of field vane testing, factors influencing field vane testing and the interpretation of field vane results.

2.2.2.1.1. Principle of Field Vane Testing

The field vane apparatus was first developed in Sweden and Germany between 1928 and 1929, Andresen (1981), and has grown in popularity due to its low cost and reasonable accuracy. The field vane apparatus consist of four rectangular metal plates attached at right angles to a central torque shaft to form a cross, Fig.2.11. The vane is pushed vertically into the soil and an increasing torque is then applied until failure of the soil occurs. The undisturbed shear strength of the clay (C_u) is calculated from the maximum torque (T) achieved at failure.

Cadling and Odenstad (1950) (cited by Andresen (1981)) derived the following equation for the field vane for the calculation of undisturbed shear strength and is given as :

$$C_u = \frac{2T}{\pi D^2\left(H+\frac{D}{3}\right)} \qquad \text{(Eqn.2.1)}$$

where

C_u = undrained shear strength
H = height of vane
D = diameter of vane
T = torque

Flaate (1966) modified the above formula by assuming that the strength on the ends of the cylinder was mobilised in proportion to the rotation strain from zero to C_u at the centre.

Flaate (1966) equation is given by;

$$C_u = \frac{2T}{\pi D^2\left(H+\frac{D}{4}\right)} \qquad \text{(Eqn.2.2)}$$

For the standard vane where H=2D, the values of Eqns.2.1 and 2.2 will differ only by 5%.

Field vane tests can be carried out at the bottom of boreholes or by using a vane borer of which the latter is the most common. Field vane tests should not be performed in soft soils having large amounts of shells, stones, or sand and silt layers, Flaate (1966).

2.2.2.1.2. **Factors Influencing Field Vane Testing**

There are many factors influencing the result and interpretation of a field vane test. Among the most important factors influencing field vane testing are the rate of rotation, the delay time, the vane sizes and shapes and the vane insertion, Tortensson (1977) and Andresen (1981).

(a) **Rate of Rotation**

The rate of rotation of the field vane plays an important part in influencing field vane results. Tortensson (1977) studied the rate of testing using seven different testing rates from one second to seven days. He concluded that the shear strength obtained by the field vane is appreciably affected by the testing rate and that at low rotational speed, the stress-strain curve does not resemble that of a strain softening material but of an ideal elastic-plastic material. In contrast to this, the residual shear strength is found to increase with decreasing rate of rotation. The angular rotation of the vane is also independent of the various speed of rotation. Tortensson (1977) also described results of three field vane tests in which the maximum torque was mobilised in seven days. He concluded that additional shear stresses can be induced by increasing the speed of rotation to 10000 times that of the standard speed. These additional stresses then decrease as the angle of rotation increases before any change in the speed of rotation occurs. For large angles of rotation, the increase in the speed of rotation will only result in a very small increase of the shear stresses. For practical purposes a rate of rotation of 0.2° per second is often adopted as standard, Andresen (1981).

(b) **Delay Time**

Normally in field vane testing, there is a delay time between the time of installation and testing. The longer the delay time, the more inaccurate the results achieved from the vane tests. A delay time of not more than five minutes has been recommended in order not to influence the result obtained in the field, Flaate (1966) and Chandler (1988). Longer periods after vane insertions will result in some consolidation and increased undrained shear strength results. For example, rest periods of more than one hour may result in an increase of strength greater than the reduction resulting from vane disturbance, Chandler (1988). Tortensson (1977) described from his study of three hundred field vane test results that a lapse of one day will increase the maximum torque values by 10% to 20%. Tortensson (1977) attributed this increase to the increase in adhesion between the vane blades and the soil with time after the installation of the vane, which leads to a reduction of stress concentration in the vicinity of the vane blades and a larger volume of soil being involved in the test.

(c) **Vane Sizes and Shapes**

Several researchers have investigated the effect of different vane shapes. Aas (1965) studied results obtained from vanes of seven different shapes and found that the ratio of undrained shear strength acting along the horizontal failure surface to that of the vertical failure surface was around 1 to 2 depending on the type of clay. The only exception was data obtained from some vanes having a height/diameter ratio of three or more which was considered unreliable. Flaate (1966) recommended that the size of the vane should follow a height to diameter ratio of two with the diameter and height of the vane equal to 50mm and 100mm. Normally the most preferred common shape of the field vane is the rectangular shape type.

i. The distribution of shear stresses around the vertical edges of the vane blades can be assumed to be uniform while at the top and bottom surfaces non uniformity exists.

ii. Vane insertion causes disturbance that results in underestimation of in-situ undrained strength and is most severe in sensitive clays. Published estimates indicate a maximum strength loss of about 15% but losses of 25% may be inferred for highly sensitive clays, e.g. Norwegian clays.

Azzouz et al (1983) carried out 3D analyses on 18 case histories of embankment failures to study the end effects of embankment failures which are usually ignored in plane strain analyses. Azzouz et al (1983) proposed that a new correction factor rather than the normal Bjerrum correction factor be used to estimate the actual in-situ undrained shear strength from measured field vane data. Azzouz et al (1983) concluded that the inclusion of end effects generally increases the conventional plane strain factor by about 5% to 15%.

Mayne and Mitchell (1988) suggested that empirical corrections be made to field vane data to account for the effects of strain rate, anisotropy and disturbance on measured shear strengths. Mayne and Mitchell (1988) also suggested that the field vane apparatus be calibrated at each site to develop profiles of OCR with depth. Mayne and Mitchell (1988) compiled a database of the results of oedometer and field vane strengths from ninety six different clays to use as a basis for the calibration. Mayne and Mitchell (1986) concluded from their analyses that there is a general relationship existing between OCR and normalised undrained strength to overburden ratio (C_u/σ_{vo}'). This was found to be different from that obtained from laboratory tests which was explained on the basis that the values differ due to different normalised shear strengths obtained from different test types.

The ratio of undisturbed and remoulded undrained shear strength (C_u/C_r) of clays is known as the sensitivity of the clay. This values varies from 1 for some overconsolidated clays to over 1000 for quick clays. Skempton and Northey (1952) concluded from their experimental work that thixotropy can account for low to medium sensitivity but not for high sensitivity. Field and laboratory results showed that leaching of salts from the porewater in clays can result in high sensitivities. Skempton and Northey (1952) also showed that the water content and the undisturbed strength of clay remains unaltered by the leaching process, although the liquid limit and the remoulded strength are reduced.

Mitchell and Houston (1969) reviewed some of the work done on the sensitivity of clays. They presented classification of sensitivity by several researchers, as shown in Table 2.4. Mitchell and Houston (1969) concluded that there were eight types of mechanisms which can cause sensitivity in clays as shown in Table 2.5. Houston and Mitchell (1969) found that sensitivity of clay increased in a consistent manner with respect to liquidity index and increasing effective stress. The low remoulded strength of quick clays relative to the undisturbed strength is due to a change in fabric from flocculated to deflocculated upon remoulding. Sensitivity may increase or decrease or remain constant depending on the degree of flocculation in the specimen during consolidation and if consolidation becomes high enough the result will be a decrease in sensitivity.

2.2.2.2. **Penetration Testing**

Another common field test used in the site investigation of soft soil deposits is the cone penetration testing. There are many types of cone penetrometers used in field testing, the most common being the Dutch Cone and the Electric Cone as shown in Fig.2.13.

2.2.2.2.1. **Dutch Cone Penetrometer**

The Dutch cone penetrometer is a mechanical or static type of cone penetration test. It has been used in Holland and Belgium since its introduction in the 1930's, De

Ruiter (1971). The test consists of a continuous penetration of a 60° cone of 10cm^2 at a rate of 20mm/sec. In the test, the penetration resistance and the local friction are recorded. The application of the tests was previously limited to the determination of the presence and extent of soft clays but recently has been used for the determination of the shear strength of the soft soils.

The undrained shear strength of the soft clay is calculated directly from the cone resistance using the equation :

$$q_c = NC_u + \gamma z \qquad \text{(Eqn.2.3)}$$

where

q_c = cone resistance

C_u = undrained shear strength

γ = bulk density

z = depth

N = cone factor

The measurement of local friction, f_s , by means of friction sleeve enables fairly accurate soil identification. Friction ratio or friction index are commonly used and are given by :

$$R_f = \frac{f_s}{q_c} x\ 100\% \qquad \text{(Eqn.2.4)}$$

and

$$I_f = \frac{q_c}{f_s} \qquad \text{(Eqn.2.5)}$$

where

R_f : friction ratio

I_f : friction index

f_s : local friction

q_c : cone resistance

The mechanical Dutch cone consists of a cone attached to a rod which is housed in an outer tube leading up to the ground surface. The system is mechanically discontinuous in that the cone and inner rod is pushed a short distance about 70mm and stopped before the outer tube follows. The outer tube eliminates the friction that would otherwise exists between the rods and the clays so that cone resistance is measured directly. Mechanical friction occurs between the rods and the tube which must be allowed in the interpretation of the data.

An improved version of the Dutch cone is the mantle cone which was later developed to incorporate a friction sleeve (or adhesion jacket) to enable the skin friction between the sleeve and the clay to be measured at any depth. The mantle cone testing involves the penetration of the cone alone then the cone plus the jacket and finally the outer tubes. The point adhesion is deduced from the difference between the resistance of the cone alone and the resistance of the cone plus the sleeve, Andresen (1981).

2.2.2.2.2. **Electric Cones**

Cone penetration testing was improved by the introduction of the electric cone penetrometer in the 1960's, De Ruiter (1971). The electric cone allow a continuous measurement of the cone resistance, q_c, and the local shaft friction, f_s, which are sensed by electrical strain gauge load cells. Details of the electric cone penetrometer is given by De Ruiter (1971). Several researchers have also introduced data acquisition systems to enable data from the cone to be stored in a data logging system which has helped in the enhancement of the use of electric cone system which has helped in the enhancement of the use of electric cone penetrometers, Jamiolkowski et al (1985).

The electric cone has the following features, ISSMFE (1977) and ASTM (1986) :

i. The cone has an apex angle of 60° and a base area of $10cm^2$.
ii. The friction sleeve is located immediately behind the cone and has an area of $150cm^2$.

Attempts by several researchers have been made to correlate the cone resistance with the friction ratio e.g. Begemann (1965) . The following factors should be considered when using the charts of cone penetration with friction ratio, Jamiolkowski et al (1985) :

1. A shift in the reference electronic signals (zero readings) from both the load cells incorporated in the cone tip is observed when electronic cones are subjected to hydrostatic pressure, De Ruiter (1981) and Campanella and Robertson (1983). This is due to the unequal areas of both the cone and friction sleeve on which the water is acting. Thus q_c and f_s do not represent the total resistance of the surrounding soil. The values are somewhat lower depending on the specific construction of the cone.

2. When using friction ratio (R_f) for identification purposes, care must be taken because both the q_c and f_s are influenced by the initial effective lateral stress σ'_{ho} as described by Schmertmann(1972)(cited by Jamiolkowski et al (1985)). The initial effective lateral stress σ'_{ho} is an important parameter which reflects strongly the stress history of the penetrated deposit. Schmertmann (1978) and Baldi et al (1983) (cited by Jamiolkowski et al (1985)) described that for sands, the influence of σ'_{ho} seems to be more important on f_s than on q_c. For sensitive clays the value of f_s is close to zero due to severe remoulding causing a large reduction in σ'_{ho}. Therefore the R_f is questionable in sensitive clays.

3. Schmertmann (1978) also described the following points for stratified deposits (cited by Jamiolkowski et al (1985)) :

i. The thickness of the thin stiff layers embedded in the soft soil mass should exceed approximately 70cm in order for the cone tip to achieve full q_c at mid height.
ii. For thin soft layers in a stiff deposit, the minimum thickness for correct measurement of q_c should exceed 20 to 30cm.
iii. The presence of thin soft embedded layers in stiffer soil deposits requires digital output of q_c and f_s at least every 2cm. Even with this the detection of soft lenses and layers whose thickness is less than 20cm requires much experience and expertise.

4. The success of the electronic CPT for soil profiling and identification requires standardisation of the cone design and calibration and testing procedures. It is also important to maintain the standard penetration rate of 2cm/sec as the measured soil response may be influenced by the permeability and strain rate sensitivity of the soil deposit.

5. Almost all electric cones in use have load cells with a maximum capacity of 50kN to 80kN which allows penetration of soils ranging from soft clays (q_c<100 kPa) to very dense sands ($q_c \not> 30000$kPa). Thus in soft to medium cohesive deposits, q_c suffers from low electronic resolution of the load cell which may be barely reliable. This is important when values of FR or u_{max}/q_c are to be evaluated. To overcome this problem, Ridgen et al (1982) (cited by Jamiolkowski et al (1985)) designed a cone tip with two load cells, one carrying a maximum q_c of 50000kPa and the q_c load of 5000kPa. A specially designed overload mechanism protects the more sensitive cell once its maximum design load has been reached.

Cone Penetrometers can also be used for pore pressure monitoring as first recognised in the mid 1970's by Janbu and Senneset (1974) and Schmertmann (1974). This led to the development of the Pore Pressure Probe by Tortensson (1975) (cited by Jamiolkowski et al (1985)) and the Piezocone, Janbu (1974) and Schmertmann (1974).

The development of the Piezocone has enabled measurements of q_c, f_s and u_{max} (penetration pressure) to be taken simultaneously which is a distinct advantage of the piezocone over other electronic cones. The Piezocone can be used for the following applications, Jamiolkowski et al (1985) :

i. Soil profiling and identification.
ii. Tentative assessment of the stress history of cohesive deposits.
iii. Evaluation of the flow and consolidation properties in cohesive deposits.
iv. Assessment of ground water conditions.
v. Indication of liquefaction susceptibility of sand deposits.

Some of the problems that arise from usage of the piezocone are determination of the magnitude of measured pore pressure depending on the location of the filter, clogging and excess wear of the filter. Details of Piezocone applications and problems are described by Jamiolkowski et al (1985) and Meigh (1987).

2.2.2.3. **In-situ Permeability Tests**

There are many types of in-situ permeability tests such as borehole permeability tests by pumping, permeability tests using piezometers, self boring cells, piezocone dissipation tests. In-situ permeability values can also be back calculated from field measurements. This section will briefly discuss only permeability tests using piezometers as these are adopted in the present study of Peninsular Malaysia coastal soft soil deposits.

2.2.2.3.1. **Permeability Tests in Piezometers**

Piezometers can be installed in predrilled holes, Casagrande (1946) (cited by Jamiolkowski et al (1985)) or pushed below the bottom of a borehole, Wilkes (1970) and Parry (1971). In general, tests on fine grained soils are difficult to perform and interpret due to changes in the effective stresses which cause changes in the consolidation and flow parameters.

Jamiolkowski et al (1985) identified the main sources of errors in permeability tests using piezometers as :

i. The danger of hydraulic fracturing due to the installation and to the head applied during the test.
ii. Smearing and remoulding of the surrounding soil during pushing of the piezometer.
iii. The permeability of the porous element with respect to the soil.
iv. The hydraulic time lag.
v. The presence of gas in the pore water.

Hydraulic fracturing can be avoided by using constant head tests with applied excess pore pressures as low as possible but sufficient to obtain a reasonable flow rate. To minimise smear effects it is suggested that the distance to push the piezometer below the bottom of the borehole should be restricted to twice the length of the tip. Further the piezometer should be kept saturated during the insertion. When applying the hydraulic head difference, consolidation or swelling occurs in the soil surrounding the piezometer and the measurement of the flow rate allows the computation of both the permeability and consolidation coefficient, Wilkinson (1967). Bjerrum et al (1972) did some laboratory tests to confirm that hydraulic fracturing can occur in-situ and that low pressure must be applied during permeability testing to overcome this problem. Bjerrum et al (1972) also developed a mathematical analysis to indicate the factors which influence the allowable pressure that can be used in permeability testing.

In a constant head test where the flow is directed from the piezometer towards the soil (outflow test), the coefficients of permeability and consolidation obtained are representative for the unload-reload condition. The values reflect the soil behaviour in the overconsolidated state and tend to be appreciably higher than those obtainable for primary loading corresponding to normally consolidated conditions.

2.2.3. Laboratory Testing

There are many types of laboratory tests to determine the properties of soft soil deposits. The types of laboratory testing can be classified into several groups such as classification, consolidation, strength, permeability, soil fabric and chemical tests.

2.2.3.1. Classification Tests

Classification testing of soft soils includes particle size distribution tests (sieve and hydrometer analysis), Atterberg limits tests (liquid and plastic limit tests), specific gravity, bulk density and moisture content. Details of the test procedures are described in BS1377 (1990) Part 1 and by Head (1980).

Various relationships have been introduced by several researchers from data obtained from classification tests, perhaps the most important is the introduction the A-Line by Casagrande in 1948 where soils with similar behaviour were group into zones. The A-Line is a relationship of plasticity index with liquid limit. The original A-Line by Casagrande given by the equation PI=0.73(LL-20) divides the clays and the silts into the top and bottom of the A-line, with soils having organic content being placed below the A-Line. Several researchers have proposed some modification to the A-Line, Seed et al (1964), Magnan (1980) and ASTM (1990). The modification to the Casagrande A-line suggested by Magnan (1980) (cited by Leroueil et al (1990)) and

ASTM (1990), includes slightly organic soils at the top and bottom of the A-Line, this is due to experience gained from testing slightly organic soils.

Data from classification tests have also been correlated with field data e.g. liquidity index versus remoulded shear strength, Houston and Mitchell (1969), sensitivity versus liquidity index, Leroueil et al (1983) (cited by Leroueil et al (1990)), shear strength/effective overburden pressure versus plasticity index, Bjerrum (1954) etc.

2.2.3.2. Consolidation Tests

Consolidation testing is considered one of the most important tests for soft soil deposits. Olsen (1977) defined consolidation testing as the efforts to measure generalised stress-strain-time relationships for soils under conditions of partial or complete dissipation of excess pore water pressures. Consolidation testing can be summarised into two sections, viz the type of consolidation testing and the interpretation of soil parameters from consolidation testing.

2.2.3.2.1. Types of Consolidation Testing

There are two main types of consolidation test apparatus adopted for soft soils. These are the standard Oedometer and the Rowe Cell, Fig.2.14. The standard Oedometer test is performed in a ring of 50mm diameter and a thickness of 20mm. The soil is loaded with dead loads in increments of twice the pervious load. The compression of the sample is measured either at end of primary consolidation or after preselected time usually 24 hours. Samples are loaded up to the maximum applied pressure and then unloaded in a series of decrements where the pressure is one fourth of the previous pressure. If the sample is taken under the water table in the field, this is simulated in the laboratory by using a seating load of 1kPa for very soft clay and peats and 10kPa for stiff clays. For stiff fissured clay, it is preferable to start with a seating load equal to the effective overburden pressure so that some fissures that opened up because of stress relief will be closed again. Variations of the standard

Oedometer test procedures include the single increment loading, constant rate of strain tests, controlled gradient test and continuous loading tests.

The Rowe Cell tests was introduced by Rowe and Barden (1964) and consist of a consolidation cell subjected to hydraulic pressure which is applied through a rubber diaphragm. The consolidation cell body is made of an aluminum bronze casting and the cell base is made of steel. The base and cover are bolted to the flanges on the cell body at eight positions with an `O' ring providing the seal at the base and the rubber loading jack providing the seal at the cover. The Rowe Cell can come in various diameters, the more common are 76mm and 154mm diameters. The Rowe Cell tests can be carried out using vertical and horizontal drainage conditions with a one way or two way drainage. Pore pressures can also be measured during testing. The advantages of the Rowe Cell over the standard Oedometer are :

i. It is easier to provide high loads for large diameter samples.
ii. The load can be applied using a flexible or a rigid platen. A flexible platen will also localize the effects of side friction.
iii. The sample is not subject to vibrations effects magnified by a lever system. which can be particularly troublesome in long term creep tests.
iv. The error in settlement caused by compression of the loading is negligible even with stiff samples.
v. It is cheaper to use if a large number of tests are to be undertaken.

Some of the disadvantages of the Rowe Cell are :

i. Longer test periods due to the larger sizes of sample compared to the Oedometer.
ii. Preparation of sample and setting up of the test is much more difficult compared to the standard Oedometer tests.
iii. Difficulty in controlling the applied pressure on the sample especially at low pressure.

iv. Larger size sampling tubes are needed in order to obtain undisturbed samples for testing compared to the standard Oedometer.

Details of consolidation testing using the standard Oedometer and Rowe Cells are described in BS1377 (1990) Parts 5 and 6 and by Head (1982; 1986). A more detailed review on consolidation testing is given by Olsen (1986).

2.2.3.2.2. **Interpretation of Consolidation Test Data**

Interpretation of consolidation tests parameters is essential in understanding the behaviour of soft soils. Crawford (1986) reviewed the evaluation and interpretation of soil parameters obtained from consolidation testing. The soil parameters obtained from consolidation testing include preconsolidation pressure (p_c), compression index (C_c), coefficient of consolidation (c_v), coefficient of secondary consolidation (C_α), swelling index (C_s) and the coefficient of volume compressibility (m_v)

(a) **Preconsolidation Pressure (p_c)**

One of the main objectives of consolidation testing is to estimate the maximum preconsolidation pressure that may be applied to the soil without causing large settlements. The preconsolidation pressure is the maximum load on a natural soil during its geologic history, although this effect may be caused by aging of the clay, Casagrande (1936), Leonards and Altschaeffl (1964) and Bjerrum (1973). For over consolidated soils, the preconsolidation pressure is poorly defined on the e-logσ' curve but for normally consolidated soils there is a dramatic change in compressibility at the preconsolidation pressure. There are various methods of determining preconsolidation pressure, the most common being the Casagrande graphical method. Other methods includes methods proposed by Schmertmann (1955) and Jose et al (1989). Leroueil et al (1983) carried out a series of special tests to determine preconsolidation pressure from samples obtained from the Glouscester test site in Canada. They concluded that all Oedometer test data show an unique preconsolidation pressure-strain rate relationship for a given clay at any given depth and that the standard oedometer tests corresponds to the slowest rate of strain and consequently yields the lowest preconsolidation pressure. Other special tests yielded much higher values of preconsolidation pressure consistent with higher rates of strains applied.

(b) **Compression Index (C_c)**

The compression index (C_c) is the slope of the linear portion of the virgin compression e-log σ' curve. Disturbance of the soil sample tends to decrease the C_c value but variation in tests methods seem to have little influence on it, Crawford (1986). Schmertmann (1955) demonstrated with samples from various depths in a post glacial deposit of silty clay, that the position of the virgin portion of the e-log σ' curve represents essentially an unique property of the deposit and is independent of the initial void ratio (e_o) and preconsolidation pressure (p_c). Mesri and Rokhsar (1974) reported a relationship between C_c and water content for some natural soils as shown in Fig.2.15.

Various correlations have been proposed by various researchers e.g. Lambe and Whitman (1963), Azzouz et al (1976) and Krizek et al (1977) (cited by Balasubramaniam and Brenner (1981)), using compression index (C_c) or compression ratio CR=(C_c/1+e_o) with other soil parameters. Some of these relationships are shown in Table 2.6. Nagaraj and Murthy (1986) did a critical reappraisal of compression index equations and derived two generalised compression index equations for normally consolidated saturated uncemented fine grained soils. The equations derived by Nagaraj and Murthy (1986) are :

$$C_c = 0.23 e_L \qquad \text{(Eqn.2.6)}$$

and

$$C_c = 0.39e \qquad \text{(Eqn.2.7)}$$

where

e_L = void ratio at liquid limit

e = void ratio at natural state.

(c) **Coefficient of Consolidation (c_v)**

Rate of primary consolidation is defined by the coefficient of consolidation (c_v) which is expressed by :

$$c_v = \frac{k(1+e)}{a_v \gamma_w} \qquad \text{(Eqn.2.8)}$$

where

k = coefficient of permeability

e = void ratio

a_v = coefficient of compressibility

γ_w = unit weight of water

Various methods have been used for evaluating the coefficient of consolidation (c_v), the most common being the Taylor's square root of time and Casagrande's logarithm of time fitting methods. Generally the values of c_v obtained from Taylor's square root of time method for normally consolidated clays are typically 1.5 times to 2.5 times larger than those obtained by the Casagrande method, Ladd (1973) (cited by Balasubramaniam and Brenner (1981)). Other curve fitting methods includes Scott's method (1962), Su (1958) and Parkin (1978) (cited by Balasubramaniam and Brenner (1981)).

Various coefficient of consolidation curve fitting methods have also been introduced for radial flow e.g. Shields and Rowe (1965) and Berry and Wilkinson (1969) (cited by Balasubramaniam and Brenner (1981)).

The coefficient of consolidation can be obtained from pore pressure dissipation data during consolidation which can be obtained from a plot of pore pressure dissipation versus time graph, e.g. Crawford (1964). The coefficient of consolidation can also be obtained from results of constant gradient tests and constant rate of strain tests, Balasubramaniam and Brenner (1981). Wong and Choa (1990) used c_v data obtained from Oedometer tests together with finite difference method to predict the rate of settlement and they found that the predicted results using c_v from three case studies were encouraging.

(d) **Secondary Consolidation**

Secondary consolidation is defined as the reduction in volume of a soil mass caused by the application of a sustained load to the mass due principally to the adjustment of the soil structure after most of the load has been transferred from the water to the soil solids, ASTM D653 (1983). The rate of secondary consolidation is expressed by the coefficient of secondary consolidation (C_α) as :

$$C_\alpha = \frac{\Delta e}{\Delta logt} \qquad \text{(Eqn.2.9)}$$

which represents the change in void ratio over one log cycle of time after the primary consolidation under a load increment is completed.

Lo (1961) carried out a series of long term consolidation tests on five remoulded and two undisturbed clays. Lo (1961) concluded that the secondary compression curves can be divided in three types as shown in Fig.2.16 :

i. Type I curve where the rate of secondary compression decreases with time.
ii. Type II curve where the rate of secondary compression is proportional to the logarithm of time for a considerable range of time and then decreases.
iii. Type III curve where the rate of secondary compression increases with time then gradually vanishes.

Mesri (1973) considered secondary compression to be a continuation of the mechanism of volume change during the primary phase. The mechanism involves the deformation of individual particles and the relative movement of particles with respect to each other. He concluded that soils having high compressibility in the primary phase also have a high secondary compressibility and produced a Table showing a classification of compressibility based on the coefficient of secondary consolidation, Table 2.7.

Mesri and Godlewski (1977) studied the relationship of C_α and C_c and found that there were no differences in the mechanism of volume change during the primary and secondary stages of consolidation. They concluded that in a particulate system every mechanism of volume change, whether viscous or non viscous, is a chain reaction process and is time dependent. They also gave the ratio of C_α/C_c for twenty two natural soil deposits which ranges from 0.025 to 0.1 with the higher values applying to organic soils, Fig.2.17 and Table 2.8. Mesri and Godlewski (1977) also observed that C_α is not a function of the load increment ratio but is dependent on the applied effective stress and its relation to preconsolidation pressure.

Katagiri (1993) investigated the relationship between C_α and C_c for three types of samples, Fig.2.18. He studied samples from clay/seawater mixtures with different water contents. Undisturbed and remoulded samples with different skeleton structure and inter-connected samples were made by the same method. Katagiri (1993) concluded clays have an unique C_α-C_c relationship and that the relationship is independent of the sedimentation condition, stress history, differences in soil structure, location of specimen and loading conditions.

(e) Swelling Index (C_s)

Consolidation test results in the recompression range are generally unreliable due to a number of factors such as small movement, apparatus errors, sample disturbance, and swelling. Simons and Soms (1969) (cited by Crawford (1986)) concluded that satisfactory results can be obtained only when initial swelling is positively prevented and apparatus and bedding error are eliminated. They suggested that the soil should be tested by subjecting the specimen to the stress system initially prevailing in the ground and then applying as closely as possible the same stress changes as those to which the soil will be subjected in the field. Das (1990) mentioned that the values of the swelling index (C_s) can be approximated as 1/5 to 1/10 of the value of the compression index.

(f) Coefficient of Volume Compressibility (m_v)

The coefficient of volume compressibility (m_v) is another parameter which can help in the understanding of the compressibility of soft soils. The coefficient of volume compressibility (m_v) indicates the compressibility per unit thickness of the soil and is expressed by :

$$m_v = \frac{a_v}{1+e_o} \qquad \text{(Eqn.2.10)}$$

where

a_v = coefficient of compressibility

e_o = initial void ratio

or is expressed in terms of c_v by

$$m_v = \frac{k}{\gamma_w c_v} \qquad \text{(Eqn.2.11)}$$

where

c_v = coefficient of consolidation

k = coefficient of permeability

γ_w = unit weight of water

Table 2.9 shows the relationships between the volume of compressibility and compressibility for typical British soils.

2.2.3.3. Shear Strength Parameters

2.2.3.3.1. Undrained Shear Strength (C_u)

The undrained shear strength (C_u) of soft soils is not a unique property of the material but depends on several factors such as the orientation of the maximum shear stress, the rate of application of loading, the size of samples used in the testing and the type of test and the measuring apparatus used, Parry and Wroth (1981). The undrained shear strength (C_u) also depends on the stress path followed during the tests and on the test itself, Leroueil et al (1990). The undrained shear strength of soft soil deposits can be obtained from strength tests where no drainage is allowed to take place during the test. The types of test used in determining undrained shear strength in the laboratory are the direct shear box, laboratory vane, triaxial and unconfined compression tests, Fig.2.19. Details of the types of testing and procedures are described in BS1377 (1990) Parts 5 and 6 and by Head (1982).

(a) Direct Shear Box Tests

Direct shear box tests are usually carried out in a standard shear box of dimensions 60mm square although other sizes e.g 100mm and 300mm have also been used. The type of direct shear box test for measuring the undrained shear strength is known as the quick shear test. In this test, no drainage is allowed during testing. The maximum time used for quick shear tests is about 20 minutes. The sample is first prepared and saturated overnight before shearing. Shearing of the sample is carried out at a rate of 1mm/min until failure occurs. The readings of the load, horizontal and vertical displacement are recorded up to the point where a decrease in load is observed i.e. after failure occurs. Shearing of the sample is continued if the peak or failure cannot be determined until the maximum length of travel of the shear box has been reached. Usually a minimum of three quick shear tests are carried out under different normal pressures in order to obtain a failure envelope. The different types of failure envelopes that are obtained from quick shear tests are shown in Fig.2.20.

(b) **Laboratory Vane Test**

Measurement of undrained shear strength using the laboratory vane is similar in principle to the in-situ vane tests. The difference being that the laboratory vane has vane dimensions of 12.7mm by 12.7mm compared to the in-situ vane dimensions of 150mm by 75mm. Experience has shown that results obtained from laboratory vane are compatible with those from the unconfined compression test, Head (1982). The laboratory vane test is suitable for soft sensitive clays having undrained shear strengths of 20kN/m^2 or less. Laboratory vane tests can be done in a sampling tube as well as in a compaction mould, Head (1982). Prior to testing, four torsion springs of the laboratory vane are calibrated to determine the torsional constant of each spring. The most suitable spring is then chosen for the type of soil to be tested. The laboratory vane is then inserted about four blade lengths below the surface of the sample in order to give a minimum cover of 50mm to the vane. The vane is rotated at a steady rate of about 1turn/sec until failure occurs. The maximum angle of deflection of the spring at failure is then recorded. The torque or the undrained strength is then calculated by

$$c = \frac{K\theta_f}{4.29} \qquad \text{(Eqn.2.12)}$$

where

c= applied torque

K=torsional constant of the spring (Nmm/rotation)

θ_f= maximum angle of deflection at failure

If the deflection of the spring reaches 100°, the test is discontinued and a stiffer spring is then used. For remoulded strength, the vane is rotated rapidly about two complete revolutions and the test is then repeated. Laboratory vane test is undertaken at four to five locations in order to obtain an average value for the undrained shear strength of the soil.

(c) **Triaxial Testing**

The type of triaxial test used for the determination of the undrained shear strength is the unconsolidated undrained test. The test is carried out under undrained conditions where no drainage is allowed during the application of the cell pressure or deviator stress. Prior to testing, the undisturbed sample is first extruded from the tube and the sample is prepared according to the diameter of the sample to be tested. The most common sample diameters used in the triaxial test are 38mm, 50mm and 100mm. The sample is first weighed and prepared using a rubber membrane which is then fitted with end caps and held by O-rings. The sample is then put into a triaxial cell which is filled up with water. The sample is then saturated until a pore pressure coefficient (B) value of 1 is achieved. The unconsolidated undrained test is then carried out by shearing the sample at a speed of about 1.5mm/min for 38mm diameter samples or 4mm/min for 100mm diameter samples until failure occurs. The readings of the load gauge are recorded at regular strain intervals. The test is completed when failure occurs or when a strain of 20% has been reached. The motor is then stopped and switched to the reverse direction to allow the specimen to unload. The failure mode of the sample is then recorded from two directions at right angles and the failed sample then weighed. The failed sample is usually weighed twice, viz. after the test and after drying overnight, in order to check the mass of the sample before and after testing. A stress-strain graph is plotted from which the stress-strain at failure is determined. For the unconsolidated undrained tests, three samples are usually tested at different stresses. One sample is tested at in-situ stress conditions while the other two samples at twice and three times the in-situ stress conditions. From the three tests, three different values of stress-strain at failure at determined and from these values, three mohr circles can be drawn. The average undrained shear strength of the soil is then determined from the intercept of the y-axis which is tangential to the three mohr circles. The undrained angle of friction (ϕ_u) which the angle of the tangential line to the horizontal are also determined. Usually for saturated clay, the undrained angle of friction (ϕ_u) is approximately zero. For the unconsolidated undrained test, no measurement of the pore pressure is carried out during the test.

(d) **Unconfined Compression Test**

The unconfined compression test is a quick test to measure the undrained strength in terms of total stress. The test is carried out by applying an axial compression at a rate of 2% per min to a cylindrical specimen until failure occurs. No drainage and no change in moisture content is allowed to take place during the test. An axial load is applied during the test and is recorded together with the corresponding strain until failure occurs. A load strain graph is then plotted. The test is repeated three times to determine the average undrained strength. The unconfined compression test is a simplified version of the unconsolidated undrained triaxial test, the difference being that in the unconfined compression, only an axial load is applied while for the unconsolidated undrained triaxial test, a confining pressure is first applied all round the sample before an axial load is applied.

2.2.3.3.2. **Consolidated Undrained Shear Strength**

The consolidated undrained shear strength of a soil can be determined from consolidated undrained shear box or triaxial testing. The preparation and testing procedures are similar to those described for the shear box and unconsolidated undrained triaxial test in the previous section except that the shearing rate of the samples is determined during consolidation of the sample prior to shearing. The shearing rate is detennined by drawing a settlement-time graph during the consolidation of the sample to derive the t_{100} intercept from which the time at failure and the shearing rate can be calculated as explained by Head (1986). The shearing rate is usually determined for every sample but since the testing machine often have standard shearing rates. The nearest shearing rate to the value calculated is often used and usually one shearing rate is used for every test which consist of three samples.

Often pore pressures are also measured during the consolidated undrained triaxial test. No drainage is allowed to take place during the shearing stage of the test similar to the unconsolidated undrained triaxial test. Consolidated undrained strength parameters are usually used in stability analysis of an embankment or slope which have consolidated to various degrees. The parameters obtained from consolidated undrained tests are C_{cu} and ϕ_{cu} for total stress parameters and c′ and ϕ' for effective stress parameters if pore water pressures are measured. C_{cu} and ϕ_{cu} values are used for stability analysis for undrained conditions while c′ and ϕ' values are used if drainage have occurred during the consolidation of an embankment or slope. The details of the tests are described by BS1377 (1990) Part 6 and by Head (1986).

2.2.3.3.3. Consolidated Drained Shear Strength

The consolidated drained shear strength of soils can be obtained from direct shear box or triaxial testing. Details of the tests are described by BS1377 (1990) Parts 5 and 6 and by Head (1982; 1986). In the shear box test, the consolidated drained strength is determined using the slow (drained) test. The testing procedure is similar to the quick drained test except that the sample is first saturated overnight and then consolidated and sheared under drained conditions. For the triaxial test, the consolidated drained strength is determined from the consolidated drained triaxial test. The test is similar to the consolidated undrained triaxial test except that the drainage is allowed to take place during the shearing stage of the testing. Consolidated drained strength parameters c' and ϕ' are usually used in the long term stability analysis of embankments or slopes.

2.2.3.3.4. **Residual or Large Strain Shear Strength**

The residual or large strain shear strength is the shear strength that occurs after the peak strength has been achieved and is measured after the shear stress reaches a steady state as displacement increases. Residual strength can be obtained for undrained, consolidated undrained and consolidated drained tests conditions from both the direct shear box test and triaxial tests although the former is preferred because of the large displacements needed in order to reach the residual condition.

Residual strength can be determined in the shear box apparatus using the multi reversal procedure or in the ring shear apparatus. The tests are carried out until the limit of the travel of the shear box is reached after which the box is returned to its starting position where the shearing is continued. The process is repeated several times until a constant value of shear resistance is achieved.

Residual strength can also be determined from a triaxial test if the sample contains a discontinuity such as a naturally sheared surface and this can be obtained within a reasonable axial displacement. An intact sample is first sliced to make a shear plane inclined at an angle θ, and is then fitted back together and tested as in a normal triaxial test. A ball bearing top cap is used instead of the normal top loading cap to eliminate tilting of the top cap, the development of additional internal forces and lateral forces on the cell piston. A cell fitted with a rotating bush should be used if the load is measured by an external load ring, Head (1986). The preparation and testing procedure is similar to the consolidated drained triaxial test until a steady residual condition can be identified before terminating the test. Residual shear strength parameters have been used for the study of long term stability of slopes and cuttings in overconsolidated clays, Skempton (1964), Skempton and La Rochelle (1965) and Symons (1968) (cited by Head (1982)).

2.2.3.3.5. Remoulded Shear Strength

Remoulded shear strength can be obtained by remoulding or recompacting the material that were tested for undrained shear strength. Remoulded shear strength can be divided into remoulded undrained shear strength, consolidated undrained remoulded shear strength and consolidated drained remoulded shear strength.

The remoulded undrained shear strength, consolidated undrained remoulded shear strength and the consolidated drained remoulded shear strength can be obtained from tests similar for the determination of the undrained shear strength, consolidated undrained shear strength and the consolidated drained shear strength e.g. laboratory vane, direct shear box, unconfined compression and the triaxial compression test.

2.2.3.4. Permeability Tests

The coefficient of permeability of clays is influenced by mechanical and physico-chemical effects. Mechanical effects includes the size, shape and geometrical arrangement of the particles. Reduction in the coefficient of permeability values can occur depending on the size of individual flow channels and increase in the tortuosity of the flow paths, Mesri and Olsen (1971). Physico chemical variables also exert a great influence on the coefficient of permeability through their influence on dispersion or aggregation of the clay particles.

Permeability values of a clay sample can be determined by several laboratory methods, the most common being from a measurement of coefficient of consolidation (c_v) from an Oedometer tests. Tavenas et al (1983a) have shown that this test greatly underestimates the value of the coefficient of permeability (k). They also described that the constant head permeability test in the triaxial apparatus seems to be the best method of estimating vertical or horizontal permeability of intact clay however such tests are slow and expensive. Therefore the most practical method for permeability testing is the falling head method in the oedometer although the small size specimens may not be totally representative, Tavenas et al (1983a).

Tavenas et al (1983b) provided a comprehensive laboratory study of the permeability of intact soft clays from Canada, USA and Sweden and made the following conclusions :

i. None of the existing relationships between permeability and void ratio are generally valid e.g. Samarasinghe et al (1982), irrespective of the type of clay, void ratio or the range of void ratio change.
ii. A linear relationship given below by the Eqn.2.13 is good for void ratio of less than 2.5 and of volumetric strains of practical interest in engineering.

$$\log k = \log k_o - \frac{(e_o - e)}{C_k} \qquad \text{(Eqn.2.13)}$$

where

k = permeability of the soil

k_o = initial permeability

e_o = initial void ratio

e = void ratio at natural state

C_k = permeability index

iii. The permeability of intact soft clays at their in-situ void ratio is a function of not only the void ratio and grain size but also of the plasticity index and the fabric of the soil. Permeability was seen to increase with increase in liquidity index.
iv. Anisotropy is not a significant feature of the in-situ permeability of marine clays
v. The permeability index (C_k) is simply correlated to the initial void ratio.

Tavenas et al (1983b) modified the conventional Oedometer test for permeability testing as shown in Fig.2.21. He studied the effect due to variations of effective stresses and volumetric changes during the tests and found that the effects are negligible for small samples having a diameter and a height of 5cm and 2cm respectively and for permeabilities higher than $2x10^{-11}$ m/s.

Pane et al(1983) presented evidence that small gradients are required to obtain valid permeability measurements in soft soils. The gradients used must be such that seepage induced effective stresses are substantially less than the maximum past effective stress. In normally consolidated soils, induced errors are always present in normal permeability tests (falling head and constant head tests), the softer the soil the greater the error. These errors are caused by the high gradients used in the tests which cause deviations from Darcy's law to occur. This can be readily minimised by using the pump flow method which can be performed faster than the conventional tests.

2.2.3.5. **Macrofabric and Mesofabric Studies**

The macrofabric and mesofabric levels of soil are where features of the soil can be observed with the naked eye or with a hand lens, respectively. Rowe (1968) introduced a method of assessing macrofabric and mesofabric of continuous samples by cutting them vertically halfway through and splitting them open horizontally. The split sample is then allowed to dry and the fabric features can be more easily distinguishable. McGown et al (1977) studied the nature and distribution of fabric features and engineering properties of stiff fissured glacial lodgement tills, boulder clays and soft laminated silty clays of West Central Scotland. They demonstrated that the macrofabric and mesofabric features must be considered when determining sample representativeness and they suggested that representative undisturbed test specimens should contain twenty or more macrofabric/mesofabric features. McGown et al (1980) proposed a method of characterising macrofabric and mesofabric features in the soil based on their nature, form and spatial arrangements. McGown et al (1980) subdivided the fabric into three levels of arrangement, viz. basic, related and referred levels. These three levels allow correlations between the individual groups and various external axes to be made, thus allowing quantitative relationship between the macrofabric and engineering properties to be developed.

2.2.3.6 Microfabric Studies

The microfabric level of the soil is the level where the fabric of the soil can only be seen with a specialised equipment such as a scanning electron microscope, polarising microscope etc.

van Olphen (1963) divided particle associations in clay suspensions into four main types, Table 2.10 :

i. Dispersed where there is no face to face association of clay particles.
ii. Aggregated which consists of face to face association of several clay particles.
iii. Flocculated which consist of edge to edge or edge to face association.
iv. Deflocculated where there is no association between aggregates.

In natural deposits however, single clay platelets arrangement are extremely rarely and various forms of clay platelets are usually observed, Barden (1972). Particle associations in sediments and residual soils assume a variety of forms but most of them are related to those suggested by van Olphen. A comprehensive study was carried out by Collins and McGown (1974). They pointed out that a distinction should be made between studies involving laboratory prepared samples and natural soil deposits. In laboratory studies, usually monomineralic clays are involved and the particle arrangement is controlled by the mineralogy and concentration of clay in the sediments as well as the salt content of the water, Collins and McGown (1974). Sides and Barden (1971) studied microfabric features of dispersed and flocculated samples of kaolinite, illite and montmorillonite and found that the surface activity of the clay has an influence on the resulting fabric. O'Brien (1971) studied laboratory prepared kaolinite and illite and found that there was little differences between the arrangement of kaolinite flocculated in distilled water or in slightly saline water. O'Brien(1971) described that in both cases the "stair stepped cardhouse" arrangements were dominant. Illite floccules in distilled water also were dominated by "stair stepped cardhouse" arrangements but in salt water there were even mixtures of edge to edge flocculation of individual platelets together with "stair stepped cardhouse" arrangements with the latter being abundant. Smalley and Cabrera (1969) also reported similar findings of microfabric features to those of O'Brien in mechanically compacted kaolinite which they described as "stepped clusters". Aylmore and Quirk (1960) called a parallel arrangement of particles in an illitic clay "domain" or "turbostratic" group.

Collins and McGown (1974) studied some natural soils of clays, silts and sands that were deposited in marine, brackish and freshwater environments. Collins and McGown (1974) identified three main microfabric features for natural soils :

i. Elementary particle arrangements which consist of single forms of particle interaction at the level of individual clay, silt or sand particles.
ii. Particle assemblages which are defined as the units of particle organization having definable physical boundaries with a specific mechanical function and which consist of one or more forms of elementary particle arrangements or smaller particle assemblages.
iii. Pore spaces within and between the microfabric features described in types i and ii.

Collins and McGown (1974) concluded that there is little evidence to support the idea that an unique microfabric is directly associated with a certain particular depositional environment. Collins and McGown (1974) found that single clay platelet arrangements are rare and that groups of clay platelets were common in the fabric of natural soils. They also observed "bookhouse" and "stepped flocculated" arrangements were more common in marine soils and that "dispersed or turbostratic" group arrangements occurred mainly in brackish water soils.

Pusch (1966) conducted a study of the micropores in the sensitive marine clay of Sweden. He compared the results with those obtained from fresh or brackish water clays and found that two dimensional relative pore area in the marine clay was about three times greater, although the unit weights and moisture contents were about the same. Pusch also found that the ratio of total pore area in a micrograph over the total area of a micrograph for two different salinities was much higher for marine clays than for freshwater clays with the same preconsolidation pressure and water content. Pusch (1973a, 1973b) (cited by Brenner et al (1981)) introduced a model to show that aggregates of closely packed particles were connected by bridges or links of fine clay plates. Pusch stated that in marine clays, the aggregates are large and dense separated by large voids while in fresh water the aggregates are small and relative porous and the voids small. Pusch (1973a) did not observe any preferred orientation in marine and brackish water illitic clays, while in soft fresh water clays, larger particles greater than 0.5μm tend to be aligned with their long axis normal to the direction of the consolidation pressure. The type of mineral also plays an important part in particle arrangements. Illitic clays consist of randomly arranged particles although "booklike" aggregates also occur. In kaolinitic clays "booklike" aggregates are the most common while for smectites the fabric may be controlled by adsorbed cations, Pusch (1973b). Additional effects on the fabric are caused by organic matter. Pusch (1973a) mentioned that organic matter appears in the form of fluffy bodies but also as distinct objects in clay particle aggregates.

In any microfabric study, the following procedures should be followed in order to achieve high quality samples for microfabric study. The first is the method of preparation of the samples and the second is the choice of the type of method used for microfabric study.

2.2.3.6.1 **Methods of Preparation**

Prior to any microfabric study, samples need to be properly dried and prepared. A few of the methods commonly used for the preparation of samples for microfabric studies are air drying, freeze drying, critical point drying and impregnation. Details of each method of preparation of samples are reviewed by Smart and Tovey (1982). This section will only discuss the freeze drying method of preparation of sample as it is relevant to the present study.

Freeze drying has been used successfully by several researchers to avoid the damage to the samples caused by surface tension. Samples of 10mm diameter by 5mm thickness are quickly frozen using liquid nitrogen or propane cooled by nitrogen. Other gases have also been used with liquid nitrogen e.g. Arcton 13, Freon 13, Isopenthane etc, Smart and Tovey (1982). The freezing of the sample is carried out at temperatures of less than -130°C to avoid formation of crystalline ice. Sublimation of the water is then carried out at temperatures between -50° and -100°C. The minimum time required for sublimation is 24 hours with the time increasing with decreasing temperature. For temperatures less than -100°C the vapour pressure of the ice is ~ 10^{-5} torr which may be less than the capability of the vacuum system.

After drying is complete, the sample is attached to a stub and is then fractured or peeled in order to obtain a dried fresh surface. Several methods of fracturing the samples were reported by Smart and Tovey (1982), among these were the use of anvil and light hammer blows, snapping (by hand using a combined bending and pulling action) and fracturing using a tensile force. The disadvantage of the fracturing technique is that some disturbance might occur during fracturing and also it is difficult to obtain a fractured surface in the intended plane for samples with open structures, Wong and Tovey (1975). Peeling techniques have been tried using silver dag by Tovey (1970) and Wong and Tovey (1975), and various resins, Barden and Sides (1971), as well as adhesive tape. The use of adhesive tape may cause a reorientation of particles because a force needs to be exerted on the sample during peeling. Wong and Tovey (1975) also tried using gelatin which seems to give more repeatable results but was still considered unsatisfactory. Wong and Tovey (1975) reported the use of a more suitable resin (Devcon 5 Minute Epoxy) which enables excellent peeling to be done even with very large samples. Successful peeling can be done using this resin in about 10 minutes to 15 minutes. The number of peelings depends on the type of soil and density of the sample.

2.2.3.6.2. **Methods Used in Microfabric Studies**

Methods commonly used in the determination of microfabric in fine grained soils are the polarising microscope, electron microscope, x-ray diffraction and pore size distribution. Table 2.11 shows all of the methods used in the study of soil fabric. Prior to any study, all samples need to be properly prepared. For the present study only the method using the electron microscope will be discussed.

The electron microscope is the only technique available that can reveal particles and particle arrangement directly, Mitchell (1976). There are two types of electron microscope that are used. They are the transmission electron microscope (TEM) and the scanning electron microscope (SEM). Details of operation of the electron microscope are described by Smart and Tovey (1982). The practical limit of resolution using the TEM is about 20Å and the SEM about 100Å. The major advantages of the SEM over the TEM are the very great depth of field and the wide range of magnification available. Further, the surfaces can be studied directly by the SEM, Mitchell (1976).

The main difficulty of using electron microscopy is the preparation of the sample surfaces that retain the undisturbed fabric of the original sample. Soils containing expansive clay minerals can cause problems due to removal of interlayer water which may cause undetected changes in the microfabric or excessive shrinkage.

Examples of microfabric studies undertaken on natural soils using the scanning electron microscope are Barden et al (1971), Barden and Sides (1971), Collins and McGown (1974) and Collins (1978).

Developments in electron microscopy is the use of cathodoluminescent observations using SEM to provide internal crystallographic information on sand grains which may be related to stress history of the material, Tovey (1973) (cited by Mitchell (1976)). Another is the use of high voltage transmission electron microscope (HVEM) which makes use of thicker sections as well as direct study of wet samples in a special cell. The HVEM can also give information on internal structure of the sample e.g. Cabrera and Hammond (1973) (cited by Mitchell (1976)) who studied dislocation networks in non clay particles from a quick clay.

2.2.3.7. **Chemical Tests**

The most common type of chemical tests that are carried out on soft soils are tests for chlorides, sulphates, carbonates, organic content. Other chemical tests are tests for pH, pore water salinity and sulphides. Details of laboratory testing procedures are described in BS1377 (1990) Part 3 and by Head (1980).

2.2.3.7.1. **Chloride Content**

The type of tests commonly used for the determination of water soluble and acid soluble chlorides are the Volhard's method and Mohr's method, BS1377 (1990) Part 3 and Head (1980). Another test is the qualitative test which gives a rough estimation of the chloride content in the soil, Head (1980).

2.2.3.7.2. **Sulphate Content**

The methods of sulphates testing that can be done on soils include testing for total sulphates in soils and water soluble sulphates in soils by using the gravimetric or ion exchange method, BS1377 (1990) Part 3 and Head (1980).

2.2.3.7.3. **Carbonate Content**

BS1377 (1990) Part 3 recommends two type of tests for the determination of carbonate content i.e. the rapid titration method and the gravimetric method. The rapid titration method is suitable for soils in which the amount of carbonates exceeds 10% and where an accuracy of 1% is sufficient. Other tests recommended are the Collins Calcimeter method, Head (1980), wet combustion method, frizzle test, Passon apparatus, Chittick apparatus and Moum apparatus, Larsson et al (1987).

Larsson et al (1987) carried out a review on carbonate testing and identified that the best testing method was the Chittick or Moum apparatus. The Passon apparatus also gave reasonable values. Larsson et al (1987) concluded that the accuracy for carbonate tests is about 0.5% at low carbonate contents (<5%) and the spread of results increases as the carbonate content increases.

The classification of carbonate content as given by Schon in 1965 is shown in Table 2.12, Leroueil et al (1990).

2.2.3.7.4. **Organic Content**

The type of tests recommended by BS1377 (1990) Part 3 for determination of organic content are the Walkley and Black's Method, loss on ignition method and the peroxide method. Other tests includes dichromate oxidation method, Head (1980), wet combustion, dry combustion, rapid titration, calorimeter method, measurement with reflections, Larsson et al (1987).

Arman (1970) evaluated the physical properties of organic soils using natural and artificial soil specimens. Arman (1970) found that soils having as much as 20% organic material can be used as a foundation material without ill effects. From his tests, Arman (1970) made the following conclusions :

i. The pH value decreases as the organic content increases.
ii. The moisture adsorption increases while the specific gravity of the soil decreases as the organic content increases. Further, when the organic soils are dried they do not absorb the same quantity of moisture that they originally contained.
iii. The permeability of soils containing more than 40% organic matter is governed by the permeability of the organic matter. The permeability of sandy clay decreases with addition of organic material whereas the permeability of silt and clay increases with the addition of organic material.
iv. The consolidation of soils having less than 20% organic material reasonably conform to Terzaghi's consolidation theory therefore settlement can be reasonably predicted.
v. Both unconfined compressive strength and triaxial shear strength of organic soils increase as organic content increases.

Franklin et al (1973) did a series of compaction and unconfined compressive strength laboratory tests on slightly organic soils. Some of the conclusions made by Franklin et al (1973) :

i. No differences were observed in soil-peat mixtures compared to slightly organic soils at similar organic contents.
ii. Oven drying of more organic fractions resulted in greater optimum densities and lower optimum water contents than those mixtures with equal organic content in which the more organic content was air dried.
iii. The unconfined compressive strength versus organic content shows a high degree of scatter indicating that the effect of organic content less than 20% was less important than other mineralogical or structural differences of the soil.

Slunga and Helander (1985) studied the influence of organic content on the undrained shear strength in cohesive soils. They used three methods for the testing of organic content. The three methods used were the colometric method, loss on ignition method and the wet combustion method. Slunga and Helander (1985) found that the results of organic content from wet combustion tests were the average of results from the colometric and loss on ignition method. Slunga and Halender (1985) also mentioned that in highly organic soils, the progressivity of failure has to be taken into account.

Larsson et al (1987) investigated the methods used for determination of organic content. They described that in organic tests, the influence of carbonates are taken into account but for sulphide soils, the organic tests do not take account of the sulphides in the test. Larsson et al (1987) described the advantages and disadvantages of most of the methods used in determination of organic content like loss on ignition tests, treatment with hydrogen peroxide, determination of the content of organic carbon, dry combustion, wet combustion, rapid titration, calorimeter method and measurement with reflections. Larsson et al (1987) concluded that the accuracy of organic content tests is about 0.5%. The accuracy of the tests reduces to 1% at 6% organic content and decreases further with increasing organic content which is due to the relatively small size of specimens tested. Higher accuracy can also be obtained by using the calorimeter method where the accuracy is about 0.2%.

Larsson (1990) studied the properties of fine grained soils containing organic matter and their variation with the composition of the soil. Larsson (1990) found that for constant rate of strain (CRS) Oedometer tests, the normal standard testing and interpretation procedures are applicable for soils with organic content but the maximum rate that can be used on organic soils is 0.0024mm/min. Larsson (1990) also found that the rate of effects expressed by the coefficient of secondary consolidation increase in the transition zone between mineral and organic soils with increasing organic content and associated increase in water content. For highly organic soils this trend becomes broken but in peat, the rate effects seem to be related more to the degree of humification.

The permeability of organic soils was found to be lower than for inorganic soils. The sensitivities of organic soils increased with increasing liquidity index but they are generally lower than for inorganic soils. Leroueil et al (1990) described the classification of soil introduced by Magnan (1980) with respect to organic content as shown in Table 2.13.

2.2.3.7.5. **Other Chemical Tests**

Other chemical tests that can be carried out on soil soil deposits include pH, pore water salinity and sulphide tests.

The common tests for determination of pH values are the pH indication papers, colorimetric or Kuhn's method, Lovibond comparator and the electrometric method.

Pore water salinity tests are carried out according to ASTM D4542 (1985) using the refractometer. The test was developed for soils having a water content equal or greater than 14%. Pore water is gradually squeezed out of the soil into a syringe using a presser having a maximum pressure of 80MPa. Details of the apparatus are shown in Fig.2.22. The pore water salinity is then obtained using a refractometer.

Bjerrum (1954) studied the effect of salinity in the pore water of Norwegian quick clay deposits. Bjerrum (1954) concluded that for Norwegian clays, the activity of the clay reduces with decreasing salt concentration leading to the lowering of Atterberg limits. Also the leaching of salt in the pore water of marine clay reduces the shear strength of the clay and that high sensitivity of marine clays is a result of leaching of salt in the pore water. Torrance (1976) studied the effects of chemical change in low salinity Leda clay during periods of storage. Torrance (1976) reported the use of a pore water extractor for extraction of pore water salinity. He concluded that none of the storage procedures used were satisfactory in that there were significant changes in the pore water chemistry of the samples stored. Torrance (1976) suggested that pore water salinity tests should be done as soon as possible and if there is a need for storage, the best storage procedure is to extrude the sample, wrap and wax, and store under refrigerated conditions.

Sulphide tests in the soils can be carried out using wet combustion accompanied by titration. Other methods includes freeze drying in a vacuum and then react with hydrochloric acid. Larsson et al (1987) described the best method for determination of sulphides as being the wet oxidising, adsorption of evolved sulphur and titration method which yields values with sufficient accuracy for geotechnical investigations.

2.3. Monitoring the Behaviour of Tropical Soft Soil Deposits

Monitoring the behaviour of tropical soft soil deposits includes the study of field behaviour obtained from instrumentation works as well as the prediction of behaviour obtained from theoretical analyses.

2.3.1. Field Behaviour of Soft Soil Deposits

The field behaviour of soft soils is normally studied from data collected from instruments that were installed in the field prior to construction. The study of the field behaviour of soft soils can be divided into four sections namely pore pressures, settlement, lateral or horizontal displacement and stability problems.

2.3.1.1. Pore Pressures

Pore water pressure is an important parameter in trying to explain the performance of an embankment/foundation etc in the field. Excess pore pressure induced in a heavily loaded foundation clay can be determined from the following basic equation, Skempton (1954) :

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \qquad \text{(Eqn.2.14)}$$

where

Δu = excess pore pressure

$\Delta \sigma_1$ = increment of major principal stress

$\Delta \sigma_3$ = increment of minor principal stress

A = pore pressure parameter

The values of $\Delta \sigma_1$ and $\Delta \sigma_3$ are obtained from theories of stress distribution in an elastic medium alone (elastic theory) and from laboratory measured values of compressive strength. The pore pressure parameter A is obtained from laboratory soil tests. There are many types of instrument used for measuring pore water pressure in-situ. The most common types being the open standpipe piezometer, electric piezometer, pneumatic piezometer and hydraulic piezometers.

2.3.1.1.1. **Open Standpipe Piezometer**

This is the simplest and least expensive type. It can be read directly and is quite accurate. It is generally installed in a borehole but some can be pressed or driven in soft clay deposits. The major disadvantage of this instrument is that there is a relatively long time lag between a pore pressure change and its registration in the standpipe.

2.3.1.1.2. **Electric Piezometer**

This type of instrument consists of an electric sensor which responds to the strain or deflection of an elastic element deforming under pressure. The main advantages of this instrument are a remote reading capability, continuous output and rapid response while the major drawback is high cost. The instrument is reasonably accurate. Some instruments can be calibrated in-situ which is a great advantage for long term measurement.

2.3.1.1.3. **Pneumatic Piezometer**

This are similar to the electric piezometer except that air or fluid is used to counter balance and monitor the pore pressure acting on the piezometer. Pneumatic piezometer are usually remotely read as it generally cannot give a continuous output and also cannot be put into a data acquisition system. Pneumatic piezometer generally cost less than the electric piezometer.

2.3.1.1.4. **Hydraulic Piezometer**

This is generally of the closed type system and consists of a U tube manometer or other type of pressure gauge attached to the piezometer in order to reduce the flow of pore water in the system and thereby improve the response time.

The types of piezometers used in the monitoring of pore pressure in the field are as shown in Fig.2.23. Details of the types of piezometers and their advantages and disadvantages as well as installation procedures are described by Dunnicliff and Green (1988).

There are four types of pore pressure dissipation behaviour that can be observed in the field. These are a marked change in dissipation rate, a very slow or insignificant pore water pressure dissipation for long periods following construction, a slow relatively constant rate of dissipation following construction and a minor change in dissipation following completion of construction, Crooks et al (1984). Although pore pressure data has been used for controlling stability of embankment during construction in the field, DeLory et al (1965), Margason and Symons (1969) and D'Appolonia and Lambe (1971), anomalous behaviour of pore pressure has also been observed in the field, Crooks et al (1984) and Mitchell, (1986), which may lead to problems in controlling stability during construction. Crooks et al (1984) reviewed 50 cases relating to anomalous behaviour in the field. The anomalies were due to :

i. The magnitude and distribution of initial pore water pressure including continued pore water pressure generation following completion of loading.
ii. The differences between field consolidation rates and those predicted based on laboratory measurements.
iii. The changes in rates of dissipation during and after construction.

Much of the anomalous behaviour can be explained as due to factors such as the location and effectiveness of drainage layers between assumed and actual stress paths, the non linearity of the pressure versus void ratio relationship, variation of permeability with changes in void ratio and proper evaluation of stress history and current stress state. However the failure of pore pressures to dissipate at a rate commensurate with the settlement and corresponding reductions in void ratio and water content has not been fully explained e.g. the Vasby test fill in Sweden and in a test fill in Penang, Malaysia, Mitchell (1986). Several researchers attributed it to the structural breakdown of clay during compression which causes an increase in compressibility and decrease in permeability. The sudden collapse of the clay structure will lead to a large decrease in the coefficient of consolidation thus a slowing down of the rate of consolidation. The structural breakdown of the clays can generate excess pore pressures which add to the partially dissipated excess pore pressure induced by initial loading, Mitchell (1986).

Theoretical models have been used by several researchers to predict pore pressure behaviour in-situ. Some of the models or theories used for pore pressure predictions are :

i. Classical Terzaghi's theory.
ii. Terzaghi's-Rendulic theory, Dunn and Razouki (1982); Younger (1992).
iii. A simple stress strain theory model, Burland (1971).
iv. YLIGHT (Yield Locus Influenced by Geological History and Time) model, Tavenas and Leroueil (1977) and Leroueil et al (1978).

v. SEPOL (Settlement Problem Oriented Computer Language) analysis, Schiffman et al (1970).

Most of these models are reasonably accurate for predictions of normal pore pressure dissipation but modification to the method/model may be needed for anomalous behaviour of pore pressure dissipation.

2.3.1.2. **Settlement**

Settlement of a structure can result in many different types of problems e.g. flooding, tilting of structures, cracks etc. This is due to settlements never being uniform and differential settlements being very common.

Settlement can be divided into three components :

i. Immediate settlement.
ii. Primary consolidation settlement.
iii. Secondary compression settlement.

Immediate settlement is also known as the initial or undrained settlement, which takes place immediately upon the loading of the structure. For saturated soils, the deformation is at constant volume and is caused by the shear strains beneath the loaded area. There is very little drainage taking place for soft soils of low permeability. Under the centre line of the loaded area, the vertical compression is accompanied by the lateral expansion.

Primary consolidation settlement is the settlement due to the excess pore pressure induced by the applied load which causes water to drain from the soil while the stress increment is transferred to the soil skeleton. Consolidation settlement is a time dependent process and produces mainly volume change but shear deformations are also involved leading to further settlement.

Secondary settlement or drained creep takes place at constant effective stress after the complete dissipation of pore water pressures has taken place. In practical cases, secondary compression is assumed not to start until all the primary consolidation has been completed although this is not necessarily the case.

Vertical settlement or displacement in soils can be measured using six devices. The six devices are survey targets, surface settlement plates, deep settlement probes (screw plates or anchors), point surface and deep settlement gauges, multipoint settlement gauges and profile gauges, Leroueil et al (1990). Details are shown in Fig.2.24.

2.3.1.2.1. **Survey Targets**

Survey targets are easy to use provided the targets are available near the embankment.

2.3.1.2.2. **Surface Settlement Gauges**

This is the most common instrument used for measuring vertical displacements. The plates are placed on the surface of the fill. Readings are taken using surveying techniques periodically during embankment construction. This instrument is cheap, easy to use and gives reliable results. Its main disadvantages is that it impedes the movement of earth moving plants during construction. This instrument cannot be used once the road pavement is laid.

2.3.1.2.3. **Deep Settlement Probes (Screw Plates or Anchors)**

Deep settlement probes are linked to the ground surface by rods. The position of the probe is recorded by levelling the upper end of the rods. The deep settlement probes have the same advantages and disadvantages as the surface settlement plates but have the additional risk of errors if the tube which surrounds the measuring rod is pulled down by the settlement of the embankment and presses on the probe. The instrument is more appropriate for test sites and the useful life of the instrument is short because the measuring rod tends to stick inside the surrounding tube.

2.3.1.2.4. **Point Surface and Deep Settlement Gauges**

Point surface and deep settlement gauges are usually hydraulic whatever their shape (cylindrical or spherical). The instrument assumes the existence of a water column between the measurement probe and the soil surface. Point surface and deep settlement gauges have the same problems as hydraulic piezometers e.g. saturation of circuits, sensitivity to temperature variations and frost as well as the same solutions e.g. purging of the tubes with deaired water, installation of tubes in trenches to link probes to the measuring point and use of antifreeze.

The precision of measurement is improved if direct differential measurement between the gauge placed under the embankment and a fixed reference gauge placed at a similar depth outside the zone of influence of the embankment is used. Point settlement and deep settlement gauges are suited to standard site conditions since they create no obstacle for earth moving plants and their measurements are accurate enough provided the saturation of the tubes linking the bottoms of the gauges to the measurements panel is maintained.

2.3.1.2.5. **Multipoint Settlement Gauges (Extensometers)**

The multipoint settlement gauge or extensometer is a type of instrument for measuring the distribution of settlement throughout the thickness of the compressible layers. The instrument can be used on project sites although they can in someway obstruct the movement of construction plants.

There are two types of multipoint settlement gauge i.e. the telescopic tube settlement gauges and gauges with flexible tube. The existing system also differs in the way in which they detect (magnetically or by contact) the rings which follow the settlement of the soil at different levels. If correctly installed all multipoint settlement gauges give comparable results.

2.3.1.2.6. **Profile Gauges**

Profile gauges are used in order to give a continuous measurement of settlements of the surface of the soil under the embankment. Measurement of the profile gauge can be achieved by two methods, Leroueil et al (1990) :

i. Inclination of the probe is measured at each point of the tube with respect to the vertical by using a pendulum. The curved profile is reconstructed just as for an inclinometer.
ii. Differences in level at each point of the tube are determined between the probe and an external point. For this purpose, the tube is linked to the fixed point by continuous column of deaired water and the pressure of this water column is then measured either at the probe or at the fixed point. The deformed shape is then directly constructed point by point.

Profile gauges are usually used on sites involving embankment on soft clays.

2.3.1.3. **Lateral (Horizontal) Displacement**

Measurement of lateral or horizontal displacement is important to the understanding of the behaviour of structures on soft clay. Lateral displacement is measured using a type of instrument known as the inclinometer as shown in Fig.2.25. The inclinometer consists of a long flexible tube which is usually installed to the bedrock in a borehole. The tube may be square or round and possess longitudinal grooves so that it can guide the measuring tube. The faces of the groove are oriented to the desired direction during installation. Measurements are then taken at fixed intervals by using the inclinometer probe. Although measurements of lateral movements can be reasonably accurately taken in the field using the inclinometer probe, the progress with regard to prediction of lateral movements through theoretical analysis has not been quite so successful. The predicted values from analysis were found to vary substantially from field results due to the following reasons, Poulos (1972) :

i. Difficulty in estimating of the Poisson's ratio of the soil.
ii. Anisotropy of the soil.
iii. Non-linearity of the stress-strain behaviour of the soil.
iv. Non homogeneity of the soil.
v. The neglect of certain factors e.g. effect of embankment stiffness and foundation. roughness, incorrect assumptions of applied soil stresses.

Poulos (1972) found that the predicted values were more than measured values and that the discrepancy appears to be greater in stiff foundations.

Burland (1971) introduced a simple stress-strain theory to estimate the horizontal displacements beneath strip loads. Burland (1971) concluded that during consolidation, the horizontal displacement near the surface develop rapidly favouring the early formation of tension cracks which could influence short term stability.

Lateral displacements of 21 embankments were analysed by Tavenas et al in 1979. Tavenas et al (1979) confirmed the development of a significant consolidation at the beginning of any embankment construction. Due to this development lateral displacement initially develops conforming to the theory of elasticity for a drained condition. Tavenas et al (1979) gave the relationship between lateral displacement and vertical settlement for overconsolidated clay as :

$$y_m = 0.18s \qquad \text{(Eqn.2.15)}$$

where

y_m = maximum horizontal displacement

s = vertical surface settlement

Tavenas et al (1979) mentioned that undrained shear distortions can develop near the end of embankment construction when the clay has become partly or entirely normally consolidated. The sequence of drained and undrained response governs both the magnitude and distribution with depth of the lateral displacements causing the predictions of lateral displacements to be unsuccessful. Delayed lateral deformations can be significant and develop linearly with the consolidation settlement.

2.3.1.4. **Stability Analysis**

Stability analysis of a slope/embankment/foundation is another important aspect. Methods of stability analysis are usually based on total stress analysis or effective stress analysis. The total stress analysis (or ϕ_u=0 analysis) makes use of undrained shear strength parameters with a minimum factor of safety at the end of construction stage, while the effective stress analysis (c',ϕ' analysis) uses short or long term loading conditions and reasonably accurate determinations of pore pressure conditions. Total stress analysis method is often preferred over effective stress analysis because of the difficulty in obtaining reliable effective stress shear strength parameters and predicting accurately pore pressure changes during the consolidation process. Pore pressure ratio (r_u) have also been used to express average pore pressure conditions in an effective stress analysis but changes in pore pressure ratio can cause significant differences in estimated safe heights of the constructed embankments, Younger (1992). Other approaches that have been used in design of embankment stability are an empirical approach based on overall corrections of undrained shear strength, USALS (undrained shear strength at large strains), SHANSEP, a semi empirical method proposed by Trak et al (1980) and Pilot et al (1982). Ladd (1991) divided stability problems into three types based on drainage and loading conditions, the three types are :

i. Undrained (short term or end of construction) which uses total stress analysis.

ii. Drained (long term) which uses effective stress analysis.

iii. Partially drained (intermediate) which applies to stage construction.

For stage constructed embankments, total stress analysis is considered unsuitable as it is too conservative compared to effective stress analysis which is more representative of the true stability conditions, Tavenas et al (1978). Cinicioglu and Togrol (1990) proposed a new design procedure for stage constructed embankments using critical state theory taking account the plastic behaviour of clays. This procedure inherits the conditions that automatically satisfies the bearing capacity consideration and enables the designer to optimise the length of consolidation pause periods. The proposed procedure can be applied for both isotropically normally consolidated and anisotropically normally consolidated soft clays. Ladd (1991) suggested a new design approach to determine embankment stability called USA (undrained strength analysis) which treats the in-situ stresses as equal to the consolidation stresses in order to compute the available undrained strength within the foundation. The method assumes an undrained failure corresponding to the consolidated undrained case.

The control and monitoring of the stability of embankment during construction is usually done using pore pressure data, DeLory et al (1965) and Margason and Symons, (1969). For embankments constructed on organic soils, Ladd et al (1969), recommended that field settlement data be used instead of piezometer data as piezometers were not effective in such soils. However, field settlement data cannot be used for embankments having wide berms. For this case Ladd et al (1969) mentioned that slope indicators and alignment stakes are more useful.

2.3.2. Theoretical Analyses

Prediction of field behaviour can be made using various theoretical analyses. The theoretical analyses of the consolidation of soft soils can be divided into three types.

These are one, two and three dimensional consolidation analysis. Comparisons are made between these three consolidation theories.

2.3.2.1. **One Dimensional Consolidation Analysis**

The prediction of various field behaviours using one dimensional consolidation analysis will be dealt with briefly although it is not directly related to the present study undertaken. The one dimensional analysis theory was first introduced by Terzaghi (1923) (cited by Balasubramaniam and Brenner (1981)). In Terzaghi's one dimensional consolidation analysis, the following assumptions was made :

i. The soil was completely saturated with water.
ii. The soil particles and the pore water are incompressible.
iii. Darcy's law is valid.
iv. The strains of the soil skeleton are controlled exclusively via a linear time-independent relation.
v. The soil is homogeneous.
vi. The strains, velocities and stress increments are small and the theory is quasi-elastic.

Although most of Terzaghi's assumptions were unrealistic or do not satisfy practical applications, it is being widely used because of its simplicity and also because most of the uncertainties in the results were mainly due to ill defined stratigraphy, soil properties and loading conditions rather than the simplification in the analysis. Deviations from Terzaghi's theory can be seen from results obtained from laboratory testing, Balasubramaniam and Brenner (1981) :

i. Settlements do not approach an ultimate value as they exhibit the phenomenon of secondary compression.
ii. The ratio of secondary to primary settlement increases with decreasing load incremental ratio and also with increasing maximum drainage distance.

iii. The rate of consolidation of mid-plane pore pressure increases with a reduction in load increment ratio during the early stages of the consolidation process.
iv. Prolonged load increment duration causes an increase in the ratio of secondary to primary settlement and initially rapid dissipation of mid plane pore pressure in the succeeding load increment.

Due to too many deviations from Terzaghi's 1D consolidation analysis theory, several researchers have modified Terzaghi's 1D theory to try and make it more realistic. The modification to Terzaghi's theory can be divided into seven categories. They are time-dependent loading, non linear stress-strain relationships, variable soil parameters, finite strains, submergence correction, time-dependent behaviour and the rheological model. A full review of one dimensional consolidation analysis together with the modifications that are made was carried out by Balasubramaniam and Brenner (1981).

2.3.2.2. **Two Dimensional Consolidation Analysis**

As Terzaghi's 1D consolidation theory normally underestimates the rate of settlement in field situations which in most cases is attributed to horizontal dissipation of pore pressures, more complex consolidation theory were introduced in order to try to simulate the real behaviour of the soil in-situ. The two dimensional consolidation theory is basically an extension of Terzaghi's 1D diffusion type theory proposed by Rendulic in 1936, Balasubramaniam and Brenner (1981), and is also known as the Terzaghi-Rendulic pseudo-consolidation theory or diffusion theory. The theory assumes that the total stress remains constant during consolidation i.e. the internal volumetric components of total stress are assumed to have the same time history of behaviour as the applied volumetric stress components, Schiffman et al (1969). During consolidation, the pore pressure dissipation rate vary depending on the distance from the drainage boundaries and differential strains that occur due to the change in Young's modulus and Poisson ratio form undrained to drained values. This will then cause an adjustment of the total stress to satisfy the stress-strain compatibility. The pseudo theory does not provide for such a coupling between the magnitude and the progress of settlement, it assumes that only the dissipation of the excess pore pressure contributes to the progress of settlement, Balasubramaniam and Brenner (1981).

Terzaghi-Rendulic theory has been used for solutions of various problems in 3D flow of water with 1D vertical strains e.g. Barron's sand drain theory in 1948 (cited by Balasubramaniam and Brenner (1981)). Davis and Poulos (1972) have obtained a series of curves for the average degree of pore pressure dissipation using finite difference techniques.

Two dimensional pseudo theory has also been used in the design of road embankments, Dunn and Razouki (1974). Their analysis included the influence of embankment geometry, layer thickness and anisotropy of the soil and their results were reviewed and presented by Murray (1978) as shown in Fig.2.26. Fig.2.26 illustrates the influence of drainage conditions (permeable or impermeable base), embankment shape (ratio of width of the top and bottom of the embankment) and layer thickness on two degrees of consolidation of 50% and 90%. It is also observed from Fig.2.26 that at 90% degree of consolidation, the shape of the embankment does not seem to have a great effect on the consolidation of the soil beneath it and therefore can be ignored.

2.3.2.3. **Three Dimensional Consolidation Analysis**

The three dimensional analysis was introduced by Biot (1941) and is known as Biot's theory, Schiffman et al (1969). It is derived directly from the theory of elasticity and is mathematically much more complex than the pseudo theory. The theory also provides for a coupling between the magnitude and progress of displacement and at any point, there is a continuous interaction between the dissipating excess pore pressure and changing total stress. The original theory by Biot assumed a homogeneous, isotropic, fully saturated medium but this has been refined repeatedly to account for compressible fluids. However until today, the theory has been limited to research problems in its application but with the introduction of more powerful numerical techniques, its application is expected to increase.

Solutions to the Biot's theory in closed analytical form have been limited because of the complex nature of the problem. The situations in which the theory have been used are, Davis and Poulos (1972) :

i. Uniformly loaded circle on a semi infinite mass, De Josselin de Jong (1957) and Schiffman and Fungaroli (1965).
ii. Uniformly loaded rectangle on a semi infinite mass, Gibson and McNamee (1957).
iii. Uniformly loaded strip on a semi infinite mass, Schiffman et al (1969).
iv. Uniformly loaded strip and circle on a finite layer, Yamaguchi and Murakami (1976).
v. Sphere subjected to hydrostatic pressure, Cryer (1963).
vi. Concentrated and circular loading of a deep soil mass overlain by a permeable layer, Mandel (1957).

The most common solutions now are by means of finite element formulation of Biot's equations, Sandhu and Wilson (1969) and Hwang et al (1971). Design charts cannot be established when the geometry of the problem loses its simplicity. Biot's theory have also been used in predicting the observed behaviour of embankments, Smith and Hobbs (1976) and Shoji and Matsumoto (1976).

An interesting feature of Biot's theory is the Mandel-Cryer effect or stress transfer effect, Mandel (1957) and Cryer (1963). This effect indicates that the pore pressure in the consolidating layer can increase some time after load application and may reach values which are higher than the applied pressure. Aboshi (1955) verified the Mandel-Cryer phenomenon with laboratory experiments (cited by Balasubramaniam and Brenner (1981)) but it is difficult to observe in the field because of various factors affecting the accuracy of piezometer readings. The Mandel-Cryer effect is best explained by viewing an element of soil which is draining, as one which is squeezing the adjacent element thus producing an increase in total volumetric stress. The pressure starts to increase near the drainage boundary and is then transferred to the interior of the soil layer. The magnitude of the Mandel-Cryer effect increases with depth because of the growing zone of influence, Schiffman et al (1969). If the surface is impervious, the effect is almost completely absent. The magnitude of the Mandel-Cryer effect also depends on Poisson's ratio, it is reduced when Poisson's ratio increases. The Mandel-Cryer effect does not show up when diffusion theory is used.

Biot's theory has also been applied to the plane strain consolidation problem of a flexible strip footing overlying a clay layer with an impermeable base, Yamaguchi and Murakami (1976), and to multi dimensional consolidation theory based on finite element analysis, Christian et al (1972).

2.3.2.4. **Comparison between 1D, 2D and 3D Consolidation Theory**

The differences between the 1D, 2D Pseudo theory and Biot's theory are as indicated in Table 2.14 :

i. The degree of consolidation U_s, where the time factor is defined in terms of one dimensional coefficient of consolidation c_1.

ii. Pore pressures, where the time factor is expressed in terms of c_2 (plane strain conditions) or c_3 (triaxial conditions), Davis and Poulos (1972).

iii. Effect of Poisson ratio on degree of settlement, which is apparent when using Biot's theory, for 2D diffusion theory, the values calculated lying between values of Poisson ratio of 0 and 0.45 calculated for Biot's theory. The differences between 2D and 3D are not great for the ratio of h/b=1, Davis and Poulos (1972).

CHAPTER THREE

REVIEW OF EXISTING DATA ON PENINSULAR MALAYSIA COASTAL SOFT SOIL DEPOSITS

3.0. Introduction

Peninsular Malaysia lies close to the Equator between latitudes 1°20'N to 6°40'N and longitudes 99°35'E to 104°20'E and has an area of 131,794 square kilometres. The soils formed in Peninsular Malaysia may be termed "Tropical Soils" as they have been produced principally by the process of tropical weathering. This chapter reviews and summarises the existing data on Peninsular Malaysia coastal soft soil deposits. It is presented in two main sections covering their origin, formation and distribution and their engineering properties.

3.1. Origin, Formation and Distribution of Peninsular Malaysia Coastal Soft Soil Deposits

3.1.1. Origin and Formation of Peninsular Malaysia Coastal Soft Soil Deposits

3.1.1.1. Source of Materials

Peninsular Malaysia coastal soft soil deposits are part of the Tropical Weathered Transported and Redeposited Materials soil group classified by McGown and Cook (1994). They are formed from the chemical weathering of parent rocks which are then transported by rivers and redeposited in a variety of environments from freshwater to marine conditions. The geomorphology of Peninsular Malaysia consists of a central range of folded mountains with very often short steep valleys leading to the coastal plains, McGown and Cook (1994). As a result of the geomorphology, most of the Tropical Weathered Transported and Redeposited Materials are found in the bottom of steep valleys along the coastal plain or in the sea. The lengths of the main rivers of Peninsular Malaysia with the gradient of the slopes from the source to the coastal plain are shown in Table 3.1. Due to the relatively short lengths of the rivers, the coastal soft soil deposits consist of materials which have very similar chemical composition to the Tropical Weathered In-Situ Materials. However, the Tropical Weathered and Transported and Redeposited Materials are heterogeneous in nature, deriving from a variety of different parent bedrocks which have been mixed with other materials such as organic debris during their transportation down rivers in flood during the monsoon seasons, McGown and Cook (1994).

3.1.1.2. **Tectonic Movements**

The coastal soft soil deposits of Peninsular Malaysia may have been deposited in deltaic, fluvial or tidal marsh environments with the characteristics appropriate to these depositional environments as discussed in Chapter 2. The main factor controlling the depositional environment in Peninsular Malaysia at any location was the relative land and sea levels. This changed in Peninsular Malaysia during the late Quaternary due to changes in the sea level associated with the retreat of the glaciers of the last ice age in the Northern and Southern hemispheres.

The effects due to tectonic movement may be ignored as Peninsular Malaysia lies in a region known as the Sunda Land which has been stable in recent geological time. The Sunda Land consists of the Sunda Shelf, Peninsular Malaysia, Bangka, Belitung, Kalimantan and Western Sarawak. These areas have been tectonically stable since the early Tertiary, Tjia (1977; 1980). Most of the tectonic movements occurred during the Cenozoic age. These movements caused downwarping of several metres and have affected several broad marginal areas of the northern shelf and the Java sea. The basins were then in-filled during the Early Tertiary which resulted in a generally undulating surface from which the land areas rose.

The main effect on the formation and depositional environment of Peninsular Malaysia coastal soft soil deposits was thus sea level changes relative to the stable land mass.

3.1.1.3. **Previous Investigations Relating to Sea Level Changes**

Keller and Richards (1967) (cited by Haile (1971) and Streif (1979)) dated peat found in a sediment core in the Malacca Straits, the soil consisting of stiff silty clay and shelly sand, to be around 10200 years to 9800 years B.P. They concluded from this that the Pleistocene/Holocene boundary, which was assumed to be at 10000 years B.P., was at about 26.5m below present sea level although the types of depositional environment relating to the samples investigated were not identified.

Biswas (1973) suggested that the sea level in Peninsular Malaysia was about 65m below present sea level at about 11200 years B.P. Streif (1979) has described how Biswas (1973) came to that conclusion on the basis of data obtained from a peat sample found in bottom sediments. Again it must be pointed out that the type of depositional environment relating to this material was not described.

Tjia et al (1975) investigated twelve radiocarbon dates obtained from samples at various locations in Peninsular Malaysia. The samples include one beachrock made up of bioclasts and pebbles cemented by calcium carbonate, eight oysters, one coral, one marine shell and one peat sample. Tjia et al (1975) did not identify the depositional environments of these materials, however they concluded from the carbon dated samples that the sea level rose some three times above present sea level between 6000 years to 2000 years B.P. In detail, they suggest that the sea level appeared to rise 2m to 3m above present level about 5400 years to 5100 years B.P, 0.7m to 1.5m above the present level about 4200 years to 3660 years B.P. and 1.4m to 2.4m above present sea level about 2900 years to 2550 years B.P.

Geyh et al (1979) suggested that from 36000 years to 10000 years B.P. continental conditions prevailed in most parts of the southern Malacca Strait and that the sea level was between 60m to 40m below present sea level. They also suggested that the sea level then rose from 13m below present sea level to 5m above present sea level from 8000 years to 4000 years B.P.. The sea level then dropped back to the present sea level. Geyh et al (1979) came to this conclusion on the basis of the results of carbon dating thirty three samples of organic material including peat, roots and soil samples. Of the thirty three samples they carbon dated, seven samples consisted of peat or in-situ roots from which Geyh et al (1979) concluded that the sea level was below the sampled level during the time of the formation of the sample. Another eight samples consisted of peat formed on top of terrestrial deposits from which Geyh et al (1979) concluded that the sea invaded the area shortly after the formation of the material.

Streif (1979) described the results from twenty five carbon dated samples obtained between Port Dickson and Pontian Kecil along the Malacca Straits of Peninsular Malaysia. The materials tested were obtained from nine peat, twelve mangrove and four organic cover sequences samples. Streif (1979) assumed that the peat samples were formed on top of terrestrial deposits and laid under freshwater conditions but were then overlain by brackish and marine sediments, indicating that the sea invaded the area after the formation of the peat. From the results obtained, Streif (1979) concluded that the sea level was approximately 15m below present sea level around 8000 years B.P. and then rose to about 5m above present sea level at about 4500 years ago and then subsided to the present sea level.

Streif (1979) also identified two Pre-Holocene terrestrial deposits. These samples consisted of wood in colluvial deposits. The surrounding depositional environment in which the colluvium and wood was laid down is not known. Streif (1979) concluded that the sea level was more than 6.8m below present sea level about 27000 years to 13000 years B.P. Streif (1979) also mentioned that the oldest Holocene sea level indicator found from mangrove deposits was around 8000 years B.P.

The Malaysian Highway Authority (1989) described that a dark brown peat layer of 0.2m thickness obtained from a borehole at the site of the Muar trial embankments have been carbon dated at about 10000 years B.P. The Malaysian Highway Authority (1989) concluded that the Pleistocene/Holocene boundary was about 15m below the present sea level, the depth at which the peat layer was found, although the type of depositional environment in which the peat layer was formed was not identified.

3.1.1.4. Discussion on Previous Sea Level Changes Data

The Pleistocene/Holocene boundary generally assumed to be around 10000 years B.P. has been mentioned by several Malaysian geological and engineering researchers and related to various depths below present sea level as described above. Unfortunately, in most cases the type of depositional environment from which, or around which, the material used for carbon dating was sampled, has not been identified. The depths that were used to differentiate the Pleistocene/Holocene boundary could thus be misleading as the "reference" materials could have been deposited, or surrounded by a variety of environments from freshwater to marine.

Evidence gathered from all the radiocarbon dated samples is plotted together with data from other worldwide sea level observations for the period 12000 years B.P until present, as shown in Figs 3.1 and 3.2. Most of carbon dated samples obtained from Peninsular Malaysia can be seen to lie above or on the boundary of other sea level change observations. The sample from Muar, in particular, lies well above the boundary of other data. This almost certainly indicates that the sample from Muar could not have been deposited in a marine environment and that some of the other samples are of doubtful origin.

3.1.1.5. Effects of Transgression and Regression

Figure 3.3 shows a schematic representation of the problem of establishing an age with depth relationships for the three types of environment (freshwater, brackish and marine). The boundaries between the different environments are shown to cross-over. Materials that are deposited in any one of the three environments can be mixed with another by the processes of transgression and regression due to the rise and fall in sea level. Relating back to the review of coastal processes in Chapter 2, these processes of transgression and regression may be described as follows :

(a) **Transgression**

This is due to a rise in sea level and can result in the migration of the shoreline in a landward direction. Transgression can lead to a swampy, organic, brackish water environment which can be indicated by the presence of organic layers. Further transgression of the sea would then lead to the development of an inter-tidal or estuarine (brackish water) environment over the previously swampy area. In this area, the deposition of silty clay layers interbedded with sand lenses would occur with the possible presence of desiccated crust as a result of intertidal fluctuations. Continued marine transgression would then lead to the swampy area reaching its maximum extent in-land and the estuarine area being fully developed with marine conditions prevailing seaward of this, McGown and Cook (1994).

(b) **Regression**

This is due to a fall in the sea level and can result in the exposure of the surface sediments causing erosion of some areas of the upper sediments or deposition of freshwater sediments from rivers and land surface run-off over other areas. The exposure of the sediments can lead to the formation, or at least the partial development of, a tropically weathered profile which might be overlain by a subsequent transgression, McGown and Cook (1994).

An useful classification of various stages of transgression and regression is shown in Fig.3.4, Curray (1964).

3.1.1.6. Overall Considerations on the Origin and Formation of Peninsular Malaysia Coastal Soft Soil Deposits

Based on the points made above, it may be suggested that the coastal soft soil deposits of Peninsular Malaysia have been formed from the breakdown of various tropical weathered parent rocks. The weathering products have been transported by short steep rivers, often in flood, to the coastal plains and re-deposited in a variety of environments ranging from freshwater to marine. Due to this the materials formed are heterogeneous in nature.

The main factor affecting the depositional environment in which the coastal soft soil deposits in Peninsular Malaysia were laid was the change in sea level during the Pleistocene and Holocene periods. The effect of tectonic movements can be ignored as Peninsular Malaysia lies in a tectonically stable region known as the Sunda Land and that almost all of the tectonic movements have occurred during the much earlier Cenozoic era.

It has been shown that it is very important that the depositional environment of materials tested to determine the sea level at different times be clearly established, since the age of materials of unknown depositional environment can provide misleading or at least confusing data, Figs.3.1 and 3.2. A complicating factor is the process of transgression and regression which can lead to two materials being deposited in different environments at the same elevation although found in close proximity to each other.

3.1.2. Upper and Lower Units within the Holocene Coastal Soft Soil Deposits

3.1.2.1. Previous Investigations Relating to Upper and Lower Units

A related aspect of the previous studies on the coastal soft soil deposits is concerned with the possible formation of an upper and a lower unit within the Holocene coastal soft soil deposits. Tjia et al (1975) (cited by the Malaysian Highway Authority (1989)) suggested the formation of an upper and lower units within the Holocene coastal soft soil deposits. They suggested that the lower unit could have been deposited from the Holocene/Pleistocene boundary period of between 10000 years B.P. to about 5000 years B.P.. They also suggested that the layer separating the upper and lower units was deposited about 4000 years B.P. when the sea level dropped about 1m below present sea level and that the upper unit was then deposited when the sea rose again at about 4000 years B.P. to 3000 years B.P.

Raj (1990) carried out a detailed geological study on continuous samples obtained from two boreholes from the Sungei Muar flood plain. He also concluded that the Holocene coastal soft soil deposits could be divided into upper and lower units. Raj (1990) suggested that the upper unit consisted of a greenish grey silty clay with fine sand partings while the lower unit consisted of entirely greenish silty clay. Shells and shell fragments of marine organisms, of unknown age, were found in the upper unit with a small thin layer of shell fragments of about 0.15m thickness separating the upper and the lower units. Raj (1992) suggested that the deposition of the lower unit could have occurred during the rise in sea level until about 5500 years to 4000 years B.P. which he associated with a mean sea level of 3m to 4m above present sea level. He then suggested that the sea level dropped gradually at about 4000 years to 3000 years B.P. and that this was associated with the erosion, exposure and weathering of the upper part of the lower unit. Further he suggested that sea level rose about 3000 years B.P. to about 2m above the present sea level. This second high sea level he stated is marked by raised beach ridges, whilst the marine transgression is marked by the upper unit which was deposited under subtidal conditions. The upper unit he states also sometimes overlies beach sands and thin peaty soft soil layers. Raj (1992) finally suggested that the sea level then dropped gradually to the present sea level from about 2500 years and this is mainly marked by the erosion and weathering of the upper part of the upper unit.

Geotechnical evidence to prove the existence of these upper and lower units within the Holocene coastal soft soil deposits is difficult to obtain, although some changes in geotechnical properties have been observed at several sites, Kobayashi et al (1990) and Aziz (1993). The suggested depths at which these changes occurs vary from site to site, Table 3.2.

Kobayashi et al (1990) found no clear distinction between the upper and lower units in Port Klang and Malacca. However in Singapore, they did find an intermediate stiff layer of 2m to 5m in thickness at about 15m below sea level separating the upper and lower units. No mention of the ages of these upper and lower units was given nor was it stated whether they were of the Pleistocene or Holocene period. Kobayashi et al (1990) gave the thickness of the upper coastal soft soil deposits of several sites in Peninsular Malaysia to be between 10m to 16m.

Dobie and Wong (1990) and Wong and Dobie (1990) analysed data obtained from Piezocone and Marchetti Dilatometer tests in three sites, viz. Sungei Dua, Alor Pongsu and Juru. They gave values for an upper and lower unit based on the existence of a buried crust found between these two units. No geological evidence was given to support these divisions of the upper and lower unit.

Aziz (1993) studied eight sites in the west coast of Peninsular Malaysia. He divided the Malaysian coastal soft soil deposits into an upper and a lower unit according to presence of shells and peat layers found from the boreholes at the sites studied, but the age and the type of depositional environment of the materials found were not identified. Aziz (1993) mentioned that the thickness of both the upper and lower units was between 6m to 12m. Re-examination of the geotechnical results obtained by Aziz (1993) does not fully support the division of the coastal soft soil deposits into two units. Much more evidence is required before this can be justified.

Mohamad et al (1994) also described upper and lower units within the coastal soft soil deposits of Peninsular Malaysia. They followed the classification suggested made by Castlebury and Prebaharan (1982) (cited by Mohamad et al, 1994) who suggested that the upper and lower units of the Holocene soft soil deposits are divided by a weathered crust. Mohamad et al (1994) did not mention the depths of the upper and lower units, although they gave some geotechnical properties of some sites which they stated were the properties of the upper unit.

The relationship between engineering characteristics and engineering geology have been studied by Batchelor and Ting (1980). They carried out a study on an airport project in Bayan Lepas, Penang. Batchelor and Ting (1980) divided the subsoils into three main geological units with each unit having their own unique engineering characteristics. They believed that their correlation can be applied to other sites in West Malaysia although this has yet to be proven.

3.1.2.2. Overall Consideration of the Existence of Upper and Lower Units within the Holocene Coastal Soft Soil Deposits

The existence of an upper and a lower unit within the Holocene coastal soft deposits as hypothesised by Tjia et al (1975) and Raj (1990; 1992) has been based on rises and falls in sea level, however, these need to be verified in more detail and need to tied up with other data on sea level changes as proposed by other researchers, see Figs. 3.1 and 3.2. Further the division of the Holocene coastal soft soil deposits into an upper and lower unit by Aziz (1993) based on peat and shell layers may be misleading as the peat layers could have been formed from highly organic marshes and swamps which may have been deposited in environments ranging from freshwater to marine. The other difficulty is to distinguish the geotechnical properties of the upper and lower units due to the inherent heterogeneity of the soft soil deposits, any likely differences between the possible upper and lower units being not significantly different from the differences widely found at any level in the deposits.

3.1.3. The Distribution of Peninsular Malaysia Coastal Soft Soil Deposits

Cox (1970a) mentioned that the lateral extent of deltaic soft soil deposits in Peninsular Malaysia was around 25km and the depth of marine clay was up to 20m.

Gobbett and Hutchinson (1973) suggested that the coastal plain is at its widest at the Sungei Perak river mouth being about 45km wide while the average width of the coastal plains is between 20km to 30km wide.

At the beginning of the 1980's the Geological Survey of Malaysia embarked on a detailed mapping programme of the distribution of Malaysia coastal soft soil deposits, Bosch (1988). The Geological Survey of Malaysia used a lithostratigraphical classification for the coastal soft soil deposits based on two main elements, viz. the sedimentary environment in which the deposits were formed and the age of the sediments. The Geological Survey of Malaysia also subdivided the coastal soft soil deposits into members as shown in Table 3.3, Bosch (1988). On this basis, they suggested that the northern coastal plains of west Peninsular Malaysia (Perlis, Kedah, Penang and Perak) are about 70km to 100km in length and 15km to 22km in width while the southern coastal plains (Johore) are about 150km in length and 7km to 25km in width. For the east coast of Peninsular Malaysia, the coastal plains vary from 20 to 50km in length and about 10 to 20km in width. The distribution and locations of the coastal soft soil deposits in Peninsular Malaysia are as shown in Fig.3.5.

Raj and Singh (1990) mentioned that the coastal plains are well developed in the northwestern and southwestern sections of Peninsular Malaysia where they are about 20km in width and about 70km in length.

With regard to the depths of the coastal soft soil deposits, Ting and Ooi (1977) stated that the depth of coastal soft soil deposits in Peninsular Malaysia varies between 9m to 14m. This statement was based on their studies of four sites in Peninsular Malaysia (Port Klang, Prai, Penang and Muar).

Abdullah and Chandra (1987) showed a typical profile of coastal soft soil deposits having depths of 20m to 22m overlying a medium stiff clay of 10m, Fig.3.6. Aziz (1993) mentioned depths of 10m to 22m in his study of eight sites on the west coast.

Kobayashi et al (1990) gave the depths of the coastal soft soil deposits to be between 10m to 16m. The depths of various sites in Peninsular Malaysia obtained from various researchers are summarised in Table 3.2. These depths are however localised depths at each site and are not related to the mean sea level.

3.2. Engineering Properties of Peninsular Malaysia Coastal Soil Deposits

The engineering properties of Peninsular Malaysia coastal soft soil deposits can be obtained from both field and laboratory testing. The data of field and laboratory testing from various researchers are shown in Tables 3.4 to 3.9. These data were obtained from the following sites :

i. Port Klang, Ting and Chan (1971).
ii. Prai, Penang, Port Klang and Muar, Ting and Ooi (1977).
iii. Kuantan, Abdullah and Chandra (1987).
iv. Muar, Malaysian Highway Authority (1989).
v. Prai, Port Klang, Malacca, Muar and Kemaman, Kobayashi et al (1990).
vi. Johor Bharu, Simpang Renggam, Machap, Bukit Raja, Alor Pongsu and Juru, Nicholls and Ho (1990).
vii. Kuala Perlis, Alor Setar, Juru, Klang, Sabak Bernam, Bagan Datoh, Pontian and Muar, Aziz (1993).
viii. Penang, Klang, Tangkak and West Johor, Mohamad et al (1994).

Data obtained from the present study were also included in Tables 3.4 to 3.8 and in all the Figures but these will be not be presented and discussed until later in Chapter 5.

3.2.1. Engineering Properties Obtained from Field Testing

3.2.1.1. Undrained Shear Strength

Table 3.4 shows a summary of the undrained shear strength parameters obtained by various researchers. Data on the undisturbed undrained shear strength of Peninsular Malaysia coastal soft soil deposits range from 8kPa to 60kPa along the west coast and from 12kPa to 29kPa for the east coast. The remoulded undrained shear strength ranges from 1kPa to 20kPa for the west coast with no data available for the east coast. The average undrained shear strength (undisturbed and remoulded) was found to generally increase with depth, Fig.3.7, with the existence of a locally higher shear strengths near to the top of the deposit. This can be attributed to the presence of a weathered crust at or near the top of the deposit. No clear relationships were evidenced between undrained shear strength and moisture content, Fig.3.8. The undrained shear strength remains about constant with organic content although the range of values is quite large, Fig.3.9.

Aziz (1993) who reviewed Ting and Ooi's data suggested that undrained shear strength increased slightly with increase in salt content while there is no change in the remoulded strength as shown in Fig.3.10, however, this does not appear to be very evident from the data.

Sensitivity values with depth range from 1.5 to 16 with the vast majority less than 6, Fig.3.11. Sensitivity with depth again shows that the average sensitivity is about 3 and remains about constant with depth. These deposits thus have low to medium sensitivity, see Table 2.4.

No clear relationships can be observed between the above sensitivity values and published liquidity index or salt concentration data, as shown in Figs. 3.12 and 3.13. When compared to Norwegian Quick clays again shown in Figs. 3.12 and 3.13, it can be observed that the sensitivity of the coastal soft soil deposits remains more or less within a defined range (low to medium sensitivity) even though liquidity index increases or salt concentration decreases. This is in contrast to the behaviour of Norwegian quick clays whose sensitivity increases as liquidity index increases or as salt content decreases due to the effect of leaching. It can therefore be concluded that the properties of Peninsular Malaysia coastal soft soil deposits are in general quite different from those of Norwegian quick clays and are not significantly affected by a leaching process.

3.2.1.2. **Piezocone Data**

Dobie and Wong (1990) interpreted piezocone test data from three sites (Sungei Dua, Juru and Alor Pongsu) located along the North-South expressway. The relationship between cone resistance with friction ratio as suggested by Robertson and Campanella (1983), Fig.3.14, shows that the coastal soft soil deposits of Peninsular Malaysia can be classified as clayey silts and silty clays or clays. Dobie and Wong (1990) also established relationships between the corrected cone resistance (q_T-σ_v) and undrained vane shear strength (S_{uv}) for the three sites, Fig.3.15, and found that the cone factor values (N_{KT}) ranges between 11 and 15 for the "upper" unit and between 16 and 18 for the "lower" unit of Peninsular Malaysia coastal soft soil deposits. Dobie and Wong (1990) based their division of an upper and lower unit on the existence of a buried crust found between them, however, no geological evidence were given to support this. Comparisons of N_{KT} values were also made with Singapore soft soil deposits and a similarity in N_{KT} values was found to exist. Relationship between piezocone parameters A_q (normalised cone resistance) and B_q (water pressure ratio) with overconsolidation ratio (OCR) were also established but more data will be required in order to verify these relationships.

3.2.1.3. Marchetti Dilatometer Data

Another instrument that has been used to study the soft soil parameters of Peninsular Malaysia is the Marchetti dilatometer. Wong and Dobie (1990) analysed results obtained using the Marchetti dilatometer from the same three sites where the piezocone tests were carried out. They found that coastal soft soil deposits at the three sites can be classified as clay to clay/silt from the soil classification charts, Fig.3.16. Plots of overconsolidation ratio with K_D, Fig.3.17 show that most of the data obtained from the three sites using the Marchetti dilatometer fall in between the line proposed by Marchetti (1980) and Lacasse and Lunne (1988).

Further data obtained from Sungei Besar and other sites in Peninsular Malaysia (Wong et al, 1993) show that both the Marchetti (1980) and Lacasse and Lunne (1988) line generally overestimate the OCR values, Fig.3.18. This indicates that both these lines may not be suitable for the estimation of OCR values of the Peninsular Malaysia coastal soft soil deposits.

3.2.1.4. Self Boring Pressuremeter Data

Recently, the Self Boring Pressuremeter or Camkometer has been used in the study of coastal soft soil deposits in Peninsular Malaysia. Wong et al (1993) used the Camkometer at Sungei Besar, Selangor. The data obtained by the Camkometer were compared with data obtained by the Marchetti dilatometer, field vane and the unconsolidated undrained triaxial tests. Generally the results obtained with depth from all these tests, viz. K_o, undrained shear strength, total horizontal stress, Fig. 3.19, gave almost similar trends except the shear modulus with depth where the results of the Camkometer seems to be higher than for other tests. This needs to be further checked as more data become available.

As mentioned in Section 3.1.2, there are difficulties in differentiating the so called "upper" and "lower" units of Peninsular Malaysia coastal soft soil deposits geotechnically, therefore both relationships obtained by Piezocone and Marchetti Dilatometer need to be reanalysed and further verified when more data become available. Other field tests like Dutch Cone Penetration and Standard Penetration tests have been carried out on the coastal soft soil deposits in Peninsular Malaysia but data obtained from these tests have not been made available for further analysis.

3.2.2. Engineering Properties Obtained from Laboratory Testing

Engineering properties that can be obtained from laboratory testing include classification properties (e.g. Atterberg limits, particle size distribution, specific gravity, bulk density etc), compressibility characteristics, strength properties as well as the chemical properties. A brief summary of the engineering properties identified to date is given below :

3.2.2.1. Classification Properties

Table 3.5 shows a summary of the classification properties obtained from various researchers and this can be summarised as follows :

The natural moisture content varies from 12% to 175% for the west coast and 21% to 107% for the east coast. Moisture content shows a slight increase with increase in organic content, although high moisture content is observed at low organic content, Fig.3.20. Plots of moisture content with salt content show moisture content to decrease slightly with increase in salt content compared to Norwegian quick clays where the moisture content seems to increase with increase in salt content, Fig.3.21. This once again indicates that the properties of Peninsular Malaysia coastal soft soil deposits are different from those of Norwegian quick clays. The values of moisture content show a very wide scatter and no clear relationship with depth, Fig.3.22. This could be attributed to the heterogeneity of these soft soil deposits.

Unit weight varies from 13kN/m^3 to 17.5kN/m^3 for the west coast and 14.5kN/m^3 to 17.1kN/m^3 for the east coast of Peninsular Malaysia. Variations of unit weight with depth are shown in Fig.3.23. This indicates that the unit weight remains relatively constant with depth and has an average value of about 14.5kN/m^3.

Specific gravity values of Peninsular Malaysia coastal soft soil deposits vary from 2.35 to 2.75 with an average of 2.65 for the west coast and 2.5 to 2.7 for the east coast.

The particle size distribution ranges from about 15% to 70% clay, 15% to 70% silt and 0% to 45% sand for the west coast and 31% to 56% clay, 16% to 49% silt and 6% to 28% sand for the east coast. Particle size distribution with depth, Fig.3.24, shows a wide range of scatter.

The liquid limit of Peninsular Malaysia coastal soft soil deposits ranges from 40% to 155% and the plastic limit from 10% to 70% for the west coast. No data are available for the east coast. The data obtained from various sites in Peninsular Malaysia generally plots above the Casagrande A-line, Fig.3.25. A proposed equation of 0.7(LL-6) is suggested for Peninsular Malaysia coastal soft soil deposits as also shown in Fig.3.25. The average liquid limit shows a slight decrease with depth but the plastic limit remains relatively constant with depth, Fig.3.26. Mohammad Nor and Yusouf (1990) investigated the effects of sample preparation on the Atterberg limits of Peninsular Malaysia coastal soft soil deposits. They carried out tests at natural moisture content, air dried at room temperature and oven dried at 90°C. Mohammad Nor and Yusouf (1990) observed that the samples tested at natural moisture content were found to have a higher liquid limit than samples that were air or oven dried, Fig.3.27. The Atterberg limits shows no clear relationship with both organic content and salt content, Figs.3.28 and 3.29. Fig.3.29 also shows that the Atterberg limits values of Peninsular Malaysia coastal soft soil deposits are much higher say than those of Norwegian quick clays to which they have sometimes been compared.

The plasticity index with clay fraction, Fig.3.30, shows that a wide range of activity of the soft soil deposits exists, ranging from inactive to active.

Liquidity index ranges from 0.1 to 2.1 for the west coast but no data are available for the east coast. The average liquidity index values generally decreases with depth, Fig.3.31. Cox (1970b) attributed the reduction in liquidity index with depth and distance from the coastline to the weathering and secondary consolidation of the soft soil sediments.

3.2.2.2. **Chemical Properties**

A summary of the chemical properties of Peninsular Malaysia coastal soft soil deposits is shown in Table 3.6 and are discussed as follows :

The organic content varies from 0.3% to 22.5% for the west coast and about 10% for the east coast. The organic content values shows a wide scatter but no clear relationship with depth, Fig.3.32.

The salinity of Peninsular Malaysia coastal soft soil deposits ranges from 0.01 gm/litre to about 40gm/litre. The depositional environment of the sites where samples were obtained is not known. These salinity values indicate that the depositional environment of the sites need not have been totally marine (32gm/litre) and probably ranged from brackish to marine. The elevation of the samples from which the test specimens were obtained are also not known. Relationships of various properties with salt concentration are shown in Figs.3.10, 3.13, 3.21 and 3.29. These show that the properties of Peninsular Malaysia coastal soft soil deposits are quite different from those of Norwegian quick clays and that the leaching process does not play an important part in determining the properties of Peninsular Malaysia coastal soft soil deposits.

pH values range from 3 to 9 for the west coast and 7.5 to 8.5 for the east coast indicating the Peninsular Malaysia coastal soft soil deposits have a wide range which remains relatively constant with depth, Fig.3.33.

The percentage of chloride content ranges from 0.01% to 4.5% for the west coast. No data was available for the east coast. Chloride ion content shows a wide range of scatter and no relationship with depth, Fig.3.34.

The total sulphate content for Peninsular Malaysia coastal soft soil deposits ranges from 0.03% to 1.7% for the west coast. No data was available for the east coast.

3.2.2.3. Mineralogy

Table 3.7 shows the main clay minerals found in Peninsular Malaysia coastal soft soil deposits. The montmorillonite content ranged from 0% to 60%, kaolinite from 0% to 80% and illite 2% to 50%. The variation of these main clay minerals with depth, Fig.3.35, shows a wide scatter with the average value remaining relatively constant with depth. Other elements that were found to be present in Peninsular Malaysia coastal soft soil deposits are silicon, aluminum, iron, magnesium, calcium, potassium, sodium, manganese and titanium, MHA (1989), silicon, aluminum, iron and magnesium minerals, Raj and Ho (1990).

3.2.2.4. Fabric Studies

3.2.2.4.1. Macro or Meso Fabric Studies

Macro and meso fabric studies are known to have been carried out on Peninsular Malaysia coastal soft soil deposits but these have not been fully presented nor analysed to date.

3.2.2.4.2. Microfabric Studies

Microfabric studies using the scanning electron microscope have been carried out by Raj and Ho (1990) on samples obtained along the route of the North-South expressway. The microfabric studies showed the presence of pyrite crystals and precipitated halite crystals which indicates that sodium and chloride ions are present in the pore water of the silty clays.

Aziz (1993) undertook some microfabric studies using the scanning electron microscope on samples obtained from several sites on the west coast of Peninsular Malaysia. The samples were first prepared using the air dried method prior to coating and scanning. Some typical microfabric features observed by Aziz (1993) include the presence of organic diatoms, pyrites and granular matrix. The organic diatoms were found to be present in all of the samples tested, Plate 3.1. Pyrites or iron sulphides (FeS_2) crystals, Plate 3.2, were found to be either scattered, accumulated in an organic body or occurred within the clay matrix. The presence of pyrites indicates that cementing action can occur between particles which can result in an increase in the strength and brittleness of the undisturbed coastal soft soil deposits. Oxidation of these pyrites can lead to the formation of sulphates which can affect concrete structures. Granular matrices which were observed in some samples consist of fine sand and silt sizes, Plate 3.3, and are prominent in the silt and fine sand lamination areas within the silty clay matrix. These laminations can act as drainage channels which can help to increase the horizontal permeability of the coastal soft soil deposits. Clusters of clay minerals and organic matter were also found to be present in the granular matrix. All samples studied were found to have a flocculated silty clay structure.

3.2.2.5. **Compressibility Characteristics**

Table 3.8 summarises the compressibility characteristics of Peninsular Malaysia coastal soft soil deposits obtained from various researchers and these characteristics can be summarised as follows :

The compression index (C_c) values for Peninsular Malaysia coastal soft soil deposits ranges from 0.35 to 3.2 on the west coast of Peninsular Malaysia while for the east coast, the values of C_c ranges from 0.02 to 1.2. Compression index (C_c) values showed a tendency to increase with organic content, void ratio, liquid limit and natural moisture content, Figs 3.36 to 3.39. The equation C_c=0.01(LL-8.9) suggested by Abdullah and Chandra (1987), Fig.3.36, seems to underestimate the compression index values. Compression index values show a wide range of scatter with the average value generally decreasing with depth, Fig.3.40.

The coefficient of secondary coefficient (C_α) values range from 0.001 to 0.03 in the west coast and 0.01 to 0.04 for Kemaman on the east coast of Peninsular Malaysia.

The coefficient of consolidation (c_v) for the west coast ranges from 0.2m^2/yr to 14.7m^2/yr. No data was available for the east coast. The coefficient of consolidation (c_v) at in-situ vertical stress shows a wide range of scatter and no clear relationship with depth, Fig.3.41.

The preconsolidation pressure (p_c) for the west coast of Peninsular Malaysia ranges from 20kPa to 180kPa. No data was available for the east coast. The average preconsolidation pressure (p_c) values is shown to generally increase with depth, Fig 3.42.

The overconsolidation ratio (OCR) of the west coast of Peninsular Malaysia ranges from 0.4 to 5 but no data was available for the east coast. The average overconsolidation ratios generally decrease with depth, Fig.3.43. Fig.3.43 also shows that the range of overconsolidation ratio (OCR) values seems to be higher at the top which then reduces and remains almost constant with depth. This can be attributed to the presence of a weathered crust at the top.

The initial void ratio (e_o) for Peninsular Malaysia coastal soft soil deposits on the west coast ranges from 0.6 to 4.1. No data was available for the east coast. Initial void ratio (e_o) is seen to decrease slightly with depth, Fig.3.44.

3.2.2.6. **Laboratory Shear Strength Properties**

Table 3.9 shows the summary of laboratory shear strength parameters of Peninsular Malaysia coastal soft soil deposits obtained from triaxial testing. For consolidated undrained tests, the values of consolidated undrained strength (C_{cu}) ranges from 2.3kPa to 17kPa for the west coast and 5.3kPa to 32.7kPa for the east coast. The angle of friction obtained from consolidated undrained tests (ϕ_{cu}) ranges from 1.5° to 6.1° (consolidated undrained triaxial tests) for the west coast and 3° to 14° for the east coast.

For effective stress parameters obtained from consolidated undrained tests, the effective cohesion (c') from various researchers in Malaysia ranges from 4kPa to 34.5kPa for the west coast and 5.3kPa to 12kPa for the east coast while the effective angle of friction (ϕ') ranges from 8° to 27° for the west coast and 24° to 38° for the east coast. The values of effective stress parameters are also questionable as they are either too large or too small. Typical values of c' and ϕ' for normally consolidated clays are given as 0kPa and around 19° to 34°, Das (1990). Balasubramaniam et al (1985) gave some typical values of c' and ϕ' for Bangkok clay in the ranges 0kPa to 15kPa and 16° to 30° and Singapore clay in the ranges 5kPa to 14kPa and 18° to 29°.

For unconsolidated undrained tests, the undrained shear strength (C_u) ranges from 17.9kPa to 25.5kPa for the east coast while not values were given for the west coast. The undrained angle of friction (ϕ_u) from unconsolidated undrained tests is given as

2.5° to 4.6° for the east coast while no values were given for the west coast. Typical value of the undrained angle of friction (ϕ_u) is usually zero.

For consolidated drained tests, the drained effective angle of cohesion (c_d) is given as 0kPa for both the west and east coast while for the drained effective angle of friction (ϕ_d) ranges from 18° to 24.5° for the west coast and 24° to 30° for the east coast.

Some of the shear strength parameters obtained by various researchers as shown in Table 3.9 are questionable as the values obtained are either too small or too large compared with typical values. This need to be further verified when more data become available. The errors in the values of the shear strength parameters obtained could be attributed to the following reasons :

i. The values obtained from the triaxial test might be wrong, for example effective stress parameters are indicated as total stress parameters.
ii. The samples used for the test are not properly saturated.
iii. The heterogeneity of the samples and the presence of sea shells and organic matter in the samples.

3.3. Conclusions

i. Peninsular Malaysia coastal soft soil deposits are formed from the breakdown of parent rocks by tropical weathering and are transported and deposited in a variety of environments from freshwater to marine conditions. The materials are brought down from the mountains by very short steep rivers, often in flood, with steep gradients, the longest river being the Pahang River about 420km long, with slope gradients of 1:15 to 1:300. Therefore the material formed is very heterogeneous and consists of a mixture of various parent rock materials together with organic material collected during the transportation of the material. Due to this heterogeneity, the properties of the materials vary at any depth and from site to site, depending on the local conditions of the site. These are shown in Tables 3.4 to 3.9 and in all the Figures in this Chapter.

ii. The formation of Peninsular Malaysia coastal soft soil deposits is greatly affected by the changes in sea level during the Pleistocene and Holocene period. The effect due to tectonic movement can be discounted as Peninsular Malaysia lies in a tectonically stable region known as the Sunda Land. The changes in sea level are associated with transgressions and regressions of the sea which has lead to the mixing of various materials which were deposited in different depositional environments. Therefore it is critical to establish the depositional environment of the materials prior to carbon dating them. Using the dating results obtained from materials of unknown depositional environment can be misleading.

iii. The heterogeneity of the Peninsular Malaysia coastal soft soil deposits can be clearly seen from the large scatter of various geotechnical properties shown in all of the Figures in this Chapter. The values of geotechnical parameters obtained from Peninsular Malaysia coastal soft soil deposits are comparable with typical values given for soft soils as shown in Table 3.10, Das (1990), except for the values of the compression indices which are much higher than the typical values.

CHAPTER FOUR

DETAILS OF THE TRIAL EMBANKMENT, INSTRUMENTATION WORKS, FIELD AND LABORATORY TESTING

4.0. Introduction

In 1992, two trial embankments were constructed at Kuala Perlis which is located in the State of Perlis, Northwest Peninsular Malaysia. The purpose of these embankments was to provide a "test bed site" for the study of the behaviour of Peninsular Malaysia coastal soft soil deposits. Details of the location, design and layout of the site, instrumentation works, field and laboratory testing carried out at the trial site are detailed in the following sections.

4.1. Details of The Trial Embankment

4.1.1. Location of the Trial Site

The trial site is located near the small town of Kuala Perlis in the State of Perlis, the northern-most state of Peninsular Malaysia. Perlis is also the smallest state in Malaysia with an area of 81,053 ha (200,200 acres). It is situated at a latitude 6° 15′ to 6° 40′ North and is surrounded by the Malacca Straits in the West, Thailand in the Northwest and Northeast and Kedah in the Southeast. The Perlis coastline is about 20km long, JICA (1984), and Kuala Perlis is situated at the northern tip. It is a fishing and commercial port and acts as a ferry terminal to the tourist island of Langkawi in the State of Kedah. The trial embankment site is situated about 2km from Kuala Perlis and about 15km from Kangar, the State Capital of Perlis, Fig. 4.1. The trial site is located nearby the Perlis coastal road and is situated about 15 metres from the coastline, Fig.4.2.

4.1.2. Design and Actual Layout of the Trial Embankments

Two trial embankments were built at Kuala Perlis namely the North and South trial embankments, Fig.4.3. The South trial embankment was constructed at a fast rate while the North trial embankment was constructed at a very slow rate so that comparisons could be made of the effect of different rates of construction on the trial embankment behaviour. At present, although both the trial embankments have been completed, the North trial embankment is still being monitored and collection of data is still being undertaken. This thesis will concentrate only on the South trial embankment.

For the South trial embankment, it was proposed during the preliminary design stage, that the South trial embankment would be constructed at a fast rate of construction of about 250mm per day with the final height of the embankment to be achieved within eight days. Due to construction problems on site, viz. problems in getting the fill material as well as mobilisation of the machinery needed for the construction of the embankment, the South trial embankment was completed in 36 days and the actual rate of filling works was about 110mm per day. The final embankment height achieved was 4m. The South trial embankment dimensions were ninety metres in length and sixty metres in width with a side slope angle of 1:1.5. A stage berm was also constructed at a height of 2m, Fig.4.4 to 4.6.

4.1.3. Geology of the Trial Embankment Site

The trial site lies on the coastal plain of Perlis, which consists mainly of flat alluvium at the coast and passing into colluvium inland. The coastal plain is just above sea level near the coast to about 6m in height at its inland extremity. Limestone and other hills are observed at a number of places which help to break up the flat coastal surface. Inland, the alluvium extends up valleys as narrow sinuous deposits. Near the coast mangrove swamps exist. The alluvial coastal plains of Kuala Perlis are of Holocene to Pleistocene age while the limestone hills are of the Setul Formation which is Ordovician to Lower Devonian age. Some of the alluvial coastal plains overly the Setul Formation as well as the Macincang Formation which consists of a thick quartzite sequence with subsidiary shale, shale, flagstone, subgreywacke, grit and conglomerate of the late Cambrian age. The details of the geology of the state of Perlis are given by Jones (1982).

4.2. Site Investigation and Instrumentation Works

4.2.1. Site Investigation Works

4.2.1.1. Preliminary Works

Prior to the main site investigation works and the installation of instrumentation in the south embankment, preliminary works were carried out. The site was first surveyed, Plate 4.1 starting from a permanent datum located at a bridge about 400m away. The trial site and the centre line of the south embankment were then established. The dimensions of the South embankment were measured and pegged from the centre line rather than from the edge of the existing road due to irregularities of the road edge. Spot levels of the original site were also taken.

Approach roads to the embankment were constructed to enable the drainage material (Batu Reput) to be dumped on the site. Polyfelt geotextile TS600 of dimensions 4m width by 175m lengths were then installed as a separator layer using a one metre overlap to join rolls, Plate 4.2. The geotextile was pegged with stakes, Plate 4.3, and alignment monitored to ensure that a straight line was achieved. Altogether ten rolls of geotextile were installed on the south embankment.

Drainage material (Batu Reput) were then dumped by lorries, Plate 4.4, on the geotextile and levelled using a backpusher, Plate 4.5. Altogether about three thousand seven hundred cubic metres of drainage material were used for the site. The site was then levelled to a finished thickness of 375mm, Plate 4.6 and the final reduced levels taken.

4.2.1.2. Boring and Sampling Works

Following the provision of the drainage layer, boring works were carried out at eleven locations as shown in Fig.4.7. The type of boring used was the wash boring technique where a water jet was used during the drilling of the boreholes. Sampling was carried out using piston thin walled samplers as they were regarded as the best samplers for these soft soils. Two sizes of sampling tubes were used, one was 86mm in diameter and about 0.8m in length while the other was 55mm in diameter and about 0.6m in length. Thus ISSMFE (1965) recommendation of length/diameter ratio for soft soils of 10 to 12 were adopted, Table 2.3.

Undisturbed samples were taken in each borehole at every 2m depth until a stiff layer was encountered at about 13m to 14m. Altogether at each borehole location some seven undisturbed samples were taken for laboratory testing. These samples were judged to be Class 1 samples according to BS5930 (1981). Besides the undisturbed samples, continuous sampling was carried out in a borehole near the centre of the trial embankment in order to obtain samples for macrofabric analysis. Again these samples were judged to be Class 1 according to BS5930 (1981).

4.2.2. Instrumentation Works

Instrumentation works were carried out at the same location as the boring and sampling works. The types of instrumentation installed in the South trial embankment are as shown in the detailed layout in Fig.4.8. Details of the installation techniques adopted for the various instruments were as described by Dunnicliff and Green (1988).

4.2.2.1. Pneumatic Piezometers

Pneumatic piezometers were installed in order to measure the pore pressures developed during and after the construction of the South trial embankment. The type of pneumatic piezometers used was the closed ended type and the piezometer tip was ceramic. Altogether sixteen pneumatic piezometers were installed at four locations. At each location, the pneumatic piezometers were installed at depths of 2m, 5m, 8m and 11m. The depths of the piezometers were decided after the soil profile was obtained from borehole investigations. Besides the sixteen pneumatic piezometers, four pneumatic piezometers were installed as reference piezometers some distance away from the trial embankment site. Prior to installation, the piezometers were saturated and checked for leakage, Plate 4.7. Readings of the piezometers were undertaken twice daily prior and during the construction of the embankment. At the end of construction of the embankment the frequency of the readings was gradually reduced to once daily. The readings of the piezometers were accomplished by introducing nitrogen gas into the piezometer tip until the pressure was stabilised, at which point the reading was taken by a readout unit, Plate 4.8.

4.2.2.2. Standpipe Piezometers

Standpipe piezometers were installed to measure the ground water level in the embankment during construction. They also acted as a backup for the pneumatic piezometers if they were damaged or clogged. The standpipe piezometers were also used for in-situ permeability tests. For the South trial embankment, three standpipe piezometers were installed at depths of 4m, 8m and 12m. Three reference standpipe piezometers were also installed at the same depths some distance from the site. The type of standpipe piezometer tip used was plastic, Plate 4.9. The readings of the standpipe piezometers were taken using a dipmeter, Plate 4.10. The readings of the water level were taken when a bleep sound was made by the probe when water was encountered. The frequency of readings of the standpipe piezometers was similar to that taken for the pneumatic piezometers.

4.2.2.3. Inclinometers/Extensometers

Inclinometer/extensometer systems, Fig.4.9 were installed to measure the lateral movement of the embankment as well as the vertical displacement occurring during and after construction of the trial embankment. Altogether five inclinometer/ extensometer systems were installed, four at the South trial embankment while the other one was installed near the coastal road in order to measure the effect of the construction of the trial embankment on the road. The inclinometers were installed and anchored at about one metre deep into weathered rock and the borehole of the installed instrument was then filled with bentonite grout having a mixture ratio of 1:8. The inclinometer/ extensometer consists of a number of tubes of one metre lengths as shown in Plate 4.11. Readings of the inclinometer were taken using a torpedo probe attached to a readout unit. The torpedo probe is inserted in the inclinometer/ extensometer tubes down to the bottom of the tube and is then withdrawn and readings taken from the readout units at half a metre intervals, Plate 4.12. The readings for the extensometer are described below in Section 4.2.2.4. The frequency of the readings taken for the inclinometer/extensometer systems was similar to that for the pneumatic piezometers.

4.2.2.4. Extensometers

Altogether five extensometers system were installed, four together with the inclinometers and one installed alone at the centre of the South trial embankment. Spider magnets, Plate 4.13, were installed at depths of 2m, 5m, 8m, 11m similar to the depths of the pneumatic piezometers. The readings of the extensometers were taken using a probe similar to the dipmeter used for taking standpipe piezometer readings except that the probe gave a bleep when the top and bottom of the spider magnet was encountered while no bleep sound was heard when the probe was in the middle of the spider magnet. The extensometers were used to measure the relative vertical movement of the ground at the prescribed locations. The frequency of the readings taken for the extensometers was similar to that for the pneumatic piezometers.

4.2.2.5. **Settlement Plates**

Settlement plates were installed to measure the cumulative vertical settlement of the ground level before, during and after the construction of the trial embankment. A total of twenty six settlement plates were installed at the South trial embankment site. Ten settlement plates were installed along the instrumentation line i.e. in the East-West direction along the sixty metre width of the trial embankment while another sixteen settlement plates were installed along the North-South direction i.e. along the length of the trial embankment which is about ninety metres long. The settlement plates were installed at an average distance of about five metres starting from the centre line of the embankment. The settlement plates consisted of a base plate of 500mm by 500mm square with an extension of 50mm diameter and 25mm internal diameter made of galvanised iron pipe welded to it, Plate 4.14. The pipes were covered by a PVC casing of 85mm diameter, one metre in length. The details of the settlement plates are as shown in Fig.4.10. The settlement plates were installed along the centre line of the embankment. Holes of about 600mm by 600mm square were dug until the original ground level was found and the settlement plates lowered onto this, Plate 4.15. The readings of the base plate were then taken. The holes were then covered up and readings of the top of the rod of the settlement plates and ground levels were taken as shown in Plate 4.16. The frequency of readings of the reduced top level of the settlement plates was similar to that for the pneumatic piezometers.

4.2.2.6. **Heave Markers**

Heave markers were installed in order to determine whether heaving outside the embankment area occurs during and after the construction of the trial embankment. Altogether sixty four heave markers were installed at a distance of about 3m around all sides of the embankment. Plate 4.17 shows the type of heave markers installed.

The frequency of readings for the heave markers was similar to that for the pneumatic piezometers.

4.2.2.7. Temporary Datums

Two temporary datum points were installed as reference points for the surveying of the instrumentation works and fill levels of the South trial embankment. One temporary datum point was installed near the side of the South embankment while the other was installed at some distance away from the trial site. The temporary datum points were constantly checked against the fixed datum point located about one kilometre away nearby a bridge. The temporary datum points consist of 50mm diameter galvanised iron pipe tubes which were installed about 1.5m into the weathered rock. The galvanised iron tubes were jointed at every three metre lengths with PVC couplings. The temporary datum points were then concreted at the base with concrete using 12mm aggregates at the bottom followed by concrete using 19mm aggregates at the top as shown in Fig.4.11.

4.2.2.8. Top Plates

Top plates similar to the settlement plates but smaller in size, 50mm by 50mm square and 0.5m in height, were installed at the top of the embankment after the end of construction to monitor the settlement of the top of the embankment. Altogether twenty four top plates were installed.

4.3. Construction of the South Trial Embankment

The Kuala Perlis South trial embankment was constructed to a finished height of 4m at a rate of construction of approximately 110mm per day. Two types of fill material were used in the trial embankment. Type I fill material was a type of soft reddish brown lateritic soil. This was used at the centrelines of the trial embankment to surround and protect all the instrumentation work. For the rest of the embankment, Type II fill material was used. This fill material consisted of weathered fractured rock which was greyish in colour. Both fill materials were levelled by using backpushers. The South trial embankment was completed in about 36 days after start of construction, the loading sequence being as shown in Fig.4.12.

4.4. Field and Laboratory Testing

4.4.1. Field Testing

4.4.1.1. Subsoil

The field tests on the subsoil were carried immediately after the completion of the preliminary works. Further field tests were also carried out after the completion of the South trial embankment.

4.4.1.1.1. Field Vane Tests

Field vane tests were carried out using the field vane equipment patented by Geonor of Norway. The dimensions of the vane were 55mm in diameter and 110mm in height and the rods about a metre long. The equipment used for field vane testing is as shown in Plate 4.18. Field vane tests were carried out at five locations prior to the construction of the South trial embankment as shown in Fig.4.13. Tests were carried out at every half a metre for the first six metres followed by every metre thereafter until a stiff layer was encountered. Prior to the test, a hole was made using an auger, Plate 4.19, the vane was then inserted into the hole, Plate 4.20, and the tip was then rotated causing the soil to be sheared. The readings of the test were then taken as shown in Plate 4.21. Two field vane tests were also carried out 434 days after the end of construction of the trial embankment.

4.4.1.1.2. **Piezocone Tests**

Piezocone tests were also carried out at five locations prior to the construction of the South trial embankment at the locations shown in Fig.4.13. The type of piezocone used was made by Hoegentogler, USA, Plate 4.22, and the filter used for the piezocone tip was made of polypropylene which was attached to the mid section of the cone. Prior to the tests, the filter was saturated in silicon oil and then attached to the cone tip. This process was undertaken with the filter and cone tip submerged in silicon oil to ensure that no air bubbles entered the filter. Readings of the piezocone tests were taken at every 0.25m until a hard layer was encountered. The readings were plotted as the cone penetrated the ground through a data acquisition system. The type of machine used for the tests is as shown in Plate 4.23. Two piezocone tests were also carried out 594 days after the end of construction of the trial embankment.

4.4.1.1.3. **In-situ Permeability Tests**

In-situ permeability tests were carried out after the installation of the standpipe piezometers. Altogether three in-situ permeability tests were undertaken prior to the construction of the South trial embankment by using the standpipe piezometers. The permeability tests undertaken were of the falling head type. During the test, water was added to the top of the standpipe and the water level allowed to fall down with the time taken for a given height drop measured. The permeability of the soil was then calculated using the standard falling head test equation.

4.4.1.2. **Fill Material**

4.4.1.2.1. **Field Density Tests**

Field density tests were undertaken in order to monitor the in-situ unit weight of the fill materials during the construction of the trial embankment. Two types of field density tests were used, the first type being the common sand replacement method while the other a water replacement method. In the sand replacement method, two types of containers were used, one was 115mm in diameter while the other was 167mm in diameter. Leighton Buzzard sand was used to calibrate the equipment. A hole was dug into the fill, the fill material removed and put into a container and weighed. The fill material was then replaced by the standard sand and the density of the fill material calculated. Details of the sand replacement method are given in BS5930 (1981). For the water replacement method which is an approximate method of determination of in-situ unit weight of the fill material, a hole of reasonable size was dug in the fill, Plate 4.24, and the fill material removed was weighed. The hole was lined with a thin plastic sheet, "cling film", and then filled up with water, Plates 4.25 and 4.26. Knowing the mass of water, hence the volume of water in the hole, the bulk density of the fill material was calculated. Altogether ninety eight field density tests were carried out, fifty using the sand replacement method, twenty nine with the small diameter cylinder and twenty one with the big diameter cylinder, and forty eight using the water replacement method. Of the ninety eight field density tests, sixty two tests were carried out on type A fill material while thirty six on type B fill material. Field density tests on the fill materials were carried out at different heights during the construction of the trial embankment.

4.4.2. **Laboratory Testing**

Altogether seventy seven undisturbed samples were obtained for laboratory testing. The undisturbed samples were sealed with wax and kept in a horizontal position in an air conditioned mobile laboratory on site while awaiting transportation to the main soils laboratory at IKRAM in Kuala Lumpur. The samples were transported using boxes padded with foam or rice husk to prevent movement of the samples. The temperature of the mobile laboratory, transport vehicle and the sampling storage room in the laboratory were maintained at approximately 25°C to 27°C as it was found from preliminary trials that this was similar to the in-situ soils temperature and a suitable temperature for working conditions in the laboratory. Moisture content testing was also carried out in the mobile site laboratory on samples of soil obtained during boring. These were used as a check against on the results obtained from the main soils laboratory testing. The samples tubes were kept horizontally prior to testing, Plate 4.27 and extrusion of the samples was also carried out horizontally using a hydraulic motorised extruder, Plate 4.28.

4.4.2.1. Subsoil

4.4.2.1.1. Classification Tests

4.4.2.1.1.1. Moisture Content Test

The procedure to obtain moisture content was as given in BS1377 (1990) Part 2 and by Head (1980). Moisture content tests were performed on all seventy seven samples from the Kuala Perlis South trial embankment in addition to the one hundred and forty five tests performed at the mobile laboratory on site.

4.4.2.1.1.2. Atterberg Limits Test

The Atterberg limit tests undertaken were the liquid and plastic limit. For the liquid limit, the falling cone test was used while for the plastic limit, the crumbling test was used as described in BS1377 (1990) Part 2 and by Head (1980). Atterberg Limit tests were performed on all seventy seven samples from the Kuala Perlis Trial embankment.

4.4.2.1.1.3. Particle Size Distribution Test

Sieving tests were undertaken in order to obtain the particle sizes of coarse granular soil particles in the soils while the finer particles passing through the 63μm sieve were tested using the hydrometer. The details of the procedures are as given in BS1377 (1990) Part 2 and by Head (1980). Particle size distribution tests were carried out on all seventy seven samples from the Kuala Perlis trial embankment.

4.4.2.1.1.4. **Specific Gravity Test**

Specific gravity tests were carried out on samples at their natural state without oven drying in order not to drive off the adsorbed water surrounding the soil particles. This was as suggested by the Geological Society Engineering Group Working Party Report for Tropical Residual Soils (1990). The rest of the procedure followed BS1377 (1990) Part 2 and by Head (1980). Specific gravity tests were performed on all seventy seven samples from the Kuala Perlis trial embankment.

4.4.2.1.1.5. **Unit Weight**

Unit weight measurements on the undisturbed soft soil were made by weighing test specimens of known dimensions prior to Rowe Cell and shear box tests. Unit weight measurements were taken from forty samples obtained from the Kuala Perlis trial embankment.

4.4.2.1.2. **Shear Strength Tests**

4.4.2.1.2.1. **Direct Shear Box Test**

Direct shear box tests were carried out on thirty samples. The size of shear box used was 60mm by 60mm square. The samples were first saturated and then consolidated prior to shearing. Samples were taken both in the vertical and horizontal alignment. Altogether twenty one samples were taken in the vertical alignment and nine samples in the horizontal alignment. Details of the tests are as given in BS1377 (1990) Part 7 and by Head (1982).

4.4.2.1.2.2. **Laboratory Vane Test**

Laboratory vane tests were carried out on sixty five undisturbed samples. Four laboratory vane tests were carried out at the top of each sampling tube. Prior to the tests, the torsion springs used in the vane test were calibrated in order to determine the stiffness of the spring. The details of the test are as given in BS1377 (1990) Part 7 and by Head (1982) and the apparatus is shown in Plate 4.29.

4.4.2.1.2.3. **Consolidated Undrained Triaxial Test**

Consolidated undrained (CU) triaxial tests were carried out on nine samples, Plate 4.30. The samples were first saturated with a back pressure until the pore pressure coefficient (B) value of 1 was achieved at which point the back pressure valve was closed. The cell pressure was then applied and the sample was allowed to consolidate until 95% consolidation of the sample was achieved. This was determined by the pore pressure build up in the sample. After 95% consolidation had been achieved, the sample was then sheared under undrained conditions until failure occurred. Side drains were used in order to speed up the consolidation process for which corrections had to be made. Altogether three specimens were tested at three different vertical effective stress. The three vertical effective stress chosen for the consolidated undrained tests were the insitu vertical stress, two and three times the insitu vertical stress. Three Mohr circles of the stress strain at failure obtained from the three samples tested were then constructed from which the effective cohesion (c′) and effective angle of friction (ϕ') values was determined. The details of the consolidated undrained triaxial test is given in BS1377 (1990) Part 8 and by Head (1986).

4.4.2.1.3. **Consolidation Tests**

4.4.2.1.3.1. **Standard Oedometer Test**

Standard Oedometer consolidation tests were carried out on thirty samples. The size of sample used was 50mm in diameter by 20mm in height. The details of the Standard Oedometer test is as given in BS1377 (1990) Part 5 by Head (1982).

4.4.2.1.3.2. Rowe Cell Test

Rowe Cell tests were carried out on thirty four samples. The samples were tested using both with vertical and radial drainage. Altogether twenty one samples were tested using vertical drainage and thirteen samples using radial drainage. The layout of the Rowe Cell testing are shown in Plate 4.31. For the vertical drainage, the flow was in two directions. The pressures applied to the samples were 12.5kPa, 25kPa, 50kPa, 100kPa and 200kPa. The diameter of the Rowe Cell used was 76mm while the height of the sample was 30mm. The samples were consolidated until a secondary coefficient of compression could be determined. For radial drainage, a peripheral drain was used to enable the drainage to occur laterally. Prior to testing, the samples were saturated for 24hrs. Details of the tests are as given in BS1377 (1990) Part 6 and by Head (1986).

4.4.2.1.4. Laboratory Permeability Test

Laboratory permeability tests were carried out on three samples using the triaxial apparatus. The samples were allowed to consolidate and permeability tests were carried out after the end of the consolidation stage. Details of test procedures using the triaxial apparatus are described in BS1377 (1990) Part 5 and by Head (1986). Laboratory permeability values were also calculated from consolidation testing using Rowe Cells and Oedometers.

4.4.2.1.5. Chemical Tests

4.4.2.1.5.1. Sulphate Test

Sulphate tests were undertaken on forty five samples to determine the amount of total sulphate content of the soft clay samples. The details of the tests are as described BS1377 (1990) Part 3 and by Head (1980).

4.4.2.1.5.2. **Chloride Test**

Water soluble chloride ion tests were undertaken on twenty eight samples. Details of method are as described in BS1377 (1990) Part 3 and by Head (1980).

4.4.2.1.5.3. **Carbonate Test**

Carbonate tests were undertaken on thirty six samples using the rapid titration method as recommended by BS1377 (1990) Part 3.

4.4.2.1.5.4. **Pore Water Salinity Test**

Pore water salinity tests were undertaken on seventeen samples at various depths. The pore water of the samples were extracted using a hydraulic press and kept in a container. The pore water to be tested was then extracted using a syringe. Details of the method, up to the extraction of the pore water, was described by ASTM (1989). The pore water was then tested for sodium chloride content as described by BS1377 (1990) Part 3 and by Head (1980).

4.4.2.1.5.5. **pH Test**

pH tests were undertaken on forty four samples using the pH meter to determine the acidity and alkalinity of the soil sample. Details of the method are as described in BS1377 (1990) Part 3 and by Head (1980).

4.4.2.1.5.6. **Organic Content Test**

Organic content tests were carried out on twenty five samples. The method used for the determination of the organic content of the soft soil samples was the hydrogen peroxide oxidation method. The details of the tests are as described in BS1377 (1990) Part 2 in the pretreatment stage for organic matter before testing for fine grained particles and by Head (1980). Other methods suggested by BS1377 (1990) Part 3 and by Larsson (1986) were not used due to lack of equipment.

4.4.2.1.6. Fabric, Mineralogy and Carbon Dating

4.4.2.1.6.1. Macrofabric Analysis

Continuous samples of 53mm diameter were obtained for macrofabric analysis using piston sampling methods. Altogether fourteen samples were taken for macrofabric studies. The samples were extruded from the tubes using a horizontal extruder and placed in a halved PVC tubes of 1m lengths. A long thin ruler was inserted about a third of the diameter of the sample and the sample was then carefully prised open in order not to disturb the macrofabric features. The samples were then investigated for macrofabric features and these recorded according to the method suggested by McGown et al (1984). Photographs of the samples were then taken and the samples allowed to dry out. The samples were checked from time to time and photographs were taken as significant macrofabric features become evident.

4.4.2.1.6.2. Microfabric Analysis

Microfabric analysis were carried out on seven samples using the Scanning Electron Microscope. Samples were cut both in the vertical and horizontal alignments in order to obtain an undisturbed vertical or horizontal face for the Scanning Electron Microscope. The freeze drying method was used for the preparation of samples as it was deemed the best method with the least disturbance to the sample. The sample size was 15mm by 15mm by 30mm. The samples were first prepared from the undisturbed sample by using a thin sharp blade and wrapped with a thin cling film to prevent the loss of moisture. Liquid nitrogen was poured into two containers, the first container was to hold the tools used for breaking the samples while the other container was for preparing the samples. A small container was then immersed into the liquid nitrogen container and allowed to cool down. Freon 22, an inert gas was then filled into the small container until liquid Freon was obtained. The sample was then unwrapped from the cling film and immersed into the small container and allowed to freeze for about thirty to forty five seconds. The frozen sample was then taken out and broken into two halves by a bending and pulling action and transferred to the freeze drying apparatus. The amount of time taken for breaking and transferring of the sample was approximately 45 seconds to 60 seconds.

Prior to the preparation of the sample, the freeze drying apparatus was brought down to a starting temperature of about -65°C. The fractured samples were then transferred to the freeze drying apparatus and the temperature of the apparatus was then set at -50°C to allow sublimation to occur. The samples were then left in the freeze drying apparatus for about 36 hours to 48 hours until it was almost completely dried. After this it was transferred to a vacuum desiccator prior to the coating of the samples. The samples were then coated with a gold coat at the freshly broken surface while the sides were covered with putty and painted. After coating, the sample was then put in the Scanning Electron Microscope and viewed.

General features of the sample were then photographed at different magnifications of x350, x700 and x1300. Interesting features were then further magnified at x2500 in order to observe more details. The general procedures of the preparation of freeze dried samples, coating and details of the scanning electron microscope are given by Smart and Tovey (1982) and Mohamed (1986).

4.4.2.1.6.3. **Mineralogy Test**

Mineralogy tests were carried out on eight samples by the National University of Malaysia. The X-ray diffraction method was used to determine the mineralogy of the Kuala Perlis coastal soft soil samples. Hydrogen peroxide was added to the sample prior to the X-ray diffraction tests to get rid of the organic content in the sample. The samples that were tested were taken at various depths and locations on the Kuala Perlis South trial embankment site.

4.4.2.1.6.4. Carbon Dating

Altogether fourteen samples were sent to the Scottish Universities Research Reactor Centre for carbon dating of which three samples were from the Kuala Perlis trial embankment site. Of the fourteen samples, six samples were shell samples while the other eight samples were organic samples. These samples were tested in order to determine the age of the soft soil sample. The samples were taken from four sites in Malaysia namely Kuala Perlis, Sungei Acheh in Penang, Bagan Datoh in Perak and Klang in Selangor. Complete shells were also studied in order to determine the types of depositional environment from which the samples were obtained.

4.4.2.2. Fill Material

Laboratory tests carried out on the two types of fill material included particle size distribution tests, moisture content and direct shearbox tests.

CHAPTER FIVE

ANALYSIS OF DATA OBTAINED FROM FIELD AND LABORATORY TESTING

5.0. Introduction

Data obtained from field and laboratory testing are analysed and summarised into three sections, viz, data obtained directly from field and laboratory testing, correlations between these data and the relationships between these data and previous work undertaken by various researchers in Malaysia and elsewhere on soft soil deposits.

5.1. Data Obtained Directly From Field and Laboratory Testing

5.1.1. Subsoil

Preliminary site investigation work carried out on the site showed that the reduced level of the site is approximately at 1.45m above mean sea level with the water table fluctuating seasonally from 1.6m to 1.1m above mean sea level. Borehole logs of the trial site are shown Appendix 5.1.

5.1.1.1. In-situ Undrained Shear Strength Parameters

In-situ undrained shear strength parameters were obtained from field vane tests. The undisturbed undrained shear strength generally increases with depth, Fig.5.1, with an average of about 12kPa at 1m depth below the mean sea level to about 32kPa at 10m depth below the mean sea level. This indicates that the Kuala Perlis coastal soft soil deposits can be classified as very soft to soft according to BS5930 (1981), Table 5.1. However, sometimes relatively higher than expected undisturbed undrained shear strength are obtained near the top of the coastal soft soil deposits. These may be attributed to the slight overconsolidation of the soil or existence of a weathered crust. The average undrained shear strength ratio obtained from field vane tests, Appendix 5.2, was about 0.69.

The remoulded undrained shear strength also increases with depth with an average of about 4kPa at mean sea level to about 10kPa at 10m depth below the mean sea level, Fig.5.1.

The undrained shear strength with depth (undisturbed and remoulded) 434 days after the start of construction of the trial embankment, Fig.5.2, shows that there is a slight increase of undrained shear strength with time at the centre of the embankment while at the edge of the embankment, the undrained shear strength remains almost constant.

The average sensitivity of Kuala Perlis coastal soft soil deposits varies with depth with an average value of 3, with a range of values ranging from 2 to 7, Fig.5.3. The sensitivity of the soil with depth 434 days after the start of the construction of the trial embankment, Fig.5.4, remains about the same as the average sensitivity value before the construction of the trial embankment indicating that sensitivity values are not affected by the construction of the trial embankment.

5.1.1.2. Cone Resistance

Cone resistance obtained from Piezocone tests, Fig.5.5, similarly shows a higher than expected cone resistance just below existing ground level which may again be attributed due to slight overconsolidation of the soil or the existence of a weathered crust. The cone resistance confirms the presence of soft material down to 10m depth below mean sea level after which the soil gradually stiffens becoming firm to stiff at 12m to 15m.

Cone resistance with depth 594 days after the start of construction of the trial embankment, Fig.5.6, shows very little change with the cone resistance prior to construction of the trial embankment.

5.1.1.3. **In-situ Permeability**

Three in-situ permeability tests were carried out using the three standpipe piezometers installed. Results obtained from these tests are shown in Table 5.2. The tests were carried out prior to the construction of the trial embankment after the laying of the drainage blanket. The in-situ permeability values obtained from these tests ranged from 7 to 12 x 10^{-5} m/s showing a slight decrease in value with respect to depth, Fig.5.7. The values of in-situ permeability were calculated using the equation given by Weber (1968) as :

$$K_{insitu} = \frac{d^2 \left| \frac{mL}{D} + \sqrt{1+(\frac{mL}{D})^2} \right|}{8LT} \qquad \text{(Eqn.5.1)}$$

where

K_{insitu} = in-situ permeability

d = diameter of standpipe

L = length of permeameter

D = diameter of permeameter

m = square root ratio of horizontal to vertical permeabilities (assume m=1 as a first approximation)

T = basic time lag

5.1.1.4. Classification Properties

5.1.1.4.1. Moisture Content

Moisture content tests were carried out both in the mobile laboratory in the field and on undisturbed samples in the main laboratory, Fig 5.8. The moisture contents down to 10m below mean sea level show a wide scatter ranging from 75% to 150% with an average value of 110%. At 10m depth below mean sea level, the moisture content values reduce to an average of 45% and a range of 30% to 60%, gradually decreasing to an average of 27% at 15m depth below mean sea level with a range of 12% to 40%. After 15m depth below mean sea level, the range of moisture content decreases to 10% to 30% with an average of 27%.

5.1.1.4.2. Atterberg Limits

Figure 5.9 shows the relationship of Atterberg limits with depth with a wide scatter of results obtained. However, the liquid limit generally increases with depth, with values ranging 45% to 125% with an average of 93%, from mean sea level down to 10m depth below mean sea level. The average value then reduces to 65% at 10m depth below mean sea level and remains constant to 15m depth below mean sea level with a range of values from 45% to 120%.

Plastic limit values with depth, Fig.5.9, ranges from 12% to 40% up to 10m depth below mean sea level with an average value of 35%. The average value then decreases slightly to 25% from 10m to 15m depth below mean sea level with a range of values from 12% to 35%. These average values of liquid and plastic limits indicate plasticity indices at mean sea level down to 10m depth below mean sea level of 58% and of 40% at 10m to 15m depth below mean sea level. Combining these data with the average moisture content data indicates an average liquidity index of 1.3 between mean sea level and 10m depth below mean sea level, and of 0.3 between 10m and

15m depth below mean sea level with significant scatter down to 10m, Fig.5.10.

5.1.1.4.3. Unit Weight

Unit weight with depth, Fig.5.11, remains relatively constant to a depth of 10m below mean sea level with an average value of 14kN/m^3 and a range of 13.5 to 14.5kN/m^3. The unit weight then increases from an average of 15.5kN/m^3 at 10m depth below mean sea level to 20kN/m^3 at 15m depth below mean sea level.

5.1.1.4.4. Specific Gravity

Specific gravity obtained at the natural state shows that it is relatively constant with depth down to 15m depth below mean sea level, Fig.5.12, the average value being about 2.65 with values ranging from of 2.55 to 2.8.

5.1.1.4.5. Particle Size Distribution

Figure 5.13 shows the particle size distribution with depth. The average clay content remains relatively constant with depth at 65% but ranges from 45% to 80%. The silt content also remains relatively constant with depth at 32% with values ranging from 20% to 45%. The average sand content remains relatively constant with depth at 5% with values ranging from 0% to 25%.

5.1.1.5. Compressibility Parameters

5.1.1.5.1. Preconsolidation Pressure (p_c) and Overconsolidation Ratio (OCR)

Preconsolidation pressures obtained from both the standard Oedometer and Rowe Cell tests are seen to increase with depth, Fig.5.14, down to 10m depth below mean sea level. The average preconsolidation pressure varies from 30kPa at mean sea level to about 65kPa at 10m depth below mean sea level. Below 10m depth there is a wide about 65kPa at 10m depth below mean sea level. Below 10m depth there is a wide scatter of data from the oedometer tests. The present effective overburden pressures are also shown in Fig.5.14. From these data the average overconsolidation ratio deduced is 4 at mean sea level and 1.37 at 10m depth below mean sea level with significant scatter of data in the upper levels. The overconsolidation values obtained from both the standard Oedometer and Rowe Cell tests is seen to decrease with depth, Fig.5.15, with large scatter in values ranging from 0.5 to 6.5.

5.1.1.5.2. **Compression Index (C_c)**

Compression index values obtained from both standard Oedometer and Rowe Cell tests are shown in Fig.5.16. Down to 10m depth below mean sea level, the compression index values ranges from 0.8 to 2.2 with an average value of 1.4 from both test methods. Below 10m depth below mean sea level, the compression index values are generally low, from 0.3 to 2.1 with an average of 1.0.

5.1.1.5.3. **Initial Void Ratio (e_o)**

Initial void ratio obtained from both standard Oedometer and Rowe Cell test samples are seen to decrease with depth, Fig.5.17, with values from mean sea level to 10m depth below mean sea level ranging from 3.8 to 1.9 with an average of 2.9. The void ratio values below 10m then decreases to much smaller values with a range of 1 to 0.6 with an average value of about 0.85.

The variation in void ratio with effective vertical stress obtained from both standard Oedometer and Rowe Cell tests is shown in Fig.5.18. The void ratio values generally decrease with effective vertical stress with a large scatter especially at low effective vertical stress. Using statistical analysis, the average values can be represented by a line shown in Fig, 5.18, having a coefficient of correlation (R^2) which varies from 0.4407 for the Rowe Cell and 0.6009 for the standard Oedometer.

5.1.1.5.4. Coefficient of Secondary Consolidation (C_α)

The coefficient of secondary consolidation (C_α) at in-situ vertical stress with depth obtained from Rowe Cell tests shows a significant scatter with values up to 0.035 although the average values remain fairly constant at 0.013, Fig.5.19. The coefficient of secondary consolidation (C_α) with effective vertical stress, Fig.5.20, shows that the value of secondary consolidation (C_α) increases with effective vertical stress up to about 100kPa before decreasing with effective vertical stress. These can be represented by a line obtained by statistical analysis having a coefficient of correlation (R^2) of only 0.2906 which indicates a significant scatter in the data.

5.1.1.5.5. Coefficient of Consolidation (c_v)

The coefficient of consolidation at in-situ vertical stress with depth obtained from both the standard Oedometer and Rowe Cell tests is shown in Fig.5.21. The coefficient of consolidation values up from mean sea level to 10m depth below mean sea level ranges from 1 to 22.5m^2/yr with an average of 8m^2/yr. From 10 to 15m depth below mean sea level, the average value of the c_v decreases slightly to 2 to 5m^2/yr with a range of 1 to 12.5m^2/yr.

The coefficient of consolidation with effective vertical stress obtained from both the standard Oedometer and Rowe Cell tests, Fig.5.22, shows a decrease in the value of the coefficient of consolidation with increase in effective vertical stress. The scatter of the coefficient of consolidation values tends to be larger at lower effective vertical stress. This indicates that the soil is very soft and compressible at lower effective stress but becomes much firmer as more load is applied. Using statistical analysis, data obtained from standard Oedometer and Rowe Cell tests can be represented by a line having a coefficient of correlation (R^2) of 0.3834 for the standard Oedometer and 0.1359 to 0.214 indicating that there is more scatter in the data obtained from the Rowe Cell tests than from standard Oedometer tests which could be attributed to the size of sample tested.

The relationship between the coefficient of consolidation with void ratio from both the standard Oedometer and Rowe Cell tests, Fig.5.23, shows that the c_v value increases up to a void ratio of 3 before decreasing. The range of scatter is very large at a void ratio of 3. This indicates that the soil can be very compressible around a void ratio of about 3 but less compressible at other values of void ratio. From statistical analysis, both the data from the standard Oedometer and Rowe Cell tests can be represented by a line having coefficient of correlation of 0.1744 to 0.3692 for the Rowe Cell and 0.3857 for the standard Oedometer tests. This indicates that there is a very significant scatter in the data obtained from Rowe Cell tests compared to the standard Oedometer.

5.1.1.5.6. **Coefficient of Volume Compressibility (m_v)**

The coefficient of volume compressibility (m_v) at in-situ vertical stress obtained from both the standard Oedometer and Rowe Cell tests, Fig.5.24, remains a constant up to 10m depth below mean sea level with values ranging from 0.5 to 6m^2/MN with an average of 3m^2/MN. From 10m to 15m depth below mean sea level, the average value of the coefficient of volume compressibility decreases slightly between 1 to 2m^2/MN with a range of values from 0.5 to 4m^2/MN.

The coefficient of volume compressibility is seen to decrease with increase in effective vertical stress, Fig.5.25, with the scatter of values being larger at lower effective stress. This indicates that the soil is very soft at low effective vertical stresses and becomes much firmer at higher effective vertical stresses. From statistical analysis, data from the standard Oedometer and Rowe Cell tests can be represented by a line having a coefficient of correlation (R^2) of 0.0674 for the Rowe Cell and 0.2912 for the standard Oedometer test. These indicate that there is a large scatter in the data from both tests with the scatter from Rowe Cell tests being larger.

The values of the coefficient of volume compressibility show a large scatter with void ratio, Fig.5.26, with an increase in value up to void ratio of 3 before decreasing slightly. This indicates that the soil is very soft and compressible around a void ratio of 3 while being less compressible at other void ratios. Data obtained from both the standard Oedometer and Rowe Cell tests can be represented by line obtained from statistical analysis having a coefficient of correlation (R^2) of 0.555 for the standard Oedometer tests and 0.0624 for the Rowe Cell tests. This indicates that there is a very significant scatter in the data obtained from Rowe Cell tests compared to the standard Oedometer tests.

5.1.1.6. Laboratory Permeability

Permeability values at in-situ vertical stress with depth calculated from Rowe Cell tests remain constant with respect to depth while those obtained from the standard Oedometer show a slight decrease with depth, Fig.5.27. The permeability values at in-situ vertical stress up to 10m depth below mean sea level, range from 1.5E-11 to 4E-08m/s with average values ranging from 1E-09 to 1E-08m/s. From 10m to 15m depth below mean sea level, the permeability values ranges from 2E-10 to 3E-09m/s with average values ranging from 3E-10 to 1E-09m/s.

Permeability values obtained from both the standard Oedometer and Rowe Cell tests show a decrease with effective vertical stress, Fig.5.28. The range of permeability values shows a large scatter at lower effective vertical stress which decreases at higher effective vertical stress. This indicates that the permeability values are larger at lower effective vertical stress or small loading but becomes smaller at higher effective vertical stress or when a larger load is applied. Using statistical analysis, the data obtained from both standard Oedometer and Rowe Cell tests can be presented by a line having coefficient of correlations of 0.2619 to 0.2844 for the Rowe Cell tests and 0.6521 for the standard Oedometer tests.

Permeability with void ratio, Fig.5.29, in general shows an increase with void ratio although this increase is less clear from data obtained from Rowe Cell tests than from standard Oedometer tests. This indicates that the permeability of the soil is much larger at higher void ratio which means that the soil is more compressible. Using statistical analysis, the data obtained from standard Oedometer and Rowe Cell tests can be represented by a line having a coefficient of correlation of 0.0936 and 0.1101 for the Rowe Cell tests and 0.7034 for the standard Oedometer tests indicating a large scatter in the data obtained from Rowe Cell tests.

Permeability values were also obtained from two triaxial permeability tests with different pressure heads. The permeability values obtained is shown in Table 5.3. The permeability values were calculated using the falling head formula as given in Head (1986) as :

$$k_v = \frac{q}{60Ai} \qquad \text{(Eqn.5.2)}$$

where

k_v = vertical permeability (m/s)

q = flow per unit time (ml/min)

A = area of samples (mm^2)

i = hydraulic gradient = $102\Delta p/L$

where

Δp = difference in pressure head = p_1-p_2

L = length of sample (mm)

5.1.1.7. Laboratory Shear Strength Parameters

5.1.1.7.1. Laboratory Undrained Shear Strength and Sensitivity

Undisturbed undrained shear strength values obtained from laboratory vane tests, Fig.5.30, show an increase with depth down to 10m depth below mean sea level, with average values ranging from 8kPa at mean sea level to 20kPa at 10m depth below mean sea level. Below 10m depth there is a wide scatter of the undrained shear data ranging from 12kPa to 60kPa with average values of 25kPa at 10m depth below mean sea level and 33kPa at 15m depth below mean sea level. The average undisturbed shear strength ratio obtained from laboratory vane tests is 0.42, Appendix 5.2.

The remoulded undrained shear strength with depth, Fig.5.30, shows a gradual increase with average values with depth ranging from 3kPa to 5kPa at mean sea level to 10m depth below mean sea level.

The sensitivity obtained from laboratory vane tests shows a significant scatter, Fig.5.31, from 2.5 to 7 at mean sea level with an average of 4 and a range of 4 to 13 at 15m depth below mean sea level with an average of 7.

5.1.1.7.2. **Effective Shear Strength Parameters (c',ϕ')**

From the data of direct shear box tests (vertical and horizontal alignment) as shown in Appendix 5.2, the average cohesion and effective angle of friction values for the direct shearbox tests (vertical and horizontal alignment) can be measured, Fig.5.32. The average cohesion and effective angle of friction from mean sea level up to 10m depth below mean sea level for samples tested in the vertical alignment is 8.82kN/m^2 and 15.5^0 while for the horizontal alignment is 11.76kN/m^2 and 16.5^0.

For consolidated undrained triaxial tests, a p'q' graph, Fig 5.53, is drawn from the data obtained from triaxial tests in Appendix 5.2. From the p'q' graph, Fig.5.53, a' and α' values of 1 and 17^0 are obtained. The values of c' and ϕ' are then calculated using the following equations.

$$c' = \frac{a'}{\cos\phi'} \qquad \text{(Eqn 5.3)}$$

and

$$\phi' = \sin^{-1}(\tan\alpha') \qquad \text{(Eqn 5.4)}$$

where

c' = effective cohesion

ϕ' = effective angle of friction

The average effective cohesion and effective angle of friction value from mean sea level up to 10m depth below mean sea level calculated from Equations 5.3 and 5.4 is $6.17 kN/m^2$ and 17.8^0.

5.1.1.8. **Chemical Properties**

5.1.1.8.1. **Chloride Content**

Figure 5.34 shows the relationship of chloride content with depth. The average value is relatively constant down to 15m depth at about 0.012% with values ranging from 0.007% to 0.026% at 10m depth below mean sea level and 0.007% to 0.016% from 10m to 15m depth below mean sea level.

5.1.1.8.2. **Sulphate Content**

Fig.5.35 shows the total sulphate content of Kuala Perlis coastal soft soil deposits with depth. The range of sulphate content ranges from 0.1% to 1.7% with an average of 1% at mean sea level to 0.05% to 0.75% with an average of 0.4% at 10m depth below mean sea level. From 10m to 15m depth below mean sea level, the range of sulphate content varies from 0.1% to 0.3% with an average of 0.25% The range of scatter is seen to be largest at mean sea level.

5.1.1.8.3. **Carbonate Content**

The average carbonate content varies from 17% to 19% with values ranging from 14% to 22% down to 10m depth below mean sea level, Fig.5.36. From 10m to 15m depth below mean sea level, the range of values varies from 14.5% to 20% with an average of 17%.

5.1.1.8.4. **pH**

Average pH values vary from 7 at mean sea level to 8 at 10m depth below mean sea level with a range of values from 6 to 9, Fig.5.37. The range of pH values then remains constant from 7 to 9 from 10m to 15m depth below mean sea level with an average of 8 indicating that Kuala Perlis soft soils are slightly alkaline.

5.1.1.8.5. **Organic Content**

Organic content is seen to decrease with depth, Fig.5.38, with the range of scatter being largest at the top. The average organic content ranges from 7% at mean sea level to 4% at 10m depth below mean sea level, with the range of values from 3% to 15%. The average value then decreases from 4% to 3% at 10m to 15m depth below mean sea level with values ranging from 2.5% to 4%.

5.1.1.8.6. **Pore Water Salinity**

The average salt content obtained from pore water salinity tests ranges from 23g/litre at mean sea level to 34g/litre at 10m depth below mean sea level with values ranging from 20g/litre to 42g/litre, Fig.5.39. The average values then decrease to 32g/litre and remain constant from 10m to 15m below mean sea level with values ranging from 25g/litre to 35g/litre indicating that the Kuala Perlis coastal soft soil deposits were deposited in a marine to estuarine environment.

5.1.1.9. **Fabric, Mineralogy and Carbon Dating Studies**

5.1.1.9.1. **Macrofabric Studies**

Macrofabric analysis of continuous samples obtained from the Kuala Perlis trial embankment. Photographs of the macrofabric features were taken from continuous samples up to a depth of 13.6m from ground level (11.8m depth below mean sea level) as shown in Plates 5.1 to 5.3. The photographs show that Kuala Perlis coastal soft soil is light to dark grey in colour to a depth about 8.4m (6.6m depth below mean sea level). This then changes to a greenish colour at 12.6m depth below ground level (10.8m depth below mean sea level) and then to stiff mottled light greyish brownish red colour at about 13.2m depth below ground level (11.4m depth below mean sea level). This indicates that the Kuala Perlis coastal soft soil deposits consists of younger deposits down to 12.6m depth below ground level (10.8m depth below mean sea level) which overlies a much older soft soil deposit. The photographs also show that the Kuala Perlis coastal soft soil deposit down to 12.6m depth (10.8m depth below mean sea level) is quite homogeneous in nature with the presence of a lot of broken shell fragments and traces of organic matter. Data of macrofabric analysis carried out on Kuala Perlis coastal soft soil deposits samples are shown in Appendix 5.2 according to the method suggested by McGown et al (1980). The data also confirms the presence of organic matter and sea shells which can affect the strength of the coastal soft soil deposits as well as the presence of sand lenses in most of the samples which can assist in drainage during consolidation of the embankment.

5.1.1.9.2. **Microfabric Studies**

Appendix 5.3 shows the photographs of microfabric features present in Kuala Perlis coastal soft soil deposits taken at various depths using the freeze dried method of preparation. The photographs were first taken at low magnifications of x350 and x700 for general microfabric features. Specific areas of the photographs were then selected and magnified to a magnification of x1300 and x2500 for more details of the microfabric features. The microfabric features were taken both in the horizontal and vertical alignment. The photographs of the microfabric features show that Kuala Perlis coastal soft soil deposits have a flocculated or perturbed arrangement with presence of organic remains of plants or organisms. The existence of organic matter can influence the properties as well as the shear strength of the coastal soft soil deposits depending on the amount of organic matter present. From the microfabric photographs, there seems to be very little difference between the microfabric features of the younger and older soft soil deposits which are clearly shown and differentiated by their colours in the macrofabric photographs. Other microfabric features present in Kuala Perlis coastal soft soil deposits include pyrites both in cluster and pyritohedral form which can affect the properties and determination of organic content as described by Larsson (1990), although their influences on Peninsular Malaysia coastal soft soil deposits have not being investigated.

5.1.1.9.3. **Mineralogy**

Figure 5.40 shows the amount of montmorillonite, illite and kaolinite with respect to depth. The average montmorillonite content varies from 25% at mean sea level to 40% at 10m depth below mean sea level with values ranging from 10% to 50%. The average values then reduces to 15% at 10m depth below mean sea level and remains constant up to 15m depth below mean sea level with a range of values from 5% to 20%.

Average kaolinite content varies from 45% at mean sea level to 50% at 10m depth below mean sea level with values ranging from 35% to 63%. From 10m to 15m depth below mean sea level, the range of kaolinite content varies from 35% to 68% with an average of 50%.

Average illite content varies from 30% at mean sea level to 45% at 10m depth below mean sea level with values ranging from 15% to 55%. From 10m to 15m depth below mean sea level, the range of illite content varies from 30% to 45% with an average of 40%.

5.1.1.9.4. **Carbon Dating**

Carbon dating results of the age of the thirteen samples obtained from four sites on the west coast of Peninsular Malaysia, investigated by the Scottish Nuclear Research Reactor Centre are shown in Table 5.4. The four sites are Kuala Perlis, Sg.Acheh, Bagan Datoh and Klang. Figure 5.41 shows the age of Kuala Perlis coastal soft soil deposits with depth which ranges from about 8000 years to 5000 years B.P. Combining the Kuala Perlis data with data from other sites in Peninsular Malaysia undertaken as part of the present study, Fig.5.42, it can be observed that the age of the soft soil varies approximately between 3000 years to 7500 years B.P. from mean sea level to 15m depth below mean sea level. From 10m to 15m depth below mean sea level, the age varies from 7500 to 8000 years B.P. while from 15m to 23m depth below the mean sea level, the age of the soft soil varies from 8000 years B.P. to 9000 years B.P.. This indicates that the rate of sedimentation of the coastal soft soil deposits from 10m to 23m depth below mean sea level, viz. between 7500 years B.P. to 9000 years B.P., was much greater than the rate of sedimentation that occurs between 10m depth mean sea level and mean sea level viz from 7500 years to 3000 years B.P. In fact the rate of sedimentation between 7500 years B.P. and 3000 years B.P. appears to be decreasing. The different rate of sedimentation that occurs can also influence the properties of the coastal soil deposits indicating that possibly there are two main layers present within Kuala Perlis coastal soft soil deposits with the boundary of the upper and lower layer occurring at between 10m to 15m depth below mean sea level.

5.1.2. **Fill Material**

5.1.2.1. **In-situ Unit Weight**

In-situ unit weight values were obtained from sand replacement and water replacement methods. Results of in-situ unit weight for two types of fill materials plotted with height of embankment are shown in Fig.5.43. The in-situ unit weight of type I fill material varies from 14.5kN/m^3 to 21kN/m^3 with an average of about 18kN/m^3. Type II fill material has an in-situ unit weight of between 16kN/m^3 to 20.5kN/m^3 with an average of about 18kN/m^3. The variability of the values of the in-situ unit weight can be attributed to the method of compaction used for South trial embankment, viz. rapid placement with no rolling or other formal compaction. Another reason is perhaps due to the various types of field instrumentation installed in the trial embankment which restricted the mobility of the lorries and backpusher in some areas.

5.1.2.2. **Moisture Content**

Moisture content of the two types of fill material used in the trial embankment is shown in Table 5.5. Type I fill material has natural moisture content values of about 3 to 4% while Type II fill material has natural moisture content values of about 12 to 15%

5.1.2.3. **Particle Size Distribution**

Particle size distribution tests results carried out on the two types of fill material used on the trial embankment are shown in Table 5.5. Type I fill material has a particle size distribution ranging from 85% sand and gravel, 12% silt and 3% clay while Type II fill material has a particle size distribution ranging from 28 to 46% sand and gravel, 28 to 33% silt and 26 to 28% clay.

5.1.2.4. Shear Strength Parameters

The effective cohesion and effective angle of friction of both Type I and II fill material obtained from slow drained direct shear box tests range between 0 to 1kPa and 47° to 53°.

5.2. Correlations Between Data from Present Study

5.2.1. Subsoil

5.2.1.1. Shear Strength Parameters

5.2.1.1.1. Undrained Shear Strength

The comparison of undisturbed undrained shear strength, Fig.5.44, shows that there is an increase in undisturbed undrained shear strength at the centre while no change is seen at the edge of the embankment. For remoulded undrained shear strength, there seems to very little increase before and after the construction of the trial embankment as also shown in Fig.5.44.

A comparison of undrained shear strength obtained from field and laboratory vane tests is shown in Fig.5.45. Results of undrained shear strength obtained from field vane tests are observed to give much higher values compared to those of the laboratory vane. A ratio of 1.6 to 1 would appear to be appropriate, which could be attributed to disturbance that occurred during sampling and transportation of the samples.

Sensitivity values before and 434 days after the start of construction of the trial embankment, Fig.5.46 shows very little difference both at the edge and centre of the trial embankment.

Sensitivity values obtained from field and laboratory vane tests, Fig.5.47, shows the sensitivity values of the field vane are much smaller than those of the laboratory vane. This could be attributed to disturbance during sampling and transportation and perhaps the dimensions of laboratory vane apparatus which are much smaller compared to the field vane apparatus. A sensitivity ratio of 0.6 to 1 is considered appropriate.

The average undrained shear strength, both undisturbed and remoulded, are seen to increase slightly with organic content, Fig.5.48, however the undisturbed undrained shear strength appears to decrease with moisture content and salt content, Figs.5.49 and 5.50, with the remoulded undrained shear strength remaining constant. .

Average sensitivity values are seen to decrease with liquidity index and salt content, Fig.5.51 and 5.52.

5.2.1.1.2. **Effective Shear Strength Parameters (c', ϕ')**

Fig.5.53 shows the relationship of effective angle of friction with plasticity index obtained for both the direct shear and consolidated undrained triaxial tests. The effective angle of friction (ϕ') with plasticity index obtained from direct shear box tests shows a large scatter. For the direct shear box tests, for samples tested both in the vertical and horizontal alignment, Fig.5.53, the average effective angle of friction with plasticity index appears to be a constant while results obtained from consolidated undrained triaxial tests shows that there is a gradual decrease in the average effective angle of friction values with increasing plasticity index.

5.2.1.1.3. **Cone Resistance**

Comparison of cone resistance before and 594 days after the start of construction of the trial embankment, Fig.5.54, shows that there is very little difference between these values both at the centre and the edge of the trial embankment.

The cone resistance (q_c) with friction ratio, Fig.5.55, shows that Kuala Perlis coastal soft soil deposits can be classified as clayey silts and silty clays or clays with some small portion being sandy silts and silts according to the definition suggested by Robertson and Campanella (1983).

The corrected cone resistance minus the effective stress (q_T-σ_v) is plotted with the undrained shear strength obtained from field vane tests, Fig.5.56. Again from Fig 5.56, the average cone factors (N_{KT}) for Kuala Perlis can be determined which is about 14.5.

5.2.1.2. **Classification Properties**

5.2.1.2.1. **Moisture Content**

Comparison of moisture content values obtained from both the mobile laboratory and the main laboratory, Fig.5.57, shows a large scatter in the values which can be represented by the equation :

$$w_{lab} = w_{insitu} \pm 40\% \qquad \text{(Eqn.5.5)}$$

where

w_{lab} : moisture content obtained from main lab.

w_{insitu} : moisture content obtained from mobile lab.

The range of scatter in the moisture content could be attributed due to method of storage, transportation and waiting time prior to testing of samples.

Moisture content values are almost constant with organic content, Fig.5.58, but decreases with increasing salt content, Fig.5.59.

5.2.1.2.2. **Atterberg Limits**

The average Atterberg limits values remains a constant with increasing salt content, Fig.5.60. This indicates that the Atterberg limits values are not affected by the salt content present in Kuala Perlis coastal soft soil deposit. However the Atterberg limits values do show a slight increase with organic content, Fig.5.61. For Atterberg limits with sensitivity, Fig.5.62, it can be seen that the average liquid limit decreases with increasing sensitivity while the average plastic limit remain constant.

Plasticity index with liquid limit, Fig.5.63, shows that data from Kuala Perlis soft soils lies above the Casagrande A-line and has a medium to extremely high plasticity. The data from Kuala Perlis can be represented by the proposed equation of PI=0.7(LL-6). Plasticity index values shows a decrease with sensitivity, Fig.5.64.

The relationship between plasticity index and clay fraction as shown in Fig.5.65 shows that the Kuala Perlis soft soils exhibit various activities from inactive to active although the average values exhibit normal activity. The activity of Kuala Perlis soft soil with respect to clay fraction, Fig.5.66, shows that Kuala Perlis soil has a swelling potential of more than 25% suggesting the presence of a reasonable amount of swelling clays or montmorillonite. The swelling line was suggested by Seed et al (1962), which is given by :

$$S = 3.6 \ x \ A^{2.44} C^{3.44} \qquad \text{(Eqn.5.6)}$$

where

S = percentage swelling

A = activity

C = percentage of clay fraction (< 2μm)

Liquidity index is seen to decrease with effective overburden pressure and depth, Fig.5.67, with the scatter being largest at lower effective overburden pressure and shallow depths.

5.2.1.2.3. **Unit Weight**

Figure 5.68 shows that the unit weight of Kuala Perlis coastal soft soil deposits decreases with increasing moisture content. The average unit weight values ranges from 19kN/m^3 at 40% moisture content to about 14kN/m^3 at about 130% moisture content.

5.2.1.3. **Compressibility Parameters**

5.2.1.3.1. **Preconsolidation Pressure (p_c)**

Comparison of preconsolidation pressure with depth and the difference between preconsolidation pressure with present overburden pressure are shown in Fig.5.69. These show that the values obtained from the standard Oedometer and Rowe Cell tests are in good agreement with each other down to 10m depth below mean sea level and that the maximum past pressure was 28kN/m^2 greater than the present overburden pressure. This could be due to a 2m layer of soil or a drop in water level of 2.8m below present ground water level.

Comparison of OCR Results with depth, Fig.5.70, shows good agreement between results of standard Oedometer and Rowe Cell except for some odd results at 12m depth below mean sea level which could be due to a hard or firm layer being encountered.

5.2.1.3.2. **Compression Index (C_c)**

Comparison of compression index (C_c) with depth, Fig.5.71, shows that results obtained from the standard Oedometer and Rowe Cell tests are quite in good agreement. The compression index shows a slight increase up to 10m depth below mean sea level with an average value of 1.4 at mean sea level to 1.8 at 10m depth below mean sea level. The average value then decreases to about 1 and remains constant up to 15m depth below mean sea level. Compression index values shows a slight increase with increasing organic content, Fig.5.72. More significant increases can also be seen for compression index with liquid limit, Fig.5.73, and compression index with moisture content, Fig.5.74, and compression index with void ratio, Fig.5.75.

5.2.1.3.3. **Initial Void Ratio (e_o)**

Comparison of initial void ratio results with depth, Fig.5.76, shows good agreement in results between standard Oedometer and Rowe Cell with the average value being 2.9 from mean sea level up to 10m depth below mean sea level. The average value then reduces to about 0.9 between 10m to 15m depth below mean sea level. Initial void ratio values show an increase with Atterberg limits, Fig. 5.77, with the increase being more gradual for the liquid limit than for the plastic limit.

The comparison of initial void ratio values with effective vertical stress, Fig.5.78, obtained from both the standard Oedometer and Rowe Cell tests shows good agreement with each other with the average value being represented by a line obtained from statistical analysis having a coefficient of correlation of 0.5305.

5.2.1.3.4. **Coefficient of Secondary Consolidation (C_α)**

Coefficient of secondary consolidation with compression index shows a scatter and no clear relationship, Fig.5.79. $C_\alpha/(1+e)$ with depth shows a decrease in the average value with depth, Fig.5.80. Using the definition suggested by Mesri(1973), Table 2.7, it can be seen that the coefficient of secondary consolidation of Kuala Perlis coastal soft soil deposit can be classified as having very low to medium compressibility.

5.2.1.3.5. **Coefficient of Consolidation (c_v)**

Comparison of coefficient of consolidation values at in-situ vertical stress with depth, Fig.5.81, shows a scatter with average of value of c_v is about 7.5m^2/yr up to 10m depth below mean sea level which then decreases to 5m^2/yr from 10m to 15m depth below mean sea level. Results obtained from both standard Oedometer and Rowe Cell tests show good agreement.

The comparison of coefficient of consolidation values with effective vertical stress, Fig.5.82 and void ratio, Fig.5.83, shows good agreement between data from standard Oedometer and Rowe Cell tests. The data from Fig.5.82 can be represented by a line obtained from statistical analysis having a coefficient of correlation of 0.2757 while for coefficient of consolidation with void ratio, Fig.5.83, the data from both tests can be represented by a line with a coefficient of correlation of 0.2852.

5.2.1.3.6. **Coefficient of Volume Compressibility (m_v)**

Comparison of the coefficient of volume compressibility (m_v) values at in-situ vertical stress with depth, Fig.5.84, effective stress, Fig.5.85, and void ratio, Fig.5.86, shows that results obtained from the standard Oedometer and Rowe Cell tests are in good agreement with each other with average values of 3m^2/MN down to 10m depth below mean sea level and 2.5m^2/MN from 10m to 15m depth below mean sea level.

Comparisons of coefficient of volume compressibility with effective vertical stress, Fig.5.85, and void ratio, Fig.5.86, can be represented by lines obtained from statistical analysis having coefficient of correlations of 0.2334 and 0.3406.

5.2.1.4. Permeability

Comparison of permeability results with depth, Fig.5.87, effective vertical stress, Fig.5.88, and void ratio, Fig.5.89, shows that the results obtained from in-situ tests are much higher than laboratory tests with values obtained from triaxial test being larger than those obtained from Standard Oedometer and Rowe Cell tests.

Comparison of permeability data obtained from standard Oedometer and Rowe Cell tests with effective stress, Fig.5.88, and void ratio, Fig.5.89, shows that the data from both tests can by represented by lines obtained from statistical analysis with coefficient of correlations of 0.4566 and 0.4429.

5.3. Comparison of Data between Present Study and Previous Work done by Various Researchers

5.3.1. Shear Strength Parameters

5.3.1.1. Undrained Shear Strength

The undrained shear strengths (undisturbed and remoulded) of Kuala Perlis coastal soft soil deposits lie within those found by various researchers in Malaysia, Fig.3.7, which generally increases with depth. Relationship of undrained shear strength with other soil parameters of Kuala Perlis soil also show similar properties to those found by various researchers in Peninsular Malaysia as shown in Figs.3.7 to 3.10.

Sensitivity values also lie within those found by other researchers, Fig.3.11. Sensitivity correlations with other soil parameters for Kuala Perlis coastal soft soil deposits also shows similar relationship to those found by other various researchers, Figs.3.12 and 3.13.

5.3.1.2. **Effective Shear Strength Parameters (c',ϕ')**

The effective angle of friction with plasticity index, Fig.5.90, shows most of the data obtained from Kuala Perlis are found to lie between the lines suggested by Kenney (1959), Holt (1962) and Skempton et al (1960).

5.3.1.3. **Cone Resistance**

Cone resistance with friction ratio from Kuala Perlis suggests that Kuala Perlis soil is mainly clayey silt and silty clay which is in good agreement as found by Dobie and Wong (1990), Fig.3.14.

The values of the relationship between q_T-σ'_v and undrained shear strength obtained from Kuala Perlis appears to be within those found by Wong and Dobie (1990) for other sites in Peninsular Malaysia, Fig.5.91. The average cone factor (N_{kT}) obtained from Kuala Perlis is 14.5 and these are within the N_{kT} values of 13 to 17 found by Wong and Dobie (1990) for their three sites.

5.3.2. **Classification Properties**

Classification properties with depth e.g. moisture content, Atterberg limits., bulk density etc of Kuala Perlis coastal soft soil deposits are in good agreement to those found by various researchers, Figs.3.20 to 3.31. Liquidity index with undrained shear strength, Fig.5.92, shows that the data from Kuala Perlis lies between the sensitivity lines of 10 to 20 suggested by Leroueil et al (1983b). This is not quite true as the most of the sensitivity values are between 1.5 to 6 which indicates that the proposed sensitivity lines suggested by Leroueil et al (1983b) are not suitable for Kuala Perlis and probably Peninsular Malaysia coastal soft soil deposits.

Liquidity index with remoulded undrained shear strength, Fig.5.93, shows that data from Kuala Perlis lies above the lines suggested by Leroueil et al(1983b), Mitchell and Houston (1969) and Wroth and Wood (1978), which indicate that these lines may not be suitable for Kuala Perlis and probably Peninsular Malaysia coastal soft soil deposits.

5.3.3. Compressibility Parameters

Compressibility characteristics with depth e.g. preconsolidation pressure, compression index, void ratio etc from Kuala Perlis soft soil deposits all show good agreement with those found by various researchers in Malaysia, Figs 3.40 to 3.44. Relationship of compressibility characteristics with other soil parameters from Kuala Perlis are also in good agreement with those found by various researchers in Peninsular Malaysia, Figs.3.36 to 3.39. Comparison of preconsolidation pressure values with depth, Fig.5.94, shows that the log e-log p suggested by Jose et al (1989) to determine p_c values are in good agreement with the Casagrande method, which was used to determine the preconsolidation pressure values. Preconsolidation pressure values determined using the equation suggested by Nagaraj and Murthy (1985) (cited by Das (1990)), seems to underestimate the value of preconsolidation pressure because the formula does not take into account moisture content values larger then the liquid limit values, which is the case for Kuala Perlis coastal soft soil deposits.

Compression index with moisture content, Fig.5.95 and compression index with liquid limit, Fig.5.96, shows that the values of compression index obtained from present study are much larger than those calculated using the formulas suggested by Terzaghi and Peck (1967), Lambe and Whitman (1967), Cox(1970), Azzouz et al(1976) and Nagaraj and Murthy(1985) with the closest being the values calculated by Cox(1970).

Coefficient of secondary consolidation with compression index, Fig.5.97 shows that most of data from Kuala Perlis are much smaller in value than those obtained from other soft soil deposits in the world. More tests are necessary in order to establish the ratio of the coefficient of secondary consolidation with compression index for Peninsular Malaysia coastal soft soil deposits.

5.3.4. **Chemical Properties**

Chemical properties with depth like organic content, pH from Kuala Perlis are in good agreement with other data found by various researchers in Peninsular Malaysia. The chloride ion content with depth, Fig.3.34. shows that data from Kuala Perlis are very much less than those obtained from other sites in Peninsular Malaysia. These may be true or can be due to differences in the method of chloride testing. More data are needed in order to verify these results.

5.3.5. **Fabric, Mineralogy and Carbon Dating Studies**

Carbon dating results from the present study are compared with other data obtained from various researchers, Fig.5.98, which shows that the data from the present study lie within sea levels hypothesised by various researchers. This indicates that the samples from the present study were deposited in an estuarine to marine environment.

5.4. **Conclusions**

i. The heterogeneity of Kuala Perlis soft soil deposits can be seen from the large range of scatter in geotechnical properties shown in Figs 5.1 to 5.36. Typical values of the range of scatter and averages are shown in Tables 5.5 to 5.7. The heterogeneity of Kuala Perlis coastal soft soil deposits compares very closely to the heterogeneity of soft soil deposits elsewhere in Peninsular Malaysia as described in Section 5.3 and in Chapter 3.

ii. Changes in geotechnical properties are observed to occur between depths of 10m to 15m below mean sea level. These changes in geotechnical properties are clearly seen by changes in moisture content with depth, Fig.5.5, unit weight with depth, Fig.5.8, compression index with depth, Fig.5.13 and initial void ratio with depth, Fig.5.14. The change in properties observed may be due to changes in the rate of sedimentation of the coastal soft soil deposits that occurred around 7500 years B.P., i.e. difference between 9000 to 7500 years B.P. and 7500 to 3000 years B.P. respectively, as shown by carbon dating of shells and organic matter from four sites, Fig. 5.42. This indicates that Kuala Perlis costal soft soil deposits may be divided into two layers with the boundary of the upper and lower layers at about 10m to 11m below mean sea level, as indicated by the change in properties. This is also confirmed by the change in colour as seen from photographs of macrofabric features which occurred at a depth of about 10 to 11m below mean sea level. However, differences in microfabric features between these layers are quite difficult to identify.

iii. Most of the empirical equations proposed by various researchers for various geotechnical properties of soft soil deposits throughout the world cannot be applied as shown in Section 5.3. More data are needed before proper equations suitable for Peninsular Malaysia coastal soft soil deposits can be suggested. This is likely to be a very useful area of further research as will be discussed at the end of the thesis.

CHAPTER SIX

ANALYSIS OF DATA OBTAINED FROM INSTRUMENTATION WORKS

6.0. Introduction

Data obtained from various instrumentation works such as settlement plates, piezometers etc, are analysed in this chapter. Details of their layout and types of instrumentation installed in the Kuala Perlis trial embankment are as explained in Chapter 4. The Kuala Perlis South trial embankment was constructed in 36 days by the dumping of fill material by lorries. The fill material was compacted only by the lorries and by backpushers which helped to level the fill material. Details of the construction and layout are explained in the previous chapter.

6.1. Analysis of Data from Instrumentation Works

6.1.1. Cumulative Settlement/Heave

6.1.1.1. Cumulative Settlement/Heave with Time

Cumulative settlement and heave with time were measured by surface settlement plates, heave markers and top settlement plates as shown in Appendices 6.1 to 6.8.

6.1.1.2. Cumulative Settlement and Heave with Distance

From the cumulative settlement/heave with time plots shown in Appendices 6.1 to 6.8, cumulative settlement and heave at different time intervals and distance were constructed for the centre lines in both the East-West and North-South directions during and after the construction of the trial embankment as shown in Figs.6.1 to 6.4.

For the centre line in the East-West direction, during construction, Fig.6.1, it can be observed that the maximum cumulative settlement at 20 days follows the normal pattern with the maximum cumulative settlement occurring at the centre however by 36 days, the maximum settlement was 0.22m to 0.25m at about 7.5m each side of the centre of the embankment with the east side slightly the greater. A crack in the fill also occurred at this time to the east of the centre line of the embankment, Plate 6.1. For the post construction period, Fig.6.2, it can be observed that maximum settlements occurred at 7.5m each side of the centre of the trial embankment, always with the east side slightly the greater. The crack in the fill persisted in a south easterly to north westerly direction possibly becoming greater with time. The maximum cumulative settlement after 622 days was about 0.95m. No major heaving seems to have occurred along the Eastern or Western edges of the trial embankment. The top of the embankment at 622 days settled about 0.8m.

For the centre line in the North-South direction, during construction, Fig.6.3, the cumulative settlement at 20 days seems to be generally less than in the East-West directions, nevertheless at the end of construction at 36 days, the maximum cumulative settlement of about 0.25m is observed at 7.5m to the south of the centre of the embankment. After the completion of the trial embankment, a similar pattern of cumulative settlement has been observed in the North-South direction and in the East-West direction, Fig.6.4. The maximum cumulative settlements occurred at 7.5m north and south of the centre of the embankment, with the south the greater settlements. The maximum cumulative settlement measured at 622 days was 0.95m.

6.1.1.3. Cumulative Vertical Settlement Profile with Time

The cumulative vertical settlement profile with time at different depths obtained from the spider magnets of extensometers are shown in Appendix 6.9. In general, it is observed that the cumulative vertical settlement decreases rapidly with depth during the consolidation period studied for the trial embankment. It should be noted that the readings obtained from extensometers are known not to be very accurate and this can be attributed to several factors as listed below :

i. The type of measuring equipment used which can affect the accuracy of the reading. Usually a probe attached to a measuring tape is used, as in the present study, and this can affect the accuracy of the reading up to ± 10mm..

ii. The instruments are easily damaged when inserted in to the ground water as ground water can easily get into the electrical probe and damage it. These electrical probes are not easily repaired. This can be observed from the graphs in Appendix 6.9, where there are gaps in the readings.

iii. Human error during the reading of the instrument.

6.1.1.4. Vertical Settlement Profile with Depth and Distance

From the vertical settlement with time data obtained from extensometers shown in Appendix 6.9, a cumulative vertical settlement in the East-West direction at different time intervals with depth and distance during and after the construction of the trial embankment is shown in Figs 6.5 and 6.6. As expected, it can be observed that during construction, Fig.6.5, the cumulative settlement at various depths are relatively small but after the end of construction, they increase with time, the cumulative vertical settlement at about 22.5m west of the centre of the embankment being greatest.

6.1.2. Pore Pressures and Excess Pore Pressures

6.1.2.1. Pore Pressures and Excess Pore Pressures Dissipation with Time

Pore pressures and excess pore pressure dissipation with time obtained from pneumatic and standpipe piezometers are shown in Appendix 6.10. In general, the pore pressures and excess pore pressures increased during the construction of the embankment and then remained either constant or dissipated slightly with time after the completion of the trial embankment. The pore pressures and excess pore pressures also increased with depth, the maximum pore pressures being observed directly underneath the centre of the trial embankment.

6.1.2.2. Excess Pore Pressures Contours Underneath the Trial Embankment.

From the graphs of excess pore pressures in Appendix 6.10, excess pore pressure contours underneath the trial embankment during and after the construction of the trial embankment were developed. Figs.6.7 to 6.9 shows excess pore pressure contours at 10, 20 and 36 days during the construction of the trial embankment. At 10 days after the start of construction of the embankment, Fig.6.7, excess pore pressure contours of 1m and 2m Hd.H_20 have been developed while at 20 days after the start of construction of the trial embankment, Fig.6.8, excess pore pressure contours of 1m to 3m Hd.H_2O have been developed. At the end of construction, viz. 36 days, excess pore pressure contours of 1m to 5m Hd.H_2O have been developed with the maximum excess pore pressures occurring at the centre of the trial embankment at depths of 5.5m onwards. It can be observed that the excess pore pressure contours reduce away from the centre of the embankment and that the excess pore pressures at the centre also get larger with depth. Similar trends are also observed at 100 days and 300 days after the start of construction of the trial embankment as shown in Figs.6.10 and 6.11. It can also be observed that very little excess pore pressure dissipation has taken place at the centre during these periods which could be attributed to the following :

i. Reductions in the permeability values under increasing effective stress, as discussed in Chapter Five. This change in permeability, in particular close to the drainage boundary, greatly slows up drainage with time thus little or no excess pore pressure dissipation occurs.

ii. Considerable changes in the structure of the soft soil deposits during consolidation of the trial embankment, as suggested by Mesri and Choi (1979), Crooks et al (1985) and Mitchell (1986).

All the pneumatic piezometers were clogged at about 370 days after the start of construction of the embankment. New piezometers were then installed at two locations, PP21 and PP23, at about 622 days after the start of construction of the embankment. From the graphs, Appendix 6.10, it is observed that very little dissipation of excess pore pressures has taken place at the centre of the embankment although some time has elapsed since the old piezometers were clogged. Unfortunately no excess pore pressures contours could be drawn due to lack of data.

The relationship of excess pore pressures with the height of fill during and after construction are shown in Figs.6.12 to 6.15. Fig. 6.12 for PP21 located 2.5m from the edge of the embankment shows that the excess pore pressures increases with height of embankment. The excess pore pressures then continued to increase slightly for a short period after the completion of the trial embankment before decreasing. All the values of excess pore pressures of PP21 are much less than the undrained loading of the trial embankment. Similar behaviour was observed at PP22 and PP24 located 22.5m west and east of the centre of the embankment as shown in Figs.6.13 and 6.14. As for excess pore pressures beneath the centre of the embankment, Fig.6.15, it is observed that excess pore pressure behaviour at the 2m depth are similar as those observed for PP21, PP22 and PP24 but at deeper depths of 5m, 8m and 11m, the excess pore pressures continues to increase for a reasonable period of time after the completion of the trial embankment before decreasing. As can be seen from Figs. 6.12 to 6.15, most of the excess pore pressures follows the behaviour hypothesised by Parry and Wroth (1981) as shown in Fig.6.16 during the construction of the embankment.

6.1.3. Lateral Displacement

6.1.3.1. Lateral Displacement with Depth

Lateral displacements with depth at different time intervals for both the East-West and North-South directions are shown in Appendix 6.11. The lateral displacement during construction seems to have been relatively slow and then became rapid after the end of construction of the trial embankment. The lateral displacement then began to slow down again after about 100 days and remained almost constant up to 622 days. The lateral displacement also decreases with depth. The maximum lateral displacement for inclinometers I22 and I24 located 22.5m west and east of the trial embankment at 622 days after the start of construction of the trial embankment were 0.4m and 0.3m respectively. The maximum recorded lateral displacement for I21 located near the edge of the trial embankment after 370 days was 0.2m after which time the inclinometer was damaged.

6.1.3.2. Lateral Displacement with Distance

Lateral displacement with distance during and after construction are shown in Figs.6.17 and 6.18 obtained from the data of lateral displacement in Appendix 6.11. It can be observed that very little lateral movement occurs during the construction of the trial embankment. The lateral movement becomes rapid just after the completion of the trial embankment, 36 days, and then slowed down with time. The lateral movement is observed to be much more towards the west than the east direction. Lateral displacements values obtained from inclinometer I22 located at 22.5m west of the centre of the embankment was observed to be the greatest. Very little lateral movement was observed from Inclinometer 31 at the road edge indicating that the adjacent road was not greatly affected by consolidation of the trial embankment.

6.1.3.3. Other Relationships of Lateral Displacement

The maximum lateral displacement of inclinometers at various locations with height of fill during and after construction are shown in Fig.6.19. The maximum lateral displacement of all inclinometers seems to be increasing gradually during the construction of the embankment. The increase in maximum lateral displacement then became very rapid after the completion of the embankment, the increase being greatest for inclinometer I22 located 22.5m west of the centre of the embankment. Tavenas et al (1979) suggested that the relationship between maximum lateral displacement with maximum vertical settlement at the centre of the trial embankment can be represented as shown in Fig.6.20. They showed that initially the clay was in an overconsolidated state where the maximum vertical settlement is much greater than the lateral displacement. The clay then changes to a normally consolidated state after certain value of maximum vertical settlement has been achieved. This type of behaviour was also observed for the Kuala Perlis coastal soft soil deposits as shown in Fig.6.21.

The ratio of maximum lateral displacement of each inclinometer with vertical displacement at the centre, is plotted with height of fill is shown in Fig.6.22. The ratio of maximum lateral displacement with maximum vertical settlement seems to increase rapidly for I22 and I24 located 22.5m west and east of the centre of the embankment during the construction of the embankment and continues up to a certain period after the completion of the embankment before decreasing. A similar trend was also observed at I31 located on the road edge but the ratio is much smaller and it starts to decrease at about a height of 2.5m and then decreases further after completion of the embankment. For inclinometer I21 located 2.5m from the edge of the embankment, the ratio of lateral displacement with vertical settlement decreases during construction of the trial embankment. The ratio then increases slightly immediately after completion before decreasing again.

The relationship between increase in volume displaced vertically (ΔV_v) with the increase in volume displaced laterally (ΔV_h) as suggested by Johnston (1973) is shown in Fig.6.23. It can be seen that the data obtained for the Kuala Perlis south trial embankment lies above the undrained response where the increase in volume displaced vertically (ΔV_v) is equal to the increase in volume displaced horizontally (ΔV_h). This indicates that the increase in volume displaced laterally is much smaller than the increase in volume displaced vertically and that the relationship shows approximately a linear increase. From the data plotted in Fig.6.22, the increase in volume displaced horizontally only represents about 3% to 13% of the increase in volume displaced vertically. These values are much lesser than those obtained by Johnston (1973) at Canvey Island which was about 25% of the vertical volume.

Combining Fig.6.22 with Figs 6.7 to 6.11 and Appendix 6.10, it can be observed that although there is a very large increase in vertical volume being displaced by the embankment, very little or no excess pore pressures have taken place at the centre of the embankment even after 300 days after the start of construction. Thus it can be concluded that the soil structure of the soft soil deposits is significantly restructuring. This has the effect of maintaining the same excess pore pressures values at the centre of the embankment.

6.2. Conclusions

i. The maximum cumulative settlement after 622 days is about 1m which occurred at about 7.5m from the centre of the embankment. This was associated with a crack in the fill occurring near that location. No significant heaving seems to have occurred around the edges of the embankment.

ii. Probe extensometers are easily damaged and their readings are also relatively inaccurate. Their use is not recommended if better (more expensive) equipment can be installed.

iii. Excess pore pressures at the centre of the embankment does not seem to have dissipated even at 300 days after the start of construction of the embankment, which can be attributed to change in permeability during consolidation and associated changes in the soil structure of the coastal soft soil deposits. This can be explained by Fig.6.22 and Appendix 6.10, which shows that there is a large increase in volume being displaced vertically although there is a lack of dissipation of excess pore pressure at the centre of the embankment. To date, this phenomena has not been investigated in great detail in the laboratory and should therefore be the subject of future research work.

CHAPTER SEVEN

COMPUTER ANALYSIS OF THE KUALA PERLIS SOUTH TRIAL EMBANKMENT

7.0. Introduction

Analysis of the behaviour of Kuala Perlis South trial embankment using two available consolidation computer programs were carried out in order to make comparisons of the observed behaviour with predicted behaviour. The comparison of these results are summarised in this Chapter.

7.1. Type of Programs Used in Computer Analysis

Two 2-dimensional consolidation programs were used to analyse and predict the total cumulative settlement and excess pore pressure dissipation occurring under the Kuala Perlis South trial embankment. The first program was the Two Dimensional Consolidation program produced by the Transport and Road Research Laboratory (TRRL) of United Kingdom also known as TWODIM. The second program was the Two Dimensional Consolidation program produced by the Bandung Institute of Technology of Indonesia, being employed in other research programmes at the University of Strathclyde. This is also known as Konsolidasi 2-Dimensi (Kon2DN). The Kon2DN consists of two versions, one is the linear version where the values of the coefficient of consolidation at vertical and horizontal direction, c_v and c_h, are assumed constant while the non linear version uses c_v and c_h as a function of void change. Both the TWODIM and the Kon2DN (linear and non linear version) use the Terzaghi-Rendulic pseudo-consolidation theory in their method of analysis.

7.2. Soil Parameters Used in the Computer Programs

The soil parameters used in the programs are briefly summarised below into two sections :

7.2.1. Two Dimensional Program developed by TRRL (TWODIM).

7.2.2. Two Dimensional Consolidation Program developed by the Bandung Institute of Technology of Indonesia or Konsolidasi 2-Dimensi (Kon 2DN).

7.2.1. Two Dimensional Program Developed by TRRL (TWODIM)

The parameters used in the TWODIM include :

i. Effective overburden pressures at centres of layers.
ii. Initial horizontal stresses at centres of layers.
iii. Elastic settlement per $kN/m^2/m$ in each layer.
iv. Values of stress-strain relationship per layer obtained from consolidation tests.
v. Values of deviator stress corresponding to pore pressure parameter (A).
vi. Coefficient of consolidation (c_v and c_h) in the vertical and horizontal direction.
vii. Coefficient of volume compressibility (m_v) in the vertical and horizontal direction.
viii. Coefficient of secondary consolidation (C_α).

The input format and data of the TWODIM used in the analysis are shown in Appendix 7.1. More details of the TWODIM are given in TRRL Laboratory Report 617, Murray (1974).

7.2.2. Two Dimensional Consolidation Program or Konsolidasi 2-Dimensi (Kon 2DN) Developed by the Bandung Institute of Technology of Indonesia

The parameters used in both versions of the Kon2DN, linear and non linear versions, are as follows :

i. Unit weight of soft soil and fill material.
ii. Natural moisture content.
iii. Elastic or Young's modulus.
iv. Coefficient of consolidation (c_v and c_h) in the vertical and horizontal direction.
v. Coefficient of Compression (C_c).
vi. Recompression Index (C_r).
vii. Coefficient of Secondary Consolidation (C_α).
viii. Initial void ratio (e_o).
ix. Preconsolidation Pressure (p_c).

For the non linear version of the Kon2DN, an additional parameter is added. The additional parameter is the permeability index. The input format and data of the Kon2DN (linear and non linear version) are shown in Appendix 7.2 and 7.3. Details of the Kon2DN program are described by Nangoi (1987) and Tandjiria (1991). Examples of analysis using the Kon2DN program have been described by Younger (1992).

7.3. Comparison of Computer Analysis Results with Field Data

Results analysed from the two programs are compared with the measured behaviour from field instrumentation and are summarised as follows :

7.3.1. **Cumulative Settlement**

Cumulative settlement at the centre of the Kuala Perlis South trial embankment from field and computer analysis with time are shown in Fig.7.1. It can be seen that generally the settlement profile obtained from computer analysis from both programs, TWODIM and non-linear version of the Kon2DN, are very good although the linear version of the Kon2DN gave good total cumulative settlement results only up to about 400days before slightly deviating from the actual total cumulative settlement values.

For the total cumulative settlement profile with time at other distances from the centre of the embankment, Fig.7.2, only results analysed from TWODIM are used since the Kon2DN can only give cumulative settlement at the centre of the embankment. It can be seen that generally the total cumulative results at various distances from the centre of the embankment obtained from TWODIM are reasonably good, however it slightly underestimates the total cumulative settlement at distances away from the centre, especially at 7.5m. This could be due to higher total cumulative settlement occurring due to the presence of a crack induced in the fill near this location, as explained in Chapter 6.

7.3.2. **Excess Pore Pressure Dissipation**

Results of excess pore pressure dissipation with time obtained from the TWODIM and Kon2DN consolidation programs have been compared with the field data taken from pneumatic piezometers, Fig.7.3. This shows the excess pore pressure dissipation at the centre of the embankment with time for different depths. In general it can be seen that the prediction of pore pressure dissipation, as given by both programs, is very much higher than the actual measured behaviour. The linear version of the Kon2DN and the TWODIM gave almost similar results at all depths while results obtained from the non linear version of the Kon2DN are only similar at depths of 8m and 11m.

In terms of the trend of the excess pore pressures, the non linear version of the Kon2DN gave a much better trend than the linear version of the Kon2DN and the TWODIM programs, although the magnitudes of the excess pore pressures obtained by the linear version of the Kon2DN and the TWODIM are usually slightly or much better than those obtained by the non-linear version of the Kon2DN.

Excess pore pressure distribution with time obtained at other distances by the two programs show similar predictions as observed at the centre of the embankment with the magnitudes of the predicted behaviour being much higher than the actual or field behaviour, Figs.7.4, 7.5 and 7.6. Most of the predictions made by the two programs generally overestimate the value of the excess pore pressures, except at 2m depth where initially it overestimates the magnitude but after about 400 to 500 days, the Kon2DN (linear version) and TWODIM underestimates the magnitude of the excess pore pressure.

7.3.3. Lateral Displacement

No analysis of lateral displacement were carried out as both the TWODIM and the Kon2DN programs cannot analyse lateral displacement.

7.4. Conclusions

i. Settlement analysis using the Kon2DN, linear and non linear version and TWODIM gave good estimations of the total cumulative settlement at the centre of the Kuala Perlis trial embankment. A major disadvantage of the Kon2DN program is that it cannot calculate settlement occurring away from the centre of the embankment whereas the TWODIM Program can do this. However it is reasonable to say that all of these programs can be used for the analysis of the total cumulative settlement at the centre of the embankment.

ii. Analysis of excess pore pressure distribution using both the Kon2DN, linear and non linear version as well as the TWODIM shows that they generally overestimate the magnitude of the excess pore pressures. Modification of both the two dimensional programs are needed in order to obtain more reasonable magnitudes of excess pore pressure distributions. Alternatively more complex programs should be used although this will be more time consuming.

iii. Analysis of lateral displacements were not carried out as both the Kon2DN and TWODIM programs were unable to analyse lateral displacements. More complex programs are therefore needed to carried out analysis of lateral displacement.

iv. Although the Kon2DN, linear and non linear version gave reasonable predictions of total cumulative settlement but poor pore pressure distribution, some problems were found with the programs as instability of the programs sometimes occurred during their running.

CHAPTER EIGHT

THE BEHAVIOUR OF EMBANKMENTS ON PENINSULAR MALAYSIA COASTAL SOFT SOIL DEPOSITS

8.0. Introduction

Previous work carried out on the field behaviour of embankments on Peninsular Malaysia coastal soft soil deposits are reviewed. Unfortunately the basic raw data is not available for most of these, however sufficient data are available for comparisons to be made between the Kuala Perlis trial embankment and two other embankments of similar height where no ground improvement methods have been used in their construction except preloading. These other sites are the Muar trial embankment and Juru trial embankment sites.

8.1. Review of the Behaviour of Embankments on Peninsular Malaysia Coastal Soft Soil Deposits

Among the earliest geotechnical literature on the behaviour of embankments on Peninsular Malaysia coastal soft soil deposits was the report by James (1970), who described the behaviour of two embankments on thick deposits of normally consolidated sediments in the State of Kedah in Northwest Peninsular Malaysia. The two embankments were constructed at heights of 2.74m and 3.3m respectively with side slopes of 1:2. Both embankments were 61m in length. James (1970) concluded that the in-situ vertical permeability was about 2 to 3 times greater in magnitude than the values obtained from laboratory testing which resulted in rapid consolidation being completed 2 to 3 months after the end of construction of the embankments and not the number of years predicted by the standard Oedometer tests.

James (1970) also observed a reduction in the permeability of the soil as well as increase in strength, the latter occurring during the early consolidation stages. He attributed that the major part of settlement was due to secondary consolidation and mentioned that the normal consolidation theory was only applicable for loads below a certain threshold level. He further mentioned that the sediments showed several characteristics of overconsolidation. With regard to pore pressure distribution, James (1970) described the pore pressure dissipation as being rapid during construction with about 75% to 95% having dissipated 2 to 3 months after the end of construction, with the rate of dissipation slowing down several months after the end of construction, Fig.8.1. He attributed this to a reduction in permeability caused by the consolidation of the soils. Constant permeability tests carried out six months after the construction of the trial embankments showed a decrease in permeability of more than 50% under both embankments, these being at higher effective stresses than before construction. The full data is however not available for the trial therefore no further analysis is possible.

Adachi and Todo (1979) described a case study of a large scale housing project in Prai in the State of Penang. Prior to the construction of the housing project, a 2m fill was constructed on soft soil deposits of 10 to 25m in thickness. Adachi and Todo (1979) described the results of field instrumentation and showed that the rate of excess pore pressure was slower than the rate of settlement. They mentioned that excess pore pressure corresponding to at least 20 to 30% of the applied loading still remained although primary consolidation has been completed.

The Prai project was also used as one of the six case studies examined by Mesri and Choi (1979) to study excess pore pressures during consolidation. Mesri and Choi suggested that when the effective vertical stress approaches a "critical" pressure, the settlement will continue at a nearly constant value of excess pore pressure and that the magnitude of the excess pore pressure will be a function of the pressure increment as well as the ratio of the critical pressure to the present effective overburden. They concluded that pore water pressure behaviour could be explained in terms of shape of the void ratio-log effective stress relationship and that the observed behaviour of the structural collapse of the soil cannot be explained at the present time. For the Prai project, Fig.8.2, Mesri and Choi (1979) explained that the excess pore pressure has dissipated by about 40% in less than one month at the mid depth of the layer but then had dissipated by only a further 10% in the following twelve months, although the settlement has increased by 15% to 35%. These data are again not available in detail and no further investigation is therefore possible.

Wong and Choa (1991) carried out a back analysis of the settlement of the fill area in Penang using the CONSOL Computer Program developed by Wong and Duncan (1984). They used laboratory c_v values obtained from standard Oedometer tests, assumed double drainage and found that the predicted and field settlements were in good agreement with each other, Fig.8.3.

In 1987, an intensive full scale trial was initiated by the Malaysian Highway Authority at the Muar Flats in State of Johore to study the behaviour of embankments using various ground improvement methods. Altogether eleven trial embankments were built with various ground improvement methods such as piles, vertical drains, vacuum preloading, vertical drains with geotextile, etc. Two control embankments of 3m and 6m heights were also constructed. Besides the 13 trial embankments, another embankment was also constructed up to a critical height until failure occurred to the embankment. Data from the Muar trial embankments were presented in an international symposium held in 1989. The Proceedings of the Symposium, M.H.A.(1989), reported that predictions of vertical settlements, excess pore pressure and horizontal displacement were found to vary from good to satisfactory to poor. Prediction of the height of the failed embankment was also made but the estimates were found to be very variable depending on the various assumptions made, especially the value of undrained shear strength of the soft soil deposits and the value of the shear strength of the fill material.

Hudson (1990) described the performance of a low embankment constructed in 1983 in Muar prior to the Muar Trial embankments. The embankment was constructed in ten months and laid in layers of 200mm thickness until a final height of 5.5m was achieved. The embankment had a 3m berm on one side which give the embankment a lozenge shape having a maximum length of 100m but with an effective length of only 70m. Hudson has indicated that although settlement is still continuing, little dissipation of excess pore pressure have been observed. He also analysed settlement data collected over seven years using the method proposed by Tan (1971) and found that the method gave a reasonable prediction for the first two years but then began to underpredict the total settlement.

Albakri et al (1990) described two trial embankments constructed in Juru in the State of Penang. The two embankments had dimensions of 100m by 56m in plan and were 4m in height. The first embankment was constructed as a control embankment while the other was installed with vertical drains. Albakri et al (1990) described some early performance of the trial embankments. The performances of the two trial embankments were later explained in more detail by Mohammad et al (1991) who stated that the embankment with vertical drains settled more than the control embankment but that slightly lower excess pore pressure values were obtained as shown in Fig.8.4.

Younger (1992) mentioned that the actual rate of settlement of the two control embankments of 3m and 6m heights in Muar would be underestimated by about 20 to 30 times if actual c_v were used. He also suggested that if lower values of c_v were used, good prediction of pore pressure dissipation could be made. Younger (1992) described the use of two dimensional computer analysis computer program known as Kon2D to predict pore pressure behaviour of the 3m control embankment in Muar. Values of the prediction were compared with actual values as shown in Fig.8.5. Further analysis of the Program using variable pore pressure parameters (A) were also carried out, Fig.8.6. Predictions obtained using this method of analysis were found to be reasonable except at the embankment toe which Younger (1992) attributed to the lateral movement or squeezing of the foundation soil at shallow depths, accompanied by the release of ground water and lower initial pore water pressure build up. Younger (1992) also mentioned that his Program had been extended to include varying permeability with time. Using this method, pore pressure dissipation at the Juru trial was predicted and showed good results compared to the original program which used the Terzaghi-Rendulic equation as shown in Fig. 8.7.

8.2. Details of Construction and Layout of the Three Chosen Trial Embankments

Having regard to the availability of data, three trial embankments were chosen for comparison. These are the Muar, Juru and Kuala Perlis trial embankments.

The Muar control trial embankment was 3 metres in height, constructed in 287 days with three rest periods within the construction period at 18 to 35 days, 52 to 140 days and between 168 to 280 days. These rest periods were a result of problems with the construction of an adjacent embankment. The dimensions of the 3 metres high Muar trial control embankment was 50 metres in length by 32 metres in width with a finished height of 3m. The 3m Muar control embankment had a side slope of 1:2.

The Juru trial control embankment was constructed in about 150 days with dimensions of 100 metres in length and 56 metres in width with an embankment finished height of 4 metres. The embankment was stage constructed with a berm at two metre height with a side slope of 2:3. Details of the cross section of the layout of the three trial embankments are shown in Fig.8.8.

As described previously, the Kuala Perlis South trial embankment was constructed in 36 days with a berm at two metre height and have side slopes of 2:3. The dimensions of the Kuala Perlis South trial embankment was 90 metres in length and 60 metres in width.

8.3. Geotechnical Properties of the Soils Beneath The Three Trial Embankments

Geotechnical properties of the soils beneath the three trial embankments were obtained from field and laboratory tests which have been reviewed in detail in Chapter 3 and are broadly characterised in Table 8.1. The geotechnical properties from the three trial embankment sites may be stated to be really quite similar, therefore the behaviour of the three trial embankments are likely to depend on the method and the rate of construction.

8.4. Comparison of the Field Behaviour of the Three Trial Embankments

The field behaviour of the three trial embankments obtained from various instrumentation installed in the three trial embankments such as settlement gauges, pneumatic piezometers and inclinometers are summarised as follows :

8.4.1. Cumulative Settlement/Height of Fill at the Centre of the Embankment

Figure 8.9 shows the cumulative settlement and height of fill at the centre of the embankment with time for the three trial embankments. It can be seen that the cumulative settlement at the centres for the three trial embankments are almost similar in trend and magnitude. The slight differences in the trend and magnitude of the cumulative settlement between the three embankments can be attributed to the following reasons;

i) The Methods and Rates of Construction

The type of construction method used in the construction of an embankment viz. Kuala Perlis (rapid method) and Juru and Muar (slow method) can influence the trend and the final height of fill achieved. It is observed that for a rapidly constructed embankment viz. Kuala Perlis trial embankment which was constructed in 36 days, very little consolidation occurs during construction therefore the final height of fill of the embankment was approximately equal to the height of embankment. For a slowly constructed embankment, viz. the Muar and Juru trial embankments, the final height of fill of the embankment was much higher than the designed height of the embankment because consolidation and settlement occurred during the construction of the embankment. Thus the cumulative settlement at any time after construction at the centre of the Kuala Perlis trial embankment, is slightly smaller than at Muar and Juru. The rate of increase in the cumulative settlement at Kuala Perlis was also very rapid up to about 50 days after the start of construction of the trial embankment compared to those of Muar and Juru.

ii) The Duration of Rest Periods Taken During the Construction of the Trial Embankment

The effect of the duration of rest periods taken during the construction of the trial embankment can be clearly observed in the Muar trial embankment which had three rest periods during construction. This resulted in the 3m high trial embankment having a final height of fill of about 4m and having settlement equivalent to a four metre high embankment which was constructed with no significant rest period during construction viz. Kuala Perlis and Juru trial embankments.

iii) The Final Height of the Fill Material

The final height of fill material will also influence the magnitude of the final cumulative settlement. For the three trial embankments, it can be seen that the Juru trial embankment has a higher cumulative settlement than Kuala Perlis and Muar because of slightly higher final height of fill.

Figure 8.10 shows the cumulative settlement at the centres of the three trial embankments with the height of fill. It can be observed that there are many similarities in the trends of the cumulative settlement at the centre of the embankment with height of fill between the three trial embankments. The differences between the values of cumulative settlement at the centres of the three embankments can be attributed to the methods of construction used and the rest period taken during the construction, as described earlier.

8.4.2. Excess Pore Pressures

Excess pore pressures at the centre of the three trial embankments (Kuala Perlis, Muar and Juru) with time are shown in Fig 8.11. All the three trial embankments shows an increase in excess pore pressures during construction with little dissipation of excess pore pressure thereafter, i.e. the excess pore pressure remains fairly constant. In general it can be said that the trends and magnitudes of excess pore pressures with time obtained from Kuala Perlis trial embankment are similar to those from Muar and Juru, although some notable differences exists. These differences in the trends and magnitudes of excess pore pressures for the three trial embankments can be due to the following reasons :

i) The Type of Construction Method

The effect of the different types of construction methods used viz. Kuala Perlis (rapid method) and Muar and Juru (slow method) are observed in Fig.8.11. For a rapidly constructed embankment viz. Kuala Perlis, the initial excess pore pressure build up was very fast compared to those of the slowly constructed embankments at Muar and Juru.

ii) The Duration of Rest Period Taken During the Construction of the Trial Embankments

The effect of the durations of rest periods taken during the construction of an embankment can be clearly observed in Muar where three rest periods were taken during the construction of the 3m high embankment. Some dissipation of excess pore pressures was observed to have taken place at each rest period. No such rest periods were taken Kuala Perlis and Juru, hence there were no dissipation of excess pore pressures during construction.

Figures 8.12 to 8.14 show the excess pore pressures from the three trial embankments with the height of fill. It can be observed that there are again similarities between the three embankments. The increase in excess pore pressures do however, show slight differences due to the following reasons :

i) **The Type of Construction Method**

It is observed that a rapidly constructed embankment viz. Kuala Perlis will give rise to a slightly higher initial excess pore pressure compared to the slowly constructed embankments viz. Muar and Juru.

ii) **The Depth of Ground Water Table**

The depth of ground water table will influence the amount of surcharge or loading that is applied to the soil because as the embankment consolidates, some of the fill material will go beneath the ground water level and therefore the effect of buoyancy requires to be taken into account. This effect was observed in Kuala Perlis where the ground water level was about the same as the original ground level. At Muar and Juru the effect of the ground water table was not observed as the depth of ground water level was very much lower than the cumulative settlement of the fill material.

8.4.3. Lateral Displacement

Figure 8.15 shows the lateral displacement at the edges of the three trial embankment at 36, 100 and 370 days. In general, the amount of lateral displacement that occurred in the three embankments increased with time although some differences in the rate of the magnitude of lateral displacement at different time periods are observed. This can be attributed to the following reasons :

i) The Type of Construction Method

It should be noted that a rapidly constructed embankment such as at Kuala Perlis will give a much higher lateral displacement initially compared to the slowly constructed embankments as at Muar and Juru, due to the possibility of plastic flow at the edges being reduced.

ii) Height of Fill

The height of fill achieved during the construction of embankment at different time periods will increase the amount of lateral displacement as shown in Fig.8.15.

iii) Period of Construction

The period of construction of an embankment will influence the rate of lateral displacement achieved. The faster the construction period, the faster the rate of lateral displacement although the final lateral displacement achieved may be the same.

iv) Effect of Time After the Construction of the Trial Embankment

The effect of time will help to increase the final lateral displacement achieved after the completion of the trial embankment although the amount of lateral displacement will reduce with time as shown in Fig.8.15.

Figures 8.16 and 8.17 shows the maximum lateral displacement at the edge of the fill with height of fill and cumulative settlement at the centres of the three trial embankments. It can be observed that generally there is an increase in maximum lateral displacement with height of fill and cumulative settlement at the centre of the embankment although there are some differences in the magnitudes of maximum lateral displacement developed during different construction time periods. These differences can be attributed to the reasons explained earlier.

The increase in volumes displaced vertically and laterally by the three trial embankment are shown in Fig.8.18. All the data obtained from the three trial embankment plot above the undrained behaviour line where the increase in volume displaced laterally equals the increase in volume displaced vertically indicating that the increase in volume displaced laterally is quite small compared to the increase in volume displaced vertically. Table 8.2 shows the increase in volume displaced laterally as a percentage of the increase in volume displaced vertically for the three trial embankment. From Table 8.2, The Juru trial embankment has the largest displaced in lateral volume which is about 13% to 37% of the vertical volume, while the Muar and Kuala Perlis embankments have almost the same amount of lateral volume displaced at 15% of the vertical volume.

Referring again to Fig.8.18 and to the dissipation of excess pore pressures with time, Fig.8.11, it can be seen that large vertical settlements are occurring with little dissipation of excess pore pressure with time and with relatively small amounts of lateral displacement. This requires that the structure of the soft soil deposits have to change a great deal during consolidation. This would cause significant reductions in the depth of the soil without the requirement to expand laterally. It would however maintain pore pressures as the volume of voids would reduce to compensate for the loss of volume of water from the voids.

Fig.8.19 a and b shows the relationship between the ratio of the increase in volume displaced vertically and horizontally with the height of fill for the three trial embankments in Peninsular Malaysia and three other embankments constructed in temperate regions, viz. St.Alban test fill B (Canada), Tavenas et al (1974), the I-95 trial embankment in USA, Poulos (1972) and the Over Causeway By-Pass, Symons and Murray (1975). The data are for time periods of up to two years from the beginning of construction. It can be observed from Fig.8.19a that for the same heights of embankments, the ratio of $\Delta V_v/\Delta V_h$ due to embankments in Peninsular Malaysia, whether rapidly or slowly constructed, are much higher than those obtained from embankments constructed in Temperate Regions. This indicates that the magnitudes of vertical settlement are very much larger for the same magnitude of lateral displacement for embankments constructed in Peninsular Malaysia than in the Temperate Regions. The ratio of $\Delta V_v/\Delta V_h$ is observed to both increase or decrease with increase in height of fill, Fig.8.19b. Why this occurs must be the subject of further research but could be related to the methods and rates of construction used. The average $\Delta V_v/\Delta V_h$ ratio at the end of construction for embankments in Peninsular Malaysia ranges between 7 to 10 and is about 5 for the Temperate Region embankments.

8.5. Conclusions

i. The field behaviour of the Kuala Perlis trial embankment are found to be quite similar to those observed at Muar and Juru although slight differences do occur. These differences can be reasonably explained to be due to the differences in the methods and rates of construction used rather than to differences in the geotechnical properties of each site. The three sites were found to have very similar soil properties as was shown in Table 8.1. Therefore it can be concluded that the field behaviour observed at Kuala Perlis is not unique and can be used as a guide to the field behaviour of embankments on Peninsular Malaysia coastal soft soil deposits. The field behaviour does, however, have some rather special characteristics associated with large changes in the structure of the soils during consolidation, as discussed previously by Mesri and Choi (1979) and Mitchell (1986). These large changes in soil structure have the effect of allowing large vertical settlements without associated large lateral deformations or dissipation of pore pressures with time. This is shown in Fig.8.19 where the ratio between the volume displacement vertically and the volume displaced horizontally with height of fill shows that the $\Delta V_v/\Delta V_h$ ratio are much higher for embankments in Peninsular Malaysia than in the Temperate Regions.

CHAPTER NINE

CONCLUSIONS

The main conclusions of this study can be summarised in four main sections as follows :

9.1. The Origin and Formation of Peninsular Malaysia Coastal Soft Soil Deposits

i. Peninsular Malaysia coastal soft soil deposits are tropical soft soil deposits classified under Tropical Weathered Transported and Redeposited Materials, McGown and Cook (1994). The soil are formed from the breakdown of various parent rocks by tropical weathering. The coastal soft soil deposits formed are very heterogeneous in nature due to the geomorphology of Peninsular Malaysia which consist of short rivers, subject to flooding down steep gradients, McGown and Cook (1994). This was confirmed by the present study which showed that the rivers in Peninsular Malaysia flood during the monsoon seasons with lengths ranging from only 60km to 420km and with gradients from the source to the coastal plains ranging from 1:15 to 1:270, Table 3.1. The heterogeneity of the soil, formed by the mixing of the tropically weathered materials from various parent rocks and organic debris during transportation, McGown and Cook (1994), can be clearly observed from the variations in engineering properties of these materials, with variations laterally and from depth to depth depending on the local conditions of the site.

ii. The formation of coastal soft soil deposits in Peninsular Malaysia has been affected by the changes in sea level during the Pleistocene and Holocene period. The effects of tectonic movements were not considered to be important since Peninsular Malaysia lies in a tectonically stable region known as the Sunda Land, Tjia (1977; 1980). The changes in sea level are characterised by transgressions and regressions of the sea which can lead to the mixing of various materials as they are deposited in different depositional environments. A critical review carried out by the present study shows that carbon dating results obtained by several researchers, Keller and Richards (1967), Tjia (1975), Streif (1979) and Malaysian Highway Authority (1989), did not clearly determine the depositional environments of the materials they tested as shown in Figs.3.1 and 3.2. Some of the samples tested were found not to have been laid down under marine conditions but either in freshwater or brackish conditions. Thus it has been shown to be critical to establish the depositional environment of the materials prior to carbon dating them as using carbon dating results obtained from materials of unknown depositional environment can be misleading in establishing the geological history of these materials and their geotechnical properties.

iii. Although the coastal soft soil deposits of Peninsular Malaysia have been divided into an upper and lower layer by several researchers, Kobayashi (1990), Raj (1990, 1992), Aziz (1993) and Mohamad (1994), no clear evidence has been found to substantiate this division. The detailed review carried out as part of the present study has shown that the proposed division into two layers failed to take account of the changes in the depositional environments. Also some researchers, Kobayashi (1990) and Mohamad et al (1994) did not establish whether these layers were formed in the Pleistocene or Holocene Period. However, evidence obtained from carbon dating of shell and organic matter obtained from undisturbed samples of four sites carried out in the present study, suggests that the coastal soft soil deposit of Peninsular Malaysia may be divided into two layers within the Holocene period as a result of different rates of sedimentation occurring before and after 7500 years B.P., Fig.5.42. This represents a change at a depth of 10m to 11m below present mean sea level. Further detailed investigations and research are necessary in order to verify the existence of these two layers at these depths around coastal Peninsular Malaysia.

9.2. Engineering Properties of the Coastal Soft Soil Deposits at Perlis and Other Sites in Peninsular Malaysia

i. The heterogeneity of Peninsular Malaysia coastal soft soil deposits, as suggested from geological evidence, can be clearly seen from the large scatter in the data of the engineering properties of the materials obtained from both field and laboratory testing at various sites. These data were obtained from tests which were carried out on undisturbed samples obtained from 17 sites in Peninsular Malaysia. The heterogeneity of the coastal soft soil deposits were confirmed by the large scatter in the engineering properties of the undisturbed samples obtained from the trial site at Perlis.

ii. The values of the geotechnical parameters obtained for Peninsular Malaysia coastal soft soil deposits were found to be comparable with typical values of soft soils throughout the world as shown in Table 3.10, Das (1990), with the particular exception of the values of their compression indices, which were found to be much higher than the typical values for soft soil deposits. This indicates that the coastal soft soil deposits in Peninsular Malaysia have extremely high compressibility. A review carried out by the present study shows that there is no definition for soft soil deposits having extremely high compressibility. The present classification according to the volume of compressibility, m_v, is only for soft soil deposits having very low to very high compressibility, Head (1980).

iii. Salinity results reviewed in the present study obtained by several researchers, Ting and Ooi (1977), Nicholls and Ho (1990), showed that the salinities of the coastal soft soil deposits in Peninsular Malaysia range from 0.01 to 43 gm/litre. Interpretations which have been made of these data are quite misleading as the depositional environment of the soils tested were not known. In fact, it is highly likely that little leaching of the salt content has occurred therefore, the salinity results indicate that the coastal soft soil deposits were not deposited in marine conditions but in freshwater to brackish to marine conditions. This is confirmed by the salinity results obtained from undisturbed samples at Perlis which ranges from 24 to 35gm/litre suggesting brackish to marine depositional conditions. Therefore the coastal soft soil deposits in Peninsular Malaysia should not be termed " marine clays " as commonly used by many researchers, as they were not necessarily deposited under marine conditions.

iv. In the past, the properties of the coastal soft soil deposits in Peninsular Malaysia have been suggested by some researchers as being similar to those of Norwegian " quick " clays and that they have been affected by the leaching process. This have been found to be inappropriate and misleading so far as the geotechnical properties of the coastal soft soil deposits of Peninsular Malaysia are concerned.

v. The present study carried out at Perlis shows that there is the presence of a reasonable amount of carbonates, shells and organic matter and pyrites in the coastal soft soil deposits but their effect on the strength and properties of the coastal soft soil deposits in Peninsular Malaysia is not clear. Further investigation is needed in this regard.

vi. From the present study carried out at Perlis, changes in geotechnical properties were observed at 10m to 11m depth below present mean sea level. These changes are clearly seen from several relationships of the properties such as moisture content with depth, unit weight with depth, compression index with depth and void ratio with depth although much lesser clearly in other relationships. This confirms the findings of the carbon dating results mentioned earlier, i.e. that there are probably two layers within the coastal soft soil deposits at Perlis. This is also supported by the photographs from macrofabric studies which show the changes in colour at that depth although differences in microfabric features between these layers were difficult to identify.

vii. It is suggested that the empirical equations proposed by various researchers for the geotechnical properties of soft soil deposits throughout the world do not apply for the coastal soft soil deposits in Peninsular Malaysia. More data and analysis are needed before appropriate empirical equations suitable for Peninsular Malaysia coastal soft soil deposits can be suggested.

9.3. Prediction of the Field Behaviour of the Trial Embankment at Perlis Using Two Standard Consolidation Programs.

i. Because of the large scatter in data due to the heterogeneity of the samples at Perlis, average values of the geotechnical properties were determined and used in the predicted analysis of the field behaviour of the trial embankment at Perlis. These values were found to give a reasonable prediction of the cumulative settlement at the centre of the trial embankment at Perlis using two consolidation programs, TWODIM and Kon2DN (linear and non-linear version). These programs provided data which were in good agreement with the measured data. However, prediction of cumulative settlement away from the centre of embankment could only be carried out by one program, TWODIM, and this was found to be much less accurate.

ii. Predictions of excess pore pressures by both the two consolidation programs tended to overestimate the magnitudes of the excess pore pressures. It is therefore suggested that modification of both these programs or the use of more complex program is necessary in order to obtain reasonable values of the magnitudes of excess pore pressures.

iii. Both the TWODIM and the Kon2DN were unable to analyse lateral displacements. More complex programs are therefore needed in order to analyse lateral displacements.

9.4. Field Behaviour of Trial Embankments at Perlis and Other Sites in Peninsular Malaysia

i. The rate of increase in the cumulative settlement of the trial embankment at Perlis was very fast during the construction of the embankment but it then rapidly slowed down after completion of the embankment. The maximum cumulative settlement at 622 days after the start of the construction of the trial embankment was about 1m which occurred not at the centre but at about 7.5m from the centre of the embankment. This was associated with a crack in the fill occurring near that location. No significant heaving seems to have occurred around the edges of the embankment. The cumulative settlements of the trial embankment at Perlis when compared to those obtained from the trial embankments at Muar and Juru show similar results although slight differences did occur due to the methods and rates of construction used.

ii. The initial increase in the magnitude of excess pore pressure at Perlis was very fast due to the rapid construction of the trial embankment. The pore pressures then remained almost constant with only some slight dissipation at the centre of the trial embankment at 300 days after the start of construction of the embankment. The lack of dissipation of excess pore pressure largely confirms previous work carried out on the Muar and Juru trial embankments.

iii. Similar behaviour were also observed for lateral displacements at the edge of the fill. Lateral displacement at Perlis occurred very rapidly during construction but then slowed down after the completion of the embankment. Similar behaviours were observed at Muar and Juru, although the magnitudes of the lateral displacements obtained at the three sites varied due to the methods and rates of construction used.

iv. The lack of dissipation of excess pore pressure has been attributed to the change in permeability during consolidation associated with large changes in the soil structure, Mesri and Choi (1979) and Mitchell (1986). For Perlis, this can be explained by Fig.6.22 and Appendix 6.10, which shows that large volumes of soil were being displaced vertically although there was a lack of dissipation of excess pore pressure at the centre of the embankment. This indicates that the soil must have restructured in order to maintain the same excess pore pressures. This phenomenon has also been observed at Muar and Juru.

v. The field behaviour of the trial embankment at Perlis was very similar to the behaviours of the trial embankments at Muar and Juru. Therefore, it is quite likely that the field behaviour at Kuala Perlis can be used as a generalised example to describe the general behaviour of trial embankments built over Peninsular Malaysia coastal soft soil deposits.

vi. It is shown that the ratio of the increase in volume displaced vertically with increase in volume displaced horizontally ($\Delta V_v/\Delta V_h$) with height of fill for the trial embankments in Peninsular Malaysia are much higher than those obtained from trial embankments in the Temperate Region. This indicates that larger deformations are occurring vertically compared to laterally for the trial embankments in Peninsular Malaysia than for those in the Temperate Regions. The ratio of $\Delta V_v/\Delta V_h$ is observed to either increase or decrease with height of fill during construction depending on the methods and rates of construction. The values of $\Delta V_v/\Delta V_h$ for the trial embankments in Peninsular Malaysia at the end of construction ranged from 7 to 10 compared to about 5 for the trial embankments in the Temperate Region.

FUTURE RESEARCH

On the basis of the present study, a number of important areas of future research work have been identified as being necessary to further develop an understanding of the behaviour of Peninsular Malaysia coastal soft soil deposits.

i. A detailed geological investigation of Peninsular Malaysia coastal soft soil deposits should be carried out down to the bedrock formation to obtain samples for geological sedimentary environmental identification and for carbon dating at various depths. This will allow further verification of the rate of sedimentation, types of sedimentation at various locations and the time of deposition within Peninsular Malaysia coastal soft soil deposits to be identified. Also the question of whether they can be divided into two layers can be better investigated. Geotechnical properties of these samples should also be carried out to support evidence obtained from the geological study.

ii. A detailed analysis of the geotechnical data presently available from several sites needs to be undertaken to develop a general properties database for Peninsular Malaysia coastal soft soil deposits.

iii. Specific laboratory studies on the influence of iron pyrites, carbonates, shells and organic content on the properties of Peninsular Malaysia coastal soft soil deposits, in particular the interaction of these factors with the particular group of clay minerals they contain will be useful, particularly the presence of montmorillonite.

iv. A laboratory study to simulate the lack of dissipation of excess pore pressure in the field during loading combined with a microfabric study of the changes in the soil structure during the loading.

v. A study of the field behaviour of lightweight or composite fill material on Peninsular Malaysia coastal soft soil deposit compared to the normal fill as are presently used in the construction of embankments.

vi. A detailed computer aided analysis using different computer programs to obtain the best analysis and more realistic values of excess pore pressures and lateral displacement for use in geotechnical designs for Peninsular Malaysia coastal soft soil deposits.

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