FOUNDATION **FAILURES OF LIGHTLY LOADED BUILDINGS ON THE SANDFILLED AREAS OF THE NIGER DELTA**

BY

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A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, AHMADU BELLO UNIVERSITY, ZARIA IN PARTIAL **FULFILLMENT** OF THE REQUIREMENTS FOR THE AWARD OF DEGREE OF **MASTER** OF SCIENCE (GEOTECHNICAL ENGINEERING)

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DECLARATION

I hereby declare that this thesis has been written by me and it is a record of my own research work. It has not been accepted in any previous publication for a higher degree.

All sources of information are specifically acknowledged by means of references.

ASUABIAT UDO EKPO

APRIL, 1998

CERTIFICATION

This thesis entitled "FOUNDATION FAILURES OF LIGHTLY LOADED BUILDINGS ON THE SANDFILLED AREAS OF THE NIGER DELTA" by EKPO, ASUABIAT UDO meets the regulations governing the award of the degree of Masters of Science in Geotechnical Engineering, and is approved for its contribution to knowledge literary presentation.

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DEDICATION

To Mr Marc Venturini, and to the evergreen memory of my beloved mother, late Mrs A S U EKPO whose sincerity has taught me to appreciate honest critics and whose lives have been and always will be emblems of total dedication and unparalleled faith.

ACKNOWLEDGEMENT

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ABSTRACT

The utilization of the hydraulic sandfilled areas of the Niger delta for residential and public buildings is limited due to the numerous problems encountered in the development of the areas. The alarming aspect of the problem is the appearance of cracks on lightly loaded buildings erected on the reclaimed areas of Borikiri in Port Harcourt and Ogolo-Ama in Okirika L.G.A.Rivers State.

In order to cope with the poor sub surface condition prevailing in the reclaimed areas of the Niger Delta, the field investigation embarked on includes, borehole drilling and penetration testing (CPT and SPT). The relevant laboratory tests (identification, classification, strength and consolidation) were carried out on the samples collected during boring. To enhance higher accuracy in the acquisition of geotechnical parameters of the underly weak clays, different methods of strength and compressibility analyses were adopted in this study.

The soil exploration revealed that the maximum thickness of the fill is about 2.5m in the investigated areas. Underlying the sandfill is a layer (3.0-7.5m) of organic clay, the clay layer is very soft to soft (< 40 kN/m^2) in consistency, high to very high in compressibility (1.21 - 2.73 m^2/MN) with numerous inclusions of decomposed plant debris. The permeability of this clay layer increased from low to high as the clay becomes sandy with depth. Below the organic clay lies a layer of predominantly loose alluvial sand.The ground water conditions (0.4 - 1.0m) below ground level at the time of the investigation were generally highly unfavourable for shallow foundation works.

The foundation problems of the lightly loaded buildings on the sandfilled areas can be attributed to the effect of developmental and environmental factors which include, the high

ground water level and its variations,the insufficient sandfill thickness, the decomposition of the mangrove vegetation in the organic clay, consolidation of the organic peaty clay, lapses in the reclamation process, inappropriate use of the sandfill areas etc. Solutions put forward in this study include; provision of means to overcome the adverse effect of the developmental and environmental problems prevalent in the sandfilled areas, improvement of the reclaimed land properties which covers some soil improvement methods and design of effective foundation types. Preloading is recommended as one of the soil improvement method. for its effective application, a design criteria is presented which gives the required surcharge thickness at a stipulated period.

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Chapter 1

INTRODUCTION

1.1 Background Information

The Niger delta, from which over 90% of Nigeria's foreign exchange earnings are produced through crude oil, is situated at the coastal sedimentary areas of Southern Nigeria. With the inclusion of areas of over 20 rivers that are partly connected with each other, the Niger Delta may be defined as the region between Imo River in the east and Benin River in the west, and covers an area of some 70,000 km^2, consisting of over 20,000 km^2 of wetlands (Rijkwaterstaat, 1989). Rivers, Delta and the newly created Bayelsa States, with an estimated population of 6.5 million people, constitute approximately 80% of the Niger delta area(Oguara, 1995).

As the world population continues to increase, rural-urban migration is on the rise, and there is increased demand for land in most urban areas. This situation is pronounced in the Niger delta of Nigeria where increase in petroleum exploration and its associated activities over the last few decades have promoted rural-urban migration particularly, Rivers State. Besides, more than 75% of the Niger delta area is seasonally or permanently flooded. The consequent congestion and pressure on land resulted in a government policy thrust on land reclamation which is mainly by hydraulic sand-filling.

The essence of any reclamation technique is the transformation of marginal land, to dry and usable land. Rutledge (1970) defined marginal land as a 'land that is unsuitable for development in its original conditions. Based on the above definition, it can be said that more than 80% of the land mass in the Niger delta falls under marginal land. This is attributed to the low and flat nature as well as the dense criss-cross network of rivers in the

region which render extensive portions of the land mass seasonally flooded.

Fubara (1982) pointed out that reclamation by hydraulic sandfilling is adopted in the Niger delta because of the relative abundance of sand in the area, the short time of completion of reclamation programme and the enhanced ground elevation in relation to sea level. Also, filling acts as a surcharge (Mitchell, 1981; Jones *et al.*, 1987) or a form of preloading of the soft compressive soil, a process which leads to the consolidation of the soft layer and eventual increase in bearing capacity of the natural soil. Sandfilling in the Niger Delta, since it was started more than a decade ago, has satisfied this splendid objective by providing usable land for the construction of residential buildings, roads and other facilities in the area.

Sandfilled areas like any other reclaimed land, are problematic in nature and should be utilized with caution especially where heavily loaded structures are located. However, this is not always the case, due to the ignorance of the people involved in the development of these areas, poverty and professional lapses on the part of the architects and engineers in charge of the development projects. For instance, soil investigation reports on these areas pointedly revealed that in most parts of the sandfilled areas, the sandfill thickness is too shallow to provide effectively competent foundation support for the proposed structures without overstressing the underlying weak soils. Such overstressing of weak soils inevitably causes foundation failures (Enoch George Associates, 1986 & 1989).

Reconnaissance survey of the sandfilled areas showed that insufficient sandfill thickness can also be attributed to inappropriete use of the sandfilled areas by the landowners and their developers. In the areas investigated, the fill materials were openly removed, and either use

for moulding blocks or sold out as construction materials for other developers. This reduced sandfill thickness hinders the elevation of a shallow foundation above the high water table associated with the sandfilled areas. Consequently, some parts of the sandfilled areas are still seasonally flooded, which renders them unsuitable for construction unless further soil improvement methods are applied.

This shortcoming in the implementation and utilization of the hydraulic sandfilled areas, has accounted for many foundation failures of structures found on the reclaimed areas. Nevertheless, there exist cases of failures of structures which are due to causes like inadequate designs, low quality materials and poor construction methods. Majority of the distressed structures, identified by deep cracks on them (plate i-x), are those subjected to excessive foundation settlement either total or differential. In this work, foundation failure of building found on the sandfilled areas of the Niger Delta will be investigated in order to identify their causes and to furnish ways of preventing them.

1.2 Location And Geology Of The Niger Delta

1.2.1 Location

The Niger delta lies at the southern part of Nigeria bordering the Atlantic Ocean (Figure 1.1) and extends from about longitude 4E 15' to 8E 30' E and latitude 4E 30' to 6E 30' N (Asseez, 1976)

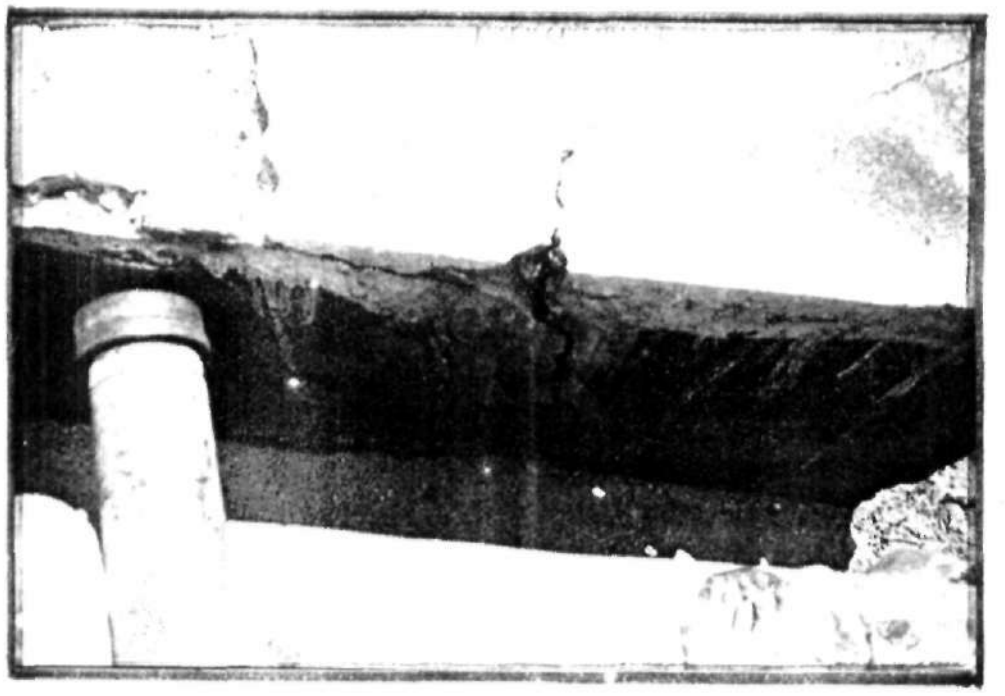


Plate (i): Distressed storey building in Ogolo-Ama sandfilled area, tilted towards one direction with deep cracks on the ground slab and walls.

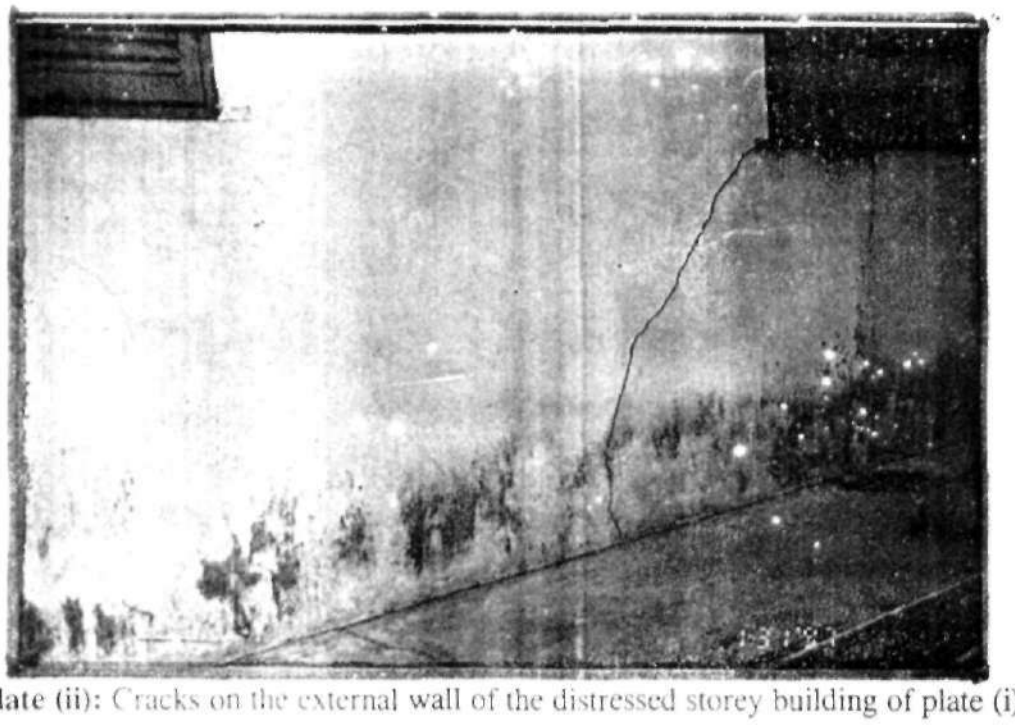


Plate (ii): Cracks on the external wall of the distressed storey building of plate (i).



Plates (iii-iv):

Different views of a distressed bungalow in Borikiri sandfilled area, tilting backward with continuous deep cracks on the floor and walls despite several repair works. The frequent occurrence of cracks have hindered finishing work for over six years of completion of building.

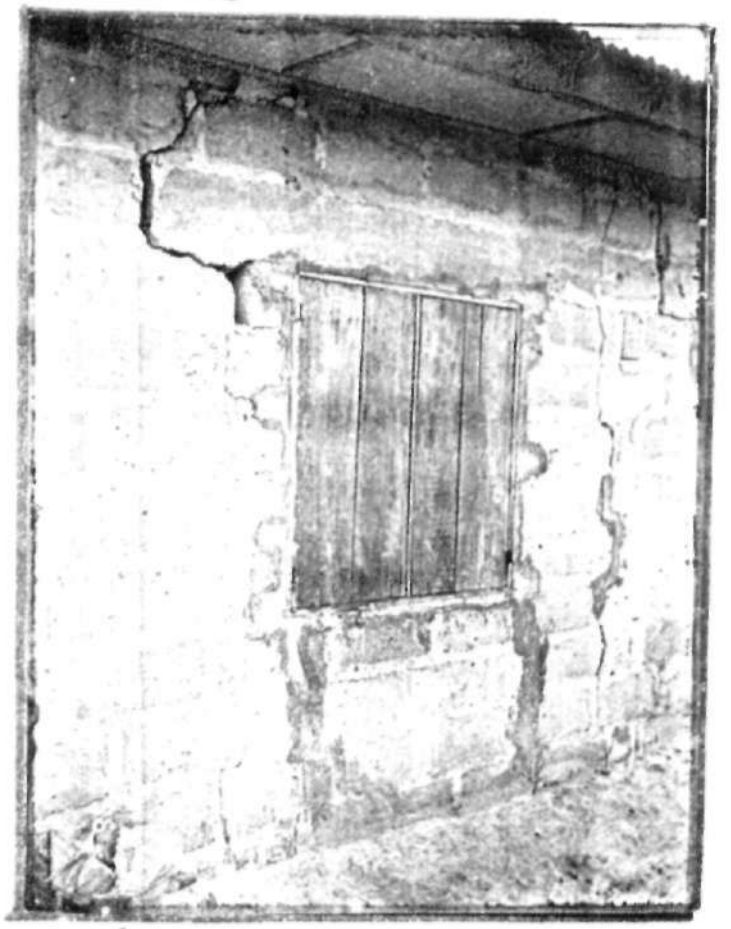
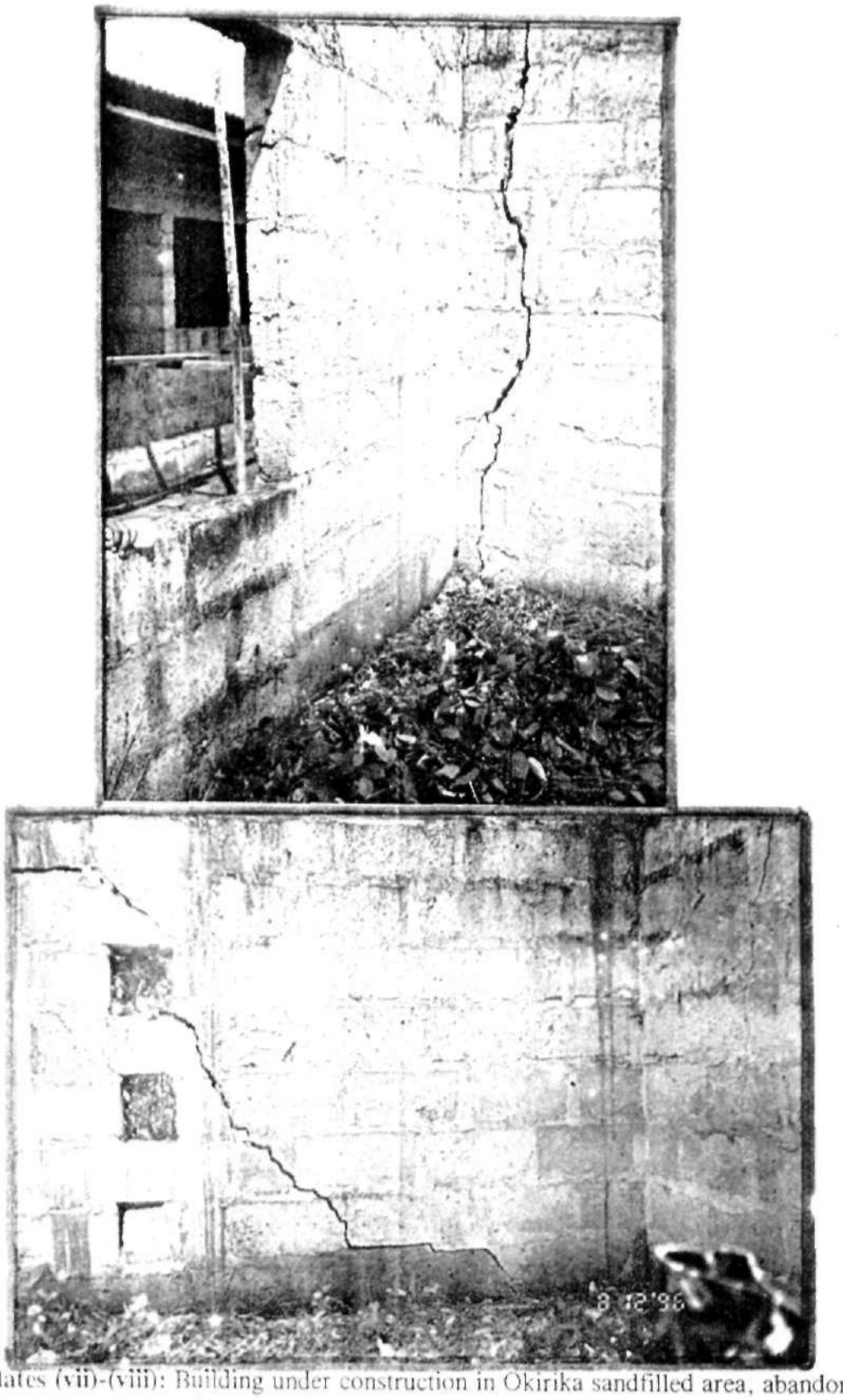




Plate (v): A distressed classrooom block in Ogolo-Ama sandfilled area, tilting towards one direction with deep cracks on the bearing walls and the drain rails.



Plate (vi): Cracks on the laboratory floor slab of the distressed classroom block in Ogolo-Ama secondary school.



Plates (vii)-(viii): Building under construction in Okirika sandfilled area, abandoned due to excessive settlement and deep cracks on the load bearing walls.



Plate (ix): An abandoned bungalow in Ogolo-Ama sandfilled area due to excessive differential settlement and cracks all over the building



Plate (x): Shows a cracked fence in Ogu sandfilled area in Okirika, Rivers State .

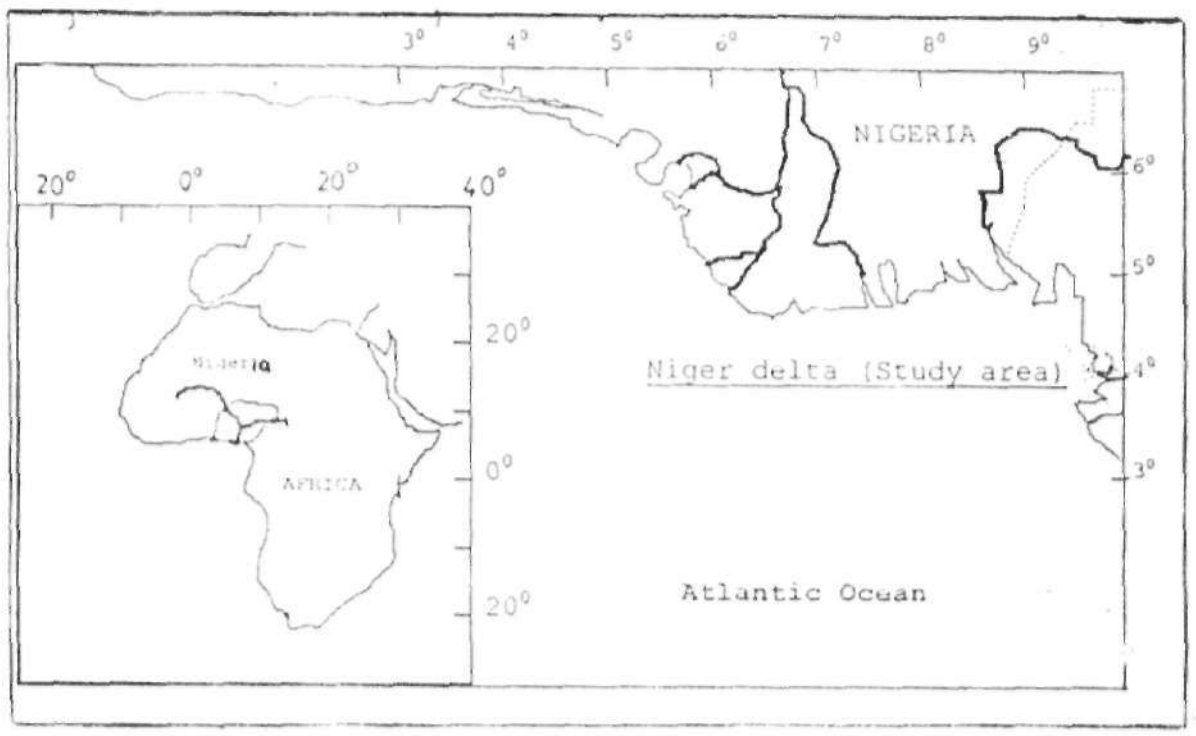


Figure 1.1 Location Map of Niger Delta (Ejesie and George, 1987)

1.2.2 Geology

Our present knowledge of the geology of the Niger delta was derived from the works of Reyment (1965), Short and Stauble (1967), Murat (1970), Merki (1970), the exploration activities of the oil Companies and the Niger Delta Basin Development Authority (NDBDA). Weber (1971) suggested that the basin which enhanced and controlled the development of the present day Niger delta developed by rift faulting during the PreCambrian era. The modern Niger delta was formed during the third and last

depositional cycle of the southern Nigeria basin which started in the Palaeocene age.

However, the present form of the Niger delta contains thick sedimentary accumulation built by physico-geological processes of rivers. Sediment thickness is about 12.5 km. (Merki 1970, Asseez, 1976). Although various morphological units arising from complex sedimentary process have been identified in the Niger delta, George and Abam (1990), the major units comprise meander belt, deltaic plain, fresh water and back swamp deposits as shown in Figure 1.2. Others are mangrove swamps, estuaries, beaches and bars. These latter units occur in the fluvio-marine transitional environment. A significant proportion of the Niger delta lies within the meander zones, where progressive meandering has resulted in channel switching abandonment, leading ultimately to the development of buried channels.

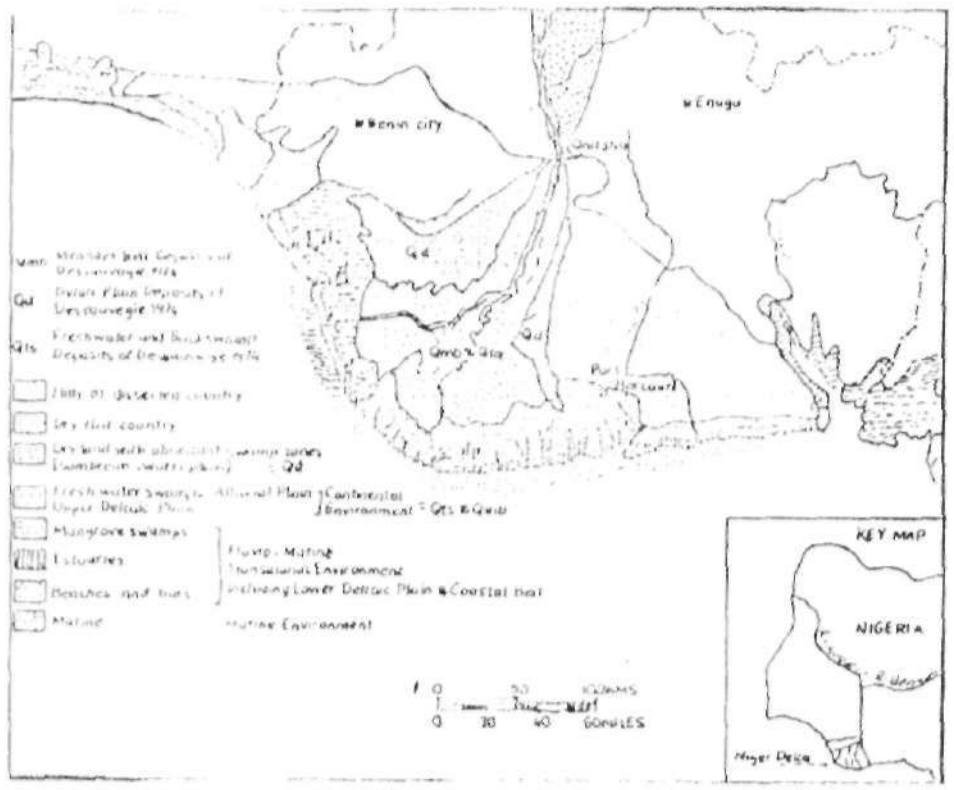


Figure 1.2: Map showing Morphological Soil Groups in present day Niger delta complex (Short & Stauble, 1967)

It is also a problem when drainage facilities are constructed. In excavation works not only does it present difficult (muddy) working conditions, seepage water faces affect the stability of sides and base level of the excavation. Any structure founded in such excavations will be subjected to the uplift pressure from pore water forces as well as flooding. To further compound the problems, the water table varies widely with highest depths reached during the wet season, while in the coastal and mangrove swamp environment the ground water levels also vary with the tidal water in the creeks and rivers.

The Niger delta has a tropical climate. Two distinct climatic seasons are observed annually as reflected on a typical evapo-transpiration and concurrent precipitation for the Niger delta as shown in Figure 1.3.

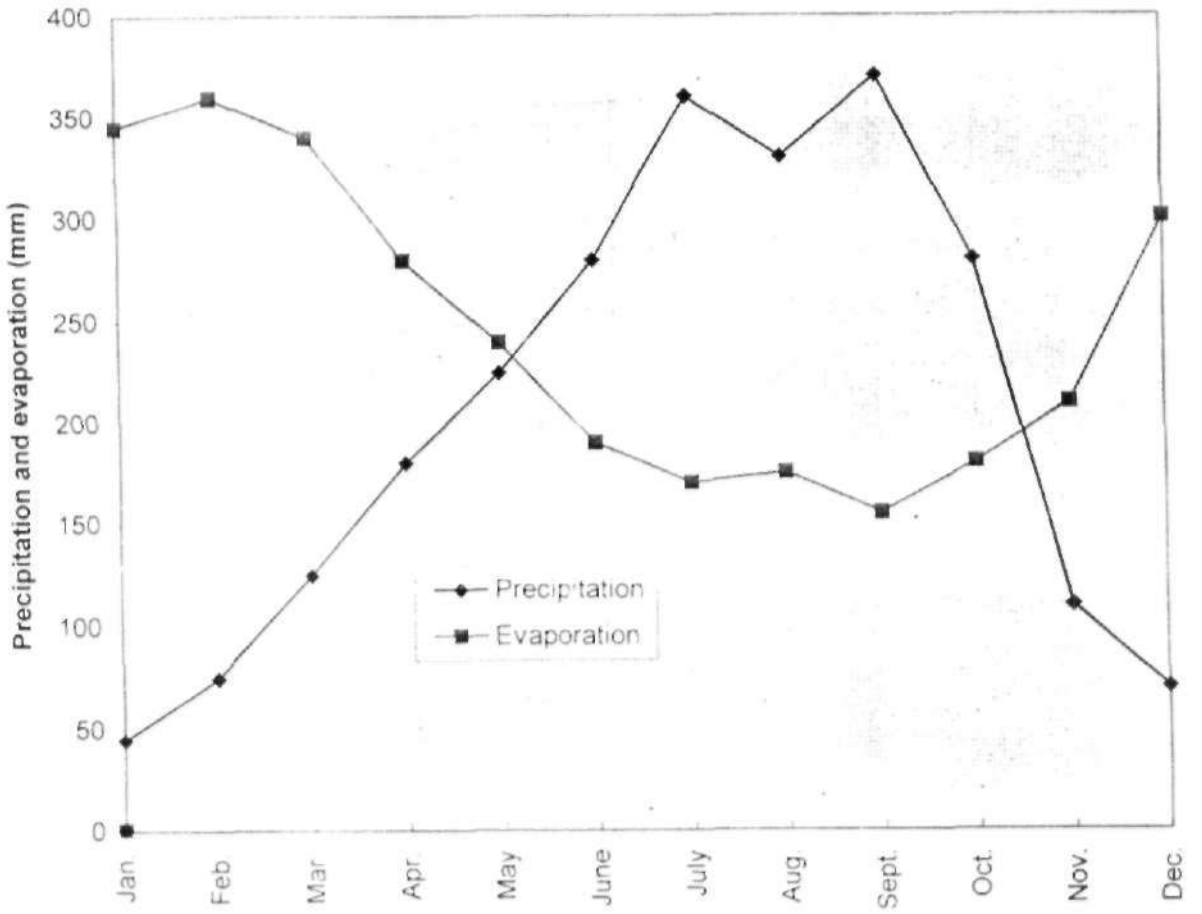


Figure 1.3 :Average Monthly Precipitation and Evaporation Graph (1960-1980) for Niger Delta (after George and Abam, 1990)

Rainy season spreads through a period of nearly eight months (April to October) while the dry season lasts for about four months (November-March). The annual mean rainfall is very heavy. In Brass and Bonny for example, the annual mean rainfall measured 4,215 mm, while the annual mean evaporation is about 1,490 mm (Korea, 1980). The annual mean rainfall decreases gradually inland to about 3,000 mm at Peremabiri and Oloibiri and 2,500 mm around Port Harcourt and Warri, as presented in Table 1.1:

Table 1.1 – Mean annual rainfall selected Locations in the Niger Delta (after Akpokodje,1987)

Location	Recorded period (Year)	Mean annual
Bonny	45	4366
Opobo	52	3698
Brass	11	3573
Forcados	52	3739
Oloibiri	24	3173
Warri	32	2880
Degema	51	2433
Port Harcourt	28	2394
Patani	16	2073

During the dry season soil moisture deficit frequently occurs resulting in shrinkage and the formation of tension cracks. The precipitation due to rainfall introduces water into the dessiccated soils resulting in swelling and further deterioration of the engineering properties of the soils.

Ogunara (1989), emphasised that the high monthly rainfall also implies high in-situ moisture content or near saturation for the soils thereby reducing their geotechnical qualities and making the foundation treatments involving compaction difficult. The ground water conditions in terms of depth of water below the existing ground level for the various zones range from:

0 to 1.0 m for coastal zone
0 to 2.0 m for mangrove zone
0 to 4.5 m for fresh water zone.

on the average the depth of water table is about 2.5 m below ground level.

This is quite unsafe for both excavation works and construction of foundation for building.

There is a wide distribution of swamps in the area caused by the abundance of near surface silt and clay deposit, a general flat topography and consequent insufficient drainage capacity. Also, most of the rivers are actively eroding, scouring and cutting their banks, thus exposing more land mass to flooding.

The sandfilled areas are located in the mangrove and fresh water swamp environment, particularly, in the coastal towns close to the edges of the rivers and creeks. Filling is done hydraulically using dredged sand from the rivers.

1.3 Justification of Study

The need for an investigation into the causes of foundation failure of lightly loaded buildings found on the reclaimed areas of the Niger delta cannot be overstressed. Increase in population promoted by oil exploration activities in the area, requires a corresponding housing development project to accommodate the people. Irrespective of the increase in population, the total land surface will always remain unchanged. The prevailing congestion and pressure on land can only be solved through utilisation of reclaimed marginal lands.

It is an attempt to provide possible ways of overcoming foundation failures in the sandfilled areas of the Niger delta particularly Port Harcourt, and the need to show that a shallow foundation can be placed successfully on the sandfill, if geotechnical techniques are adhered to during the development of these areas.

A successful execution of this study will go a long way to show the community that the government policy to embark on hydraulic sandfill is an avenue for urban and rural

development but not a financial waste as some of the riparian owners are beginning to view the entire project. Besides, with the availability of the reclamation material (sand) and the suitability of the soil conditions in the Niger delta to reclamation by hydraulic sandfilling, all the marginal lands in the area will be reclaimed by hydraulic sandfilling method. Thus, investigation into the causes of failures of structures found on these reclaimed areas is very essential as it will facilitate adequate utilisation of these reclaimed lands.

1.4 Aim and Objectives

The main purpose of this research work was to investigate the possible causes of foundation failures of lightly loaded buildings found on the sandfilled areas of the Niger delta, particularly Port Harcourt, with the view to identify the causes. In order to achieve this goal, the study was designed to achieve the following objectives:

(i) To carry out an intensive study into the geological histories and their effect on the geotechnical properties of the sandfilled areas.

(ii) To carry out a detailed geotechnical investigation of subsoil at developed sandfilled sites with structures subjected to foundation failures, in order to understand the mechanism involved in the failure characteristics of the superstructures

(iii) To determine the stratigraphy of the subsurface deposits underlying the site.

(iv) To carry out settlement analysis by utilising different methods of determining strength and compressibility characteristics of the weak soil underlying the sandfill material. This comparison of results will enhance accuracy.

(v) To recommend measures that can be used by a designer to adequately use the reclaimed areas for construction purpose.

1.5 Scope of Research

The range of activities in this study was limited to the sandfill areas of the Niger delta, particularly Rivers State. Structures under consideration in this work are lightly loaded buildings. Special attention was given to buildings subjected to foundation failure. In order to cut down the huge financial expenses involved in the collection of soil samples at deeper depth in the sandfilled areas of the delta actual detailed investigation was only implemented on two typical sandfilled sites, within the vicinity of the distressed buildings. For the remaining sandfilled areas, the geotechnical soil investigation reports reviewed in the literature were utilised for the assessment of the subsoil characteristics. Also, trial pit System was be adopted where possible for more extensive coverage of the reclaimed areas. Actually, as mentioned earlier, there may be other forms of building problems that emanated from structural, construction method and material quality. For the purpose of this study only the causes of foundation failures of lightly loaded building were investigated. The focus of the study was on the mangrove swamp environment where majority of the hydraulically sandfilled areas in Rivers State are situated.

Chapter 2

LITERATURE REVIEW

2.1 Clarification of Terms

The clarification of the following terms is necessary for the basic understanding and appreciation of this work:

(i) Foundation: is a collective term for a system of structural elements through which structural loads are transmitted to underlying soils and rocks. The criteria commonly used to evaluate foundations include suitability of location, safety from failure and freedom from objectionable deformation.

(ii) Structural Failure: The object of structural design is to ensure that a structure does not fail, either functionally or ultimately during its life span; and serves its intended function. Functional failure refers to a serviceability limit state defined by BS 8110 (1990) as a state which the structure has developed excessive deflation, cracks and has generally become unsightly. Ultimate failure refers to a state in which the structure is collapsing or near collapse.

The code of practice BS8110 provides a guide to what are considered 'reasonable' limits of cracks widths. These acceptable limits are:

(a) the surface crack widths, when assessed, should not in general exceed 0.3mm; (b) in case where the environment can be considered aggressive (e.g. Seawater) the assessed surface crack widths at points nearest the main reinforcement should not exceed 0.004 times the nominal cover to main reinforcement.

(ii) Reclamation: is the operation or process of changing the condition or characteristics

of land which cannot be utilised to its highest effectiveness, so that improved utilisation can be achieved. This may be accomplished by irrigation of arid lands, drainage of swamp or waterlogged land, protection of flood menace of land constantly subject to overflow etc. (Vollner, 1967). However, in this work, the term reclamation is described as the act of raising the level of land which is either just below or adjacent to water or the creation of new land from coastal fringes or in tidal estuaries or areas covered either continuously or intermittently by water (Bray, 1979; Westbury, 1976). The land is effectively being reclaimed from water and swamp. Also, in the Niger delta, reclamation extends to the hydraulic sandfilling of the coastal banks for beach replenishment, shoreline protection in order to control eroding, scouring and cutting of the river banks and to provide competent base for jetties and harbour construction.

(iv) Delta: a predominantly alluvial deposit built out by stream into the sea or other body of water. It may be of wide extent and usually have a typical form of the Greek letter delta (Δ) owing to splitting of the rivers into diverging branches or distributaries. Most delta are partly above and partly below water. The land ward portion is usually widely covered in times of flood and is made up of fluvial deposits. The seaward beds, covered by water at all times during formation, are classed as marine or lacustrine (Stroke and Varnes, 1955).

2.2 The Niger Delta Soils

The Niger delta is a topographically low, periodically inundated and virtually flat land comprising mainly of comparatively weak and highly compressive alluvial soils (George and Abam, 1990). The soils of the Niger delta are generally fine grained and their engineering properties revealed that they are essentially clays with varying proportions of

sand, silt and organic matter. The clays, silt and organic contents increase seawards. The value of the various engineering parameters are mainly influenced by the amount of clay (and/or sand) and organic matter.

Oguara (1995) states that the present form of the Niger delta is typified by highly variable soils. The variability being marked by erratic horizontal and vertical distribution patterns of rivers. The surface is generally characterized by a profile of very compressible clay to the top underlain by silty and sandy strata. Further down, the texture becomes predominantly sandy and gravely. The stratal boundaries, in many cases, are not well demarcated. Nevertheless, available evidence indicates that the delta clays may extend to a depth greater than 4m in some places. In general, the soil of the Niger delta, mainly consist of sandy clay with very low coefficient of permeability.

Akpokodje *et al* (1986) proposed that in order to be able to define, understand and solve geotecnical design and construction problems of a particular site in the Niger delta, one must know:

(a) the relationship between the present and past form of the site

(b) the present/past geological and geomorphological processes.

Based on the above mentioned criteria, the authors classified the superficial soils of the Niger delta into four major groups, namely:

(i) Reddish brown sandy clay loam soil of low to medium plasticity (RBSCL-1)

(ii) Brownish sandy clay (BSC-2)

(iii) Light grey slightly organic fine and silty clay (LGFSC-3)

(iv) Dark organic/peaty clay of high and extremely high plasticity (DOPC-4)

It is important to note that Akpokodje's (1986) classification does not include the active/abandoned beach deposits along coastline and channel/meander sand bars and natural

levees associated with the major rivers and creeks. Furthermore, the author pointed out that the LGFSC-3 and DOPC-4 soil groups can also be found within the other two main soil groups where marshes and seasonally flooded swamps occur. The soils are usually acidic in reaction (pH 5.2-6) with traces of sulphate in the LGFSC-3 and DOPC-4 soil groups and an appreciable amount of chloride in the DOPC-4 soil group only.

Furthermore, Akpokodje (1987) worked intensively on engineering properties of the superficial soils of the Niger delta. Based on his findings, the ranges of variations of the geotechnical properties of the superficial soils within the four major groups are as presented in Table 2.1.

Base on his findings, the author stated that the values of the Skempton activity of the soils range from 0.53 to 2.78 which classify the soils as inactive, normal and active clays, with the high values for DOPC-4 soil group. The high activity of the DOPC-4 soil group may be due to the presence of organic matter and montmorillonite which can be easily formed in the poorly drained environment. The relatively low (less than 0.82) colloidal activities of the three other soil groups is in line with the kaolinite-dominated composition of these soils.

Also, the consolidation characteristics of the various soil groups can be explained by a combination of particle size, the type and quantity of clay/organic matter, the in-situ bulk density and the permeability. From the result of the consolidation tests, it can be said that settlement of structures founded on RBSCL-1 and BSC-2 soil groups will be small and rapid whereas those on the LGFSC-3 and DOPC-4 soil groups will be large and slow.

Table 2.1: Summary of the **Geotechnical Characteristics of the various soil groups** (Akopodje, 1987).

Particle Size, Moisture Content and Plasticity					
Soil Type	Percentage of fines (silt + clays)	Percentage of Gravel	Organic Content (%)	Moisture Content (%)	Plasticity Index PI (%)
RBSL - 1	NP	NP	NP	NP	LM
BSC - 2	NP	NP	NP	NP	LM
LGSEC - 3	81 - 90	NG	SO	NP	LH
DOPC - 4	91 - 100	NG	10 - 40	>100	54 - 150 (EH)

Strength Characteristics			
Soil Type	Skemton Activity	Angle of Internal Friction (°)	Undrained Cohesion kN/m²
RBSL - 1	0.53 - 0.82	NP	NP
BSC - 2	< 0.82	NP	NP
LGSEC - 3	< 0.82	3 - 20	9 - 28
DOPC - 4	0.82 - 2.78	0 - 5	7 - 20

Consolidation Characteristics

Soil type	Number of samples	Bulk density (kg/m³)	Mv (m²/MN)			Cv (m²/year)		
			Pressure ranges (kN/m²)			Pressure ranges (kN/M²)		
			50-100	100-200	200-400	50-100	100-200	200-400
RBSCT-1	12	1642-2141	0.346-0.512	0.301-0.500	0.279-0.421	59.67-115.59	35.71-95.23	19.97-45.15
BSC-2	10	1550-2064	0.457-0.689	0.560-0.801	0.291-0.685	35.08-40.78	28.27-34.29	27.82-29.95
LGSEC-3	15	1272-1771	0.312-1.075	0.311-0.952	0.291-0.721	20.24-30.45	15.72-26.84	10.95-25.79
DOPC-4	10	954-1547	0.226-1.79	0.128-1.541	0.099-1.541	0.752-22.484	0.631-19.534	0.345-14.115

Note: LM - Low to Medium
LH - Low to High
NG - Negligible
NP - Not Presented

2.3 Subsoil Characteristics Of The Sandfilled Areas

2.3.1 General consideration

Despite many decades of land reclamation by hydraulic sandfilling in Nigeria, published work related to the soil characteristics of the sandfilled areas of the Niger Delta is very scanty. However, in the Niger delta extensive soil investigation programmes to ascertain the suitability of these areas for development had been implemented both by the State Government and private firms. Prominent among such soil investigation reports are those presented by Enoch George Associates (1986, 1989, 1993 and 1994), Trevi Foundations (1993 and 1994) which covered almost all the sandfilled areas reclaimed between 1979

presented by Enoch George Associates (1986, 1989, 1993 and 1994), Trevi Foundations (1993 and 1994) which covered almost all the sandfilled areas reclaimed between 1979 and 1985 in Rivers State.

The existing published works and the mentioned soil investigation reports jointly disclosed the following:

(i) the sandfilled areas are situated in the transitional environment of the Niger delta; sandfilling is carried out in the coastal towns close to river banks and creeks, and also on the swampy areas of fresh water and mangrove swamps environment.

(ii) the sandfilled areas are made up of three distinct soil strata, namely the sandfilling material extremely variable in thickness, overlying weak and highly compressible organic clays. Under the organic clays there is loose to medium dense alluvial sand.

(iii) the ground water condition in these reclaimed areas is very high and extremely variable with tidal variations.

The average depth of the geotechnical investigation as estimated from the soil reports is 25.0 m below ground level. A typical geotechnical soil profile in relation to the stratigraphic sequence is shown in Fig. 2.1.

The geotechnical and construction properties of the subsoil characterising the sand-filled areas are as presented in the next subsection.

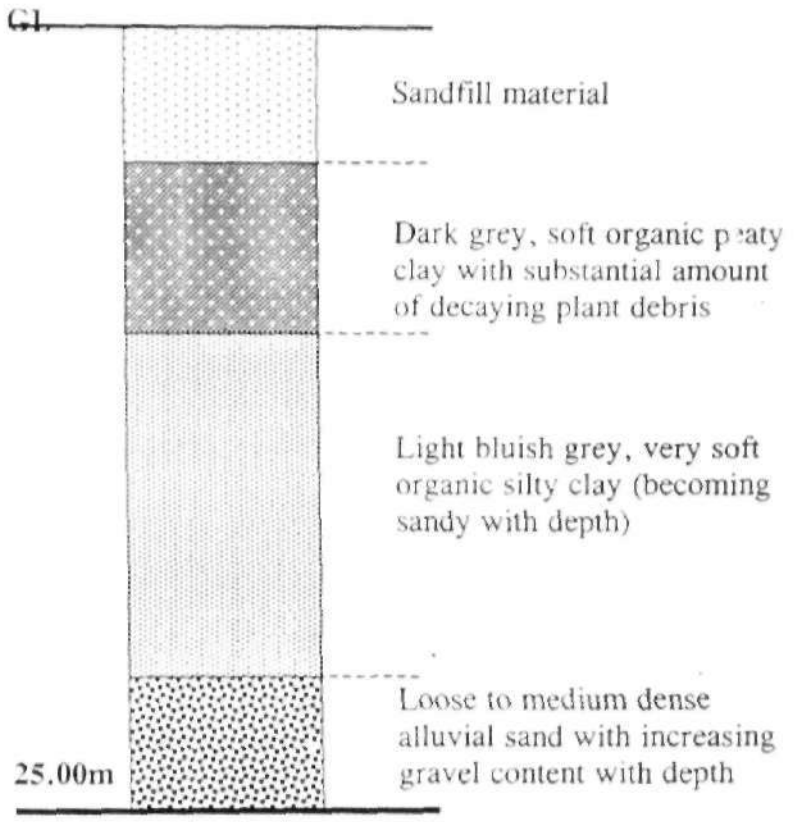


Figure 2.1 Typical Subsoil Profile of the Reclaimed Areas of the Niger Delta (not to scale).

2.3.2 Material used for sand-filling

The fill material was placed hydraulically on the sites more than a decade ago using dredged sand from the rivers ocean and creeks. The thickness of the sand-fill is extremely variable within a given site and from one sand-filled location to another.

In some sand-filled areas it is very common to find some portion of the land devoid of the sand-fill material. The maximum average sand-fill thickness recorded in all the areas investigated is about 2.5m. The sand-fill material is generally clean, fine to medium coarse sand in a predominantly loose state of compaction. However, few areas showed the sandfill material to be medium dense to dense in relative density (Enoch George Associates, 1986).

According to BS 1377(1990) the sandfill material is uniform poorly graded sand content is greater than 75%, gravel content 4 to 15% while the silt/clay content is often less than 5%. In the majority of sandfilled areas, lumps or pockets of organic clays are found in the fill material especially close to the weak soils. The soil investigation reports also revealed that the sandfill material in most of the areas is too shallow to provide competent foundation support for the proposed structures without overstressing the underneath soft organic clays. Furthermore, it was also pointed out that the high ground water level which is literally standing at the surface of the ground hardly favours foundation works.

2.3.3 Weak and saturated soils of the sandfilled areas

In general, the sand-filled material is underlain by weak and highly compressive clays in all the reclaimed area investigated in the Niger delta. The thickness of this stratum varies considerably through any given site and from one sand-filled area to another. The strata boundaries, in many cases, are not well demarcated, hence the thickness is not easily determined to a high degree of accuracy. Nevertheless, available evidence indicate that the least thickness of the weak soils is about 3.0 m in the areas investigated while the maximum thickness of this clay layer so far presented is about 23.0 m below the sand-fill material (Trevi, 1993; Enoch George Associates, 1994). The weak soil deposit is generally very soft to soft in consistency, extremely compressible and highly plastic with significant abundance of decaying vegetation. The decayed plant vegetation content decrease with the increase in depth and is more pronounced in some sand-filled areas. The soft clays generally have shear strength values less than 15 kN/m^2 (George and Abam, 1990), thus are not suitable as foundation soil.

The values of the geotechnical parameters of these weak clay soils are mainly influenced by the amount of sand, organic matter and the geomorphological process at work in the

sand-filled areas. However, in the Niger delta, hydraulic sand-filling is carried out both in the fresh water swamps and the mangrove swamp environment. The engineering properties of the weak soils peculiar to the two geological zones are as presented below.

(i) **Fresh water zone**

For the weak soils in the fresh water zone, they are chiefly expansive soils; the major problems is that of swelling and shrinkage. The weak soil, which is silty clay, responds rapidly to seasonal volume changes. During the dry season, the clay material shrinks and hardens considerably with remarkable gain in shear strength (undrained shear strength up to 180 kN/m^2) and large angle of internal friction up to 20° (Oguara, 1995). This normally leads to tension cracks and weak undulated clay aggregates in the areas that are not sandfilled. However, on full saturation during the wet season, much of this strength is lost due to swelling and subsequent reduction in effective stress. Such seasonal variations cause appreciable damage to structures found on this weak clay soils.

(ii) **Mangrove swamp environment**

In the mangrove swamp environment, on which this study is focused, the weak soils encountered in the sand-filled areas are of two categories, namely the dark, soft organic peaty clay with substantial decomposed plant debris "Chikoko mud". Below the "Chikoko mud" lies the deposit of light bluish grey, very soft, silty clay with or without decomposed plant debris generally designated as "Marine mud". These clay deposits commonly have high natural moisture content (above 100%) and liquid limits greater than 50% and are very permeable with large coefficient of consolidation and volume compressibility indicating high rate of consolidation and excessive settlement (Oguara, 1995). Semeriwarima (1991) gave the properties of "Chikoko mud" samples obtained from three coastal towns situated in the mangrove swamp environment in Rivers State as summarised in Table 2.2:

Table 2.2 **Geotechnical Properties of "Chikoko Mud" Samples Obtained from Three Coastal Town Located in the Mangrove Swamp Environment of the Niger Delta (after Semeriwarima, 1991)**

PROPERTY	LOCATION		
	Ogu Okirika	Marine Base (P.H.)	Eagle Island (P.H.)
Clay (%)	16.1	9.83	11.98
Silt (%)	20.9	40.04	29.10
Sand (%)	63.0	50.13	58.92
Moisture Content (%)	160.8	119.0	360.5
Liquid Limit (%)	66.25	74.5	82.2
Plasticity Index (%)	31.82	37.6	14.8
Specific Gravity	2.01	2.0	2.02
Linear Shrinkage	7.14	7.16	7.86
CBR(%)	49	58	34
Max. Dry Density Kg/m^3	1272	1284	955
Optimum Moisture Content (OMC) (%)	24.8	25.5	41.2
Permeability K (m/sec x 10^{-8})	5.92	7.93	7.95
Coeff. Of Volume change Mv (m^2/kN)	0.67	0.084	0.068
Cv (mm^2/min)	1.047	0.331	1.51

while the summary of geotechnical characteristics of the light grey organic silty clays for a typical sandfilled area is as presented in Table 2.4.

The very soft, light grey organic silty clays becomes predominantly sandy and gravely with depth. In most sandfill areas, only one of these clay deposit is present; where the two soft clean layers are encountered, "Chikoko mud" overlies the "Marine mud".

2.3.4 Loose to medium dense alluvial sand

Both the standard penetration test and the cone penetration test are used extensively in the Niger delta, to determine the engineering properties of these deposits. This deposition underlie the light grey organic silty clays in the sandfilled sites investigated. The stratum is predominantly loose to medium dense in its state of compaction. The density of this natural sand layer as well as its' gravel content increases with depth. In relatively few

areas, clayey strata underlie this deposit (Enoch George Associates, 1988 and 1989). The loose to medium dense natural sand deposit is often found at higher depths hence only deep foundations on piles or caissons are the only means of using the stratum as foundation soil.

Table 2.3 Summary of the Geotechnical Properties of the Soft, Compressible, Weak, Organic Silty Clay from a Typical Sandfilled Area in Rivers State (Enoch George Associates, 1989)

PROPERTY	Range of Values		
	Min	Max.	Average
Natural Moisture Content (%)	62.0	147.0	119.0
Liquid Limit (%)	92.0	174.0	146.0
Plastic Limit (%)	48.0	80.0	70.0
Plastic Index	38.0	103.0	80.0
Bulk Unit Weight (kg/m^3)	12.0	15.4	13.60
Undrained Shear Strength (kN/m^2)	6.0	22.0	21.0
In situ voids ratio	.60	1.20	.90
Change in voids for D = 25 kN/m^2	.02	0.37	.20
Laboratory Vane Shear Strength (kPa)	6.0	23.0	22.0
Coeff. of Volume change Mv (m^2/kN)	1.70	1.80	1.60
Cv (mm^2/min)	0.21	2.60	2.00
Permeability	$1\ E^{-8}$	$1\ E^{-8}$	$1\ E^{-8}$

2.4 Foundation on Hydraulic sandfill with underlying Weak Soils

Usually, the design of foundation on any type of land should meet two fundamental requirements if it is to perform satisfactorily. These are :

- That the load transferred by the foundation from the superstructures should not result in the excessive stressing of the soil to cause shear failure of the soil below the foundation. This criterion implies that there should be no shear failure of the soil, and is known as the Bearing Capacity Criterion

That compressibility of the soil should not result in excessive differential settlement

between different segments of the structure. This is known as the Settlement Criterion.

Thus, in designing a suitable foundation, the behaviour of the foundation with regards to these two criteria and the effect of environmental factor such as variation in moisture content on these criteria are considered. However, on normal dry land, these criteria are satisfied with ease by the application of certain foundation design principles. But in the case of hydraulic sandfilled areas, extra precaution have to be incorporated in order to overcome the numerous problems usually encountered in the development of the reclaimed land.

George and Abam (1990) proposed two alternatives open to engineers in charge of the development projects in the sandfilled areas of the Niger delta. These options are:

I) To avoid seating foundations within the vulnerable depth in the clay zone.

II) To prevent infiltration of water into the vulnerable depth within clay soils.

Due to the high water level in the ground and in the criss-crossed rivers and creeks, construction activities in the Niger delta involving excavation invariably encounter problems of water influx especially during wet season. For the same reason, these authors recommended that the minimum sandfill height in the reclamation project in the areas must exceed the maximum expected flood level, basically not less than 3.0 m above the weak soil.

Osinubi (1995) proposed the selection of rational type of foundation on hydraulic sandfilled areas with underlying weak soils. He emphasised that construction on hydraulic sandfilled bases with underlying weak soils has its peculiarities and he based his proposal on the fact that, the weak soil being of any type, the most important thing is that its compressibility considerably differs from that of the overlaying sandfill material

as well as the lower less compressible alluvial soil. The ratio, in terms of the thickness of the sandfill material and its underlying weak layer may differ. Thus, in his recommendation, it was stressed that the use of sandfilled material as the bearing layer for foundation of building leads to the reduction of compressible stress transmitted to the underlying weak soil. This implies that the greater the thickness of hydraulic sandfill material between the foundation footing and the surface of the weak layer, the more rational the decision taken to use the sand fill as bearing layer.

Furthermore, Osinubi(1995) from settlement analysis of buildings on sandfilled lands underlying weak soils, proposed the use of strip foundation, raft foundation and piled foundation and also went further to provide the conditions and the design modifications under which each of the options could be adopted.

In addition, the soil investigation reports (Enoch, George Associates, 1989, Abam and Okogbue 1993) on the sandfilled areas of the Niger delta, jointly disclosed that the sandfill thickness is too shallow and variable, the organic clays are too weak and highly compressible to support significant foundation loads. Soil improvement methods, such as placement of additional sandfill thickness on the site are often recommended before the use of shallow foundation. For heavy structures, the authors emphasised on the use of pile foundation to preclude the effect of the excessive settlement of the organic clays.

2.4.1 Foundation problems in the sandfilled areas

Flooding and consolidation settlement are the most important engineering problems militating against the development of marginal lands in the Niger Delta area (Oguara, 1989). The problems derive directly from the geological and generally flat topography of these deltaic environments. Consolidation problems are often associated with weak and

compressive soils, thus in the reclaimed areas under investigation, consolidation characteristics of the underlying weak soils may be the major problem. However, there are other factors that may contribute to the prevailing problems in these areas.

In the Niger delta, sandfilling is carried out hydraulically using dredged sand. Several problems are associated with hydraulic filling. Abam and Okogbue (1993), presented the following as some of the problems:Unreliable estimation of the height/amount of fill required to bring about enough consolidation and maximum strength achievement.

(ii) Hydraulic transport and deposition of materials generally results in relatively loose and soft fill with high moisture content, a problem that is complicated by the high level of water table in the delta.

(iv) As the supension flows away from the hydraulic discharge point, the large soil particles settle out at a much greater distance from the discharge point. This results in segregation of the deposited particles especially when a single point is used. Such a segregation could lead to differential settlement of foundations.

Furthermore, it was pointed out that fresh hydraulic fills are loose and waterlogged with their engineering properties being generally poor. It was also suggested that some improvement technique is necessary on the fill prior to construction of permanent facilities. Besides, from reclamation procedure, hydraulic fills of predominantly sandy nature must be placed in such a way that the small proportion of fine material are not allowed to collect into mud pocket in the reclamation areas (Bray, 1979). To avoid this problem, it is often advisable to start the reclamation operation from land towards the rivers or sea in order to enhance sufficient drainage. However, this principle was not given enough provision because the engineering properties of the sandfill material is further reduced due to the existence of pocket of organic clays in the sandfill material.

However, in the Niger delta, to evaluate the suitability of the reclaimed area of Borikiri for development, Enoch George Associates, 1989 investigated the consolidation characteristics of weak and compressible clays underlying the Borikiri sandfilled area. In order to ensure an even coverage in the exploration, the site was subdivided into seven zones. Based on the findings, the consolidation settlement results is as shown in table 2.4.

Table 2.4: Predicted settlement at different zones of the Borikiri stie (Enoch George Associates, 1989).

Zone	Range of Thickness of Sandfill (m)	Ground water level (m)	Predicted Primary settlement (m)	Time of 90% settlement (yr)
1.	0 - 1.0	0.2 - 1.8	150 - 400	0.5 - 5
2.	1.0 - 1.3	0.5	200 - 300	0.5 - 5
3.	2	1.3	75 - 300	0.5 - 5
4.	0 - 1.5	0.6	275 - 800	2 - 24
5.	1.0 - 2.5	0.3 - 0.7	100 - 350	1 - 14
6.	0 - 1.5	0.2 - 0.5	200 - 700	1 - 14
7.	0 - 2.5	0.3 - 0.7	200 - 400	1 - 6

Based on findings (table 2.4), Enoch Associates advised that soil improvement methods should be embarked on in parts of the sandfilled areas before development to preclude excessive settlement of the expected residential buildings.

However, the work so far reviewed in the preceding sections, remarkably portrayed a typical sandfilled area in the Niger delta as a man made land, with stratal successions: sandfill material, weak and highly compressible soils, and alluvial sand respectively.

From its geological origin, the area is also characterised by high water table which responds rapidly to tidal variations in the region. Thus, it can be inferred from the theoretical background, that sandfill thickness, thickness of the organic clays and their compressibility, the high ground water levels and their variations, become the determining factors in the settlement analysis and foundation design of structures found

on these reclaimed lands. Furthermore, the analysis and evaluation of foundations movement of building on sandfilled areas are also controlled by the mentioned factors.

Thus, the adverse effect of the subsurface conditions and the environmental factors on structures placed on the sandfilled areas will form the integral steps both in the investigation into the causes of foundation failures and in the provision of adequate design methods to overcome the foundations problems in the reclaimed areas.

Lastly, in the light of the observations made above, one may be prompted to ask whether or not such information as presented by various authors and organisations are really facts especially in relation to the subsoil conditions in the sandfilled area of the Niger delta.

This gave rise to the idea of carrying out a study on these reclaimed areas to investigate the prevailing soil conditions so as to explain the causes of failures of lightly loaded structures on the sandfilled areas of the Niger delta.

Besides, although there are existing soil report covering these reclaimed areas, due to the heterogeneous nature of soil, it is very unpredictable and any attempt to generalise soil behaviours from any given site to another even within the same environment is in many cases misleading. In the case of hydraulically sandfilled areas, the variability of soil behaviour is complicated by the time dependent consolidation of the weak soil induced by the weight of the fill material and other environmental factors.

Chapter 3

METHODOLOGY, FIELD STUDY, LABORATORY TESTS AND RESULTS

3.1 Methodology

In order to obtain a comprehensive geotechnical appraisal in the sandfill areas of the Niger delta, the study was conducted in four phases in accordance with a typical organisation chart for a site investigation as shown in Figure 3.1.

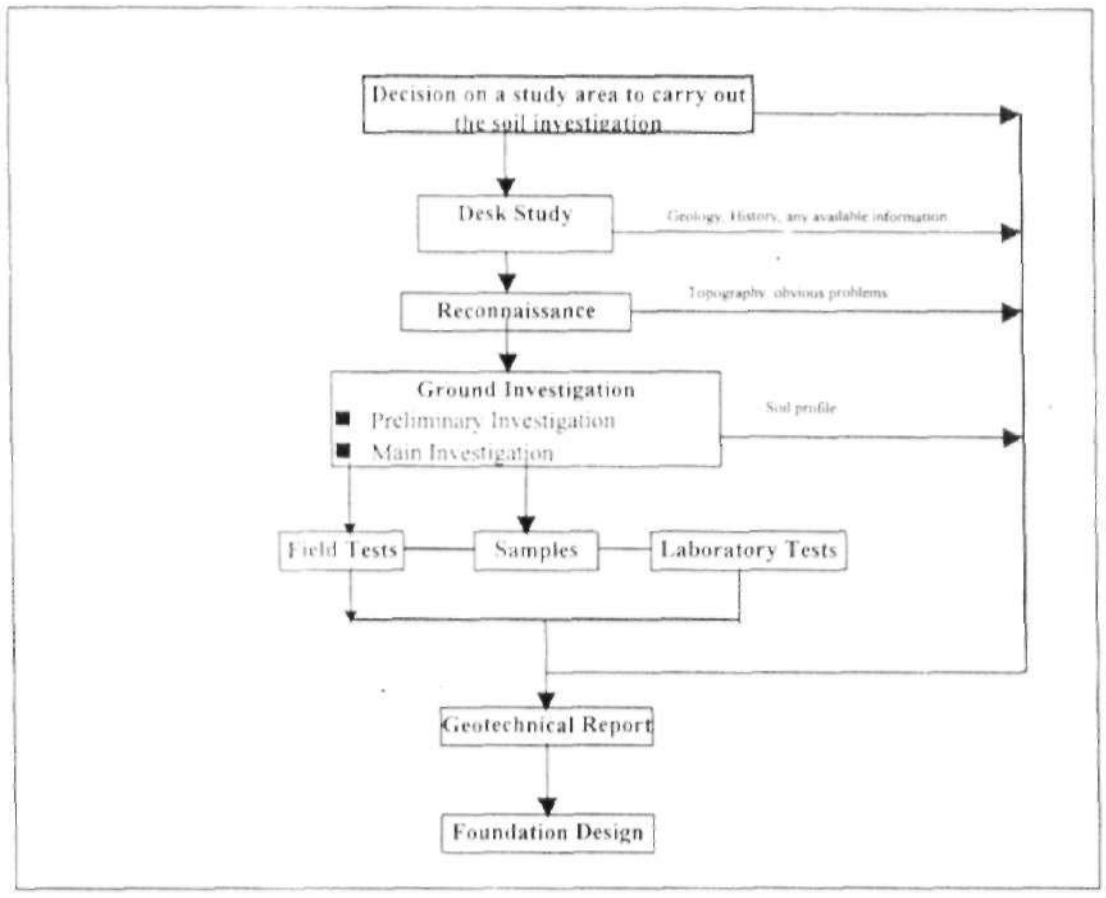


Figure 3.1 Typical Organizational Chart for Site Investigation (Carter and Symons, 1989)

By assembling the information described in the flow chart, the difficulties often encountered in drawing a specification for the work required in ground investigation was highly reduced.

3.1.1 Soil exploration technique on marginal land

The geotechnical investigation in the sandfill areas of the Niger delta is a very tedious and expensive project. This is basically due to the high ground water level characterising the areas and the very weak saturated soils underlying the sandfill material. This condition precludes the use of simple exploratory equipment like hand-auger, excavation by trial pits etc. In this work, details of the investigation method adopted, the equipment used, including the various modifications made in order to cope with the soil condition in these areas are presented under the relevant headings.

For an appropriate and precise assessment of the subsoil profile, the soil investigation was based on the requirement of BS 5930 (1981) while the laboratory test procedure is in accordance with the of provision of BS 1377 (1990). Besides works published by soil experts on the methods of ground investigation for civil engineering purpose in the reclaimed areas of the Niger delta are incorporated to enhance higher accuracy in the evaluation of the subsoil data.

(i) Boring equipment

The standard percussion boring rigs available in the southern part of Niger are the "Pilcon" and "Dando" types. The sizes and weights of these rigs render them highly cumbersome to be towed into position through the congested sandfilled areas where buildings are constructed haphazardly without any general layout plan. For this reason, the locally constructed rig, generally known as "hand rig" was used. The hand rig is in general similar to the standard light cable percussion boring rig with the exception that the winch is hand-operated. It consists essentially of a winch, four shear legs and pulleys. Each leg is formed

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with two tubular steel pipes which can easily be assembled or dismantled at the site. The different components of the rig are carried through virtually impenetrable terrain barrier to the project site by personnel. All standard drilling tools for percussion drilling and in-situ testing in boreholes are used.

Furthermore, since the rig is hand operated, a smaller sampler was used for collection of the undisturbed samples instead of the standard 100 mm diameter sampler (U100). The dimension of the undisturbed sample tube is 60 mm diameter by 240 mm length (U60). This U-tube size is very suitable for hand operated rigs in weak soils of the sandfilled areas under investigation. The smaller sampler has effect on the sample preparation for triaxial test. Instead of the usual three sets of samples requirement of BS 1377 (1990), only two sets of undisturbed sample can be extracted from the sampler. Nevertheless, the triaxial test results obtained using the two sets of sample preparations have been found very satisfactorily comparable to the three sets of samples commonly used (Enoch George Associates, 1988, 1994, George and Abam, 1989).

(ii) Penetrometers

Most of the penetration testing in the Niger delta is conducted with `Gouda' type Dutch cone penetrometers. However, dynamic penetrometers of `Borros' and `Lindmeyer' types are also used, although less frequently. The deployment of the much heavier 100 and 200 kN types is controlled substantially by environmental and soil conditions. The light-weight and manually operated type with a thrust of 25 kN is versatile and easily deployed on various project sites in the Niger delta.

However, the anchoring of this equipment to the very quick clays prevalent in the

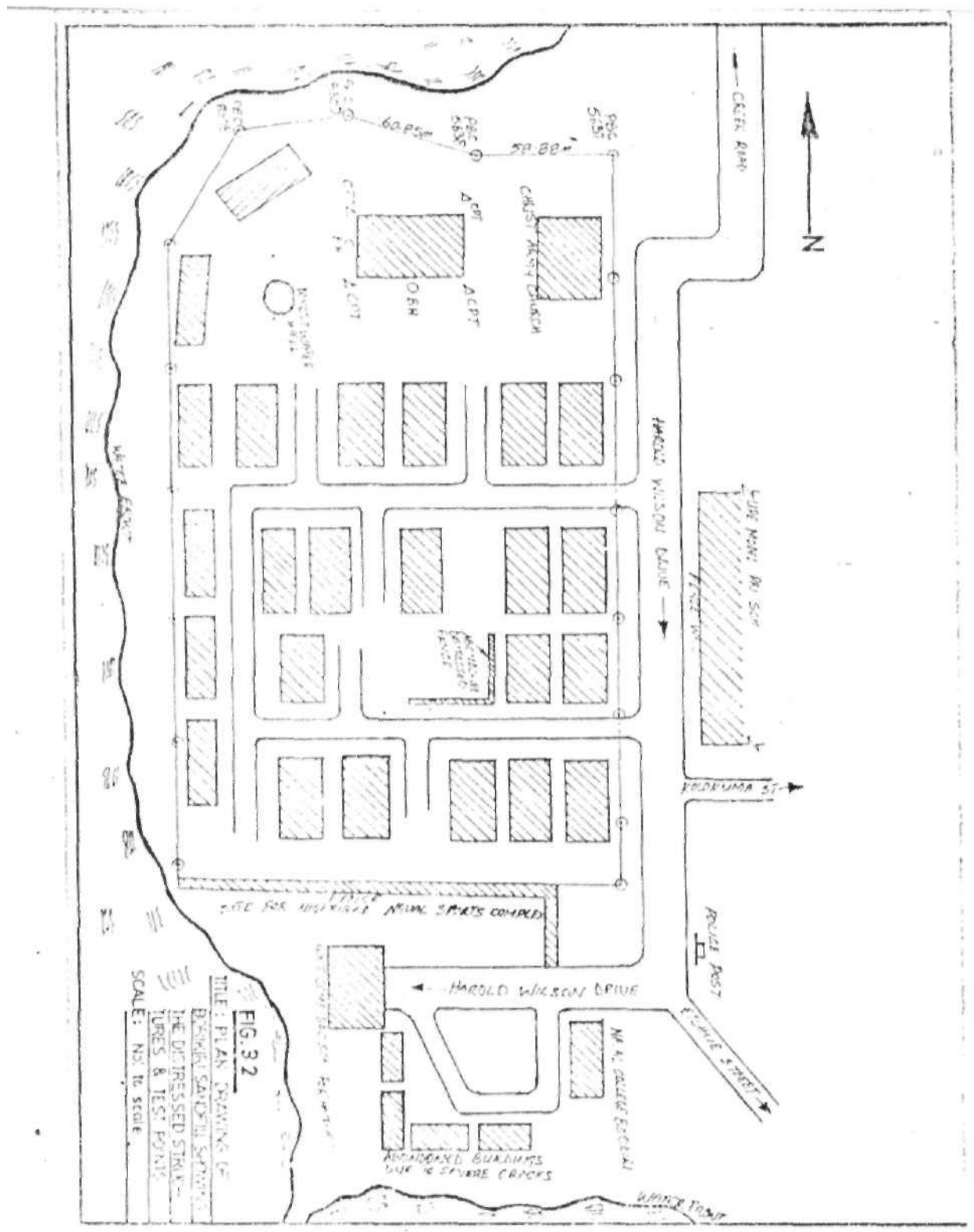
mangrove swamps often presents a major problem. Locally constructed anchor beams are used in some cases to develop the reaction required for the thrust of 25 kN. The anchor beams permit the use of more anchors than are normally required (George and Abam, 1989). A 25 kN thrust will normally result in penetration depth of 20 m in medium dense sands and stiff clays. However, in weak soils, penetration depth up to 30 m may be achieved.

3.2 Description of Study Area

The detailed geotechnical investigation was carried out in two developed sandfilled areas in the mangrove swamp environment of the Niger delta, namely Borikiri in Port Harcourt and Ogolo-Ama in Okirika local government area of Rivers State. The site description for the two sites is as presented below:

3.2.1 Borikiri sandfilled area

Borikiri sandfill is situated at the south-eastern tip of Port Harcourt and enclosed by rivers, creeks and mangrove swamps. At the south-eastern part of the site lies the Government Comprehensive Secondary School which provides a provisional land access to the site. The site is a former mangrove swamp that was reclaimed between 1979 and 1982 by hydraulic sandfilling and developed more than nine years ago. Consequently, the topography of the area is near-level. The soil investigation was executed at the vicinity of a distressed bungalow. The building was constructed in 1990 and is believed to be the first structure constructed on the sandfill site after its commission in 1987. The distressed building is located at the beginning of Borikiri sandfill from the end of the Port Harcourt Creek Road Market. The plan drawing, showing the location of Borikiri sandfill area, the distressed building and the position where field testing took place is shown in Figure 3.2.


CREEK ROAD
N
HAROLD WILSON DRIVE
HODEKUMA ST.
POLICE POST
POLICE STREET
HAROLD WILSON DRIVE
FIG. 32

3.2.2 Ogolo-Ama sandfilled area

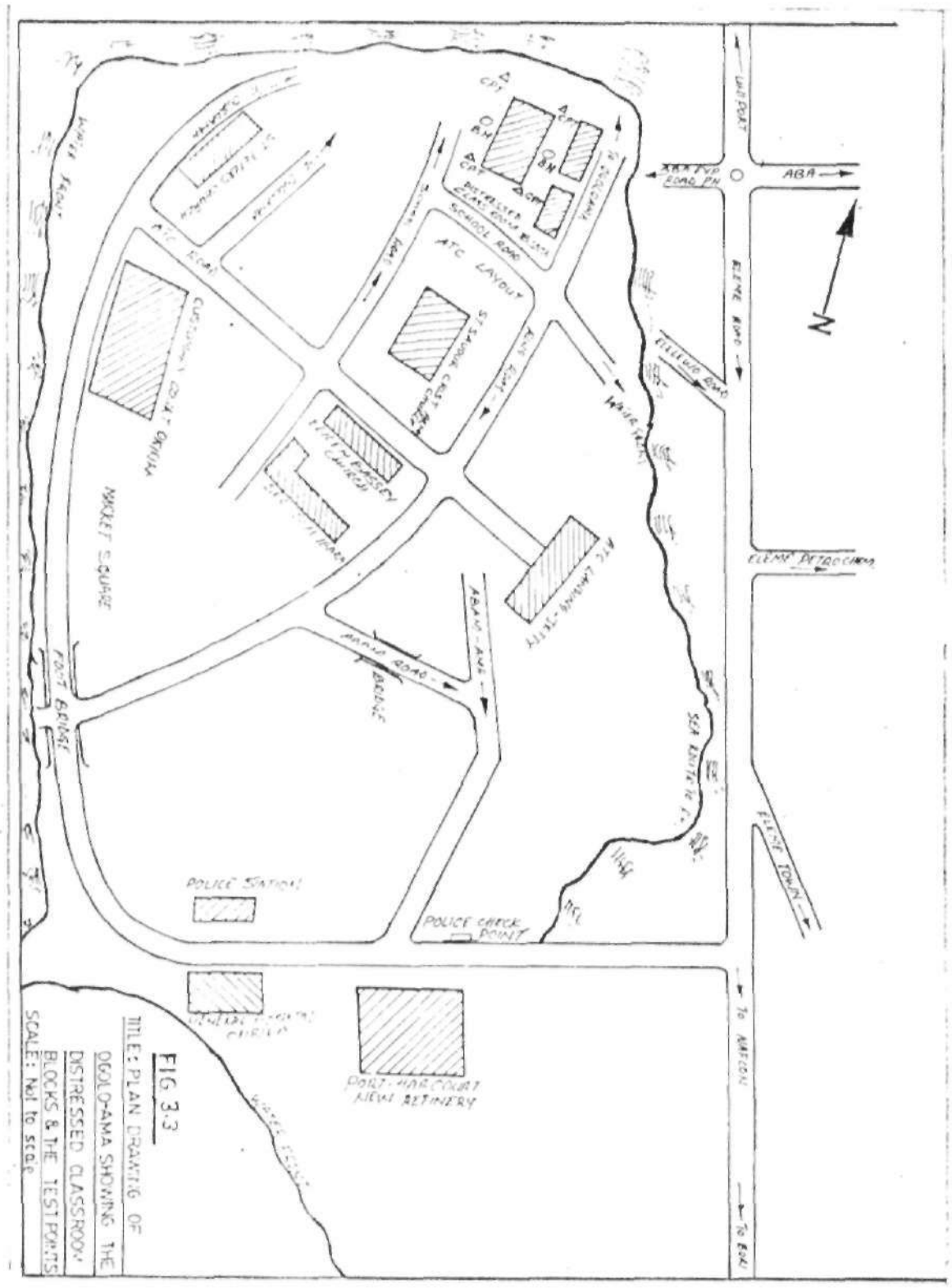
The sandfilled area is located in a coastal town in Okirika local government area of Rivers State. The main access to Ogolo-Ama sandfill is from Port Harcourt town through Tai-Eleme using the Port Harcourt new refinery road. The Port Harcourt new refinery road passes through Okirika township and terminates at Ogolo-Ama sandfilled area. Another access to the site is by speed boat from Okirika water-side in Port Harcourt town and passing through rivers or creeks adjacent to the sandfilled area.

Land reclamation in Ogolo-Ama was carried out between 1982 and 1984. The area was a former mangrove swamp, reclaimed by hydraulic sandfilling, consequently the topography is flat. The soil investigation was executed within the vicinity of the secondary school where some of the classroom blocks are seriously cracked and parts of the buildings are tilting in one direction. The survey drawing showing the location of the classroom blocks, and the position where field tests were conducted is shown in Figure 3.3.

3.3 Field Investigation

3.3.1 General

The field investigation at Borikiri and Ogolo-Ama were carried out for the common purpose of using the information obtained in assessing the failure characteristics of the sandfilled areas of the Niger delta. To achieve this, two main approaches were considered namely: the determination of the thickness of the sandfill and the stratal succession of the underlying deposits. In planning the investigation, the following factors were taken into consideration:


ABA
ATC LAYOUT
SCHOOL ROAD
ATC GATE
ATC ROAD
MARKET SQUARE
FORT BRIDGE
ABAM - AVE
POLICE STATION
POLICE CHECK POINT
PORT-HARCOURT NEWI REFINERY
To AMECON
To ENU
FIG 3.3
TITLE: PLAN DRAWING OF OGALO-AMA SHOWING THE DISTRESSED CLASSROOM BLOCKS & THE TEST POINTS
SCALE: Not to scale

(i) The high ground water level which prevented the use of hand-auger for collection of undisturbed samples.

(ii) The need to obtain adequate borings and penetration soundings to yield the required information.

(iii) The need to obtain adequate soil samples for field identification and classification and other laboratory tests.

At Borikiri sandfill, the field investigation which involved two boreholes and four penetrometer soundings began on the 20th of August, 1996 and was completed on the 26th of August, 1996. The soil investigation was executed around a distressed residential building. One of the two boreholes was located very close to the sections of the house that were seriously cracked while the other borehole was located in front of the house due to lack of space for the rig to be positioned adjacent to the building.

The four penetrometer soundings were situated at the corners of the distressed building. Both the boreholes and the probes terminated at the proposed depth of 12m. The site plan, showing the distressed building and the position of the test points is as presented in Figure 3.2.

The second field study location was situated in Ogolo-Ama in the vicinity of a secondary school, around a distressed classroom block. The field investigation on this site was started on the 20th of February 1996 and was completed on the 26th of February 1997. Two boreholes were drilled out to the terminating depth of 15,0m while the four core penetrometer soundings carried out terminated at the depth of 20.0m. The position of the boreholes and probes are as presented on the site (Figure 3.3). Generally, on the two sites, borehole and probes were positioned so that all different ground conditions were

covered. The equipment used and the method adopted for the field investigation at the sandfilled areas under consideration were similar. The details of the test procedure and results obtained during the field investigation are as described below:

3.3.2 Soil borings and standard penetration test (SPT)

The soil borings were carried out using the light cable percussion rig of the `Pillar Wayfares' type. The equipment consists of a winch, 1500 kN capacity driven by a diesel engine and a derrick about 6m in height. The legs of the derrick fold down to form a simple trailer that can be towed by vehicle. Five people are needed to operate the rig during boring; these are the rig operator and four labourers. Borings were sampled at 0.75 m intervals from the ground surface to the termination of the boreholes. The cohesive soils were sampled with the conventional U-tubes. All samples were examined and roughly classified in the field, the disturbed samples were collected in bags for laboratory testing while tube samples were waxed on site.

Standard Penetration Test (SPT) were performed in the boreholes for the assessment of the relative densities of the cohesionless soil. In this test a 50 mm diameter split tube sampler was driven 450 mm into the soil with a 65 kg hammer falling freely a distance of 760 mm. The number of blows required to effect the last 300 mm penetration is termed the penetration resistance N, and provided an indication of relative density of the strata tested. Typical borehole logs, showing the subsoil description with depth, ground water level during the investigation, moisture content with depth and the SPT N-values are as shown in Figure 3.4a and 3.4.b for the Borikiri and Ogolo-Ama sandfilled areas, respectively.

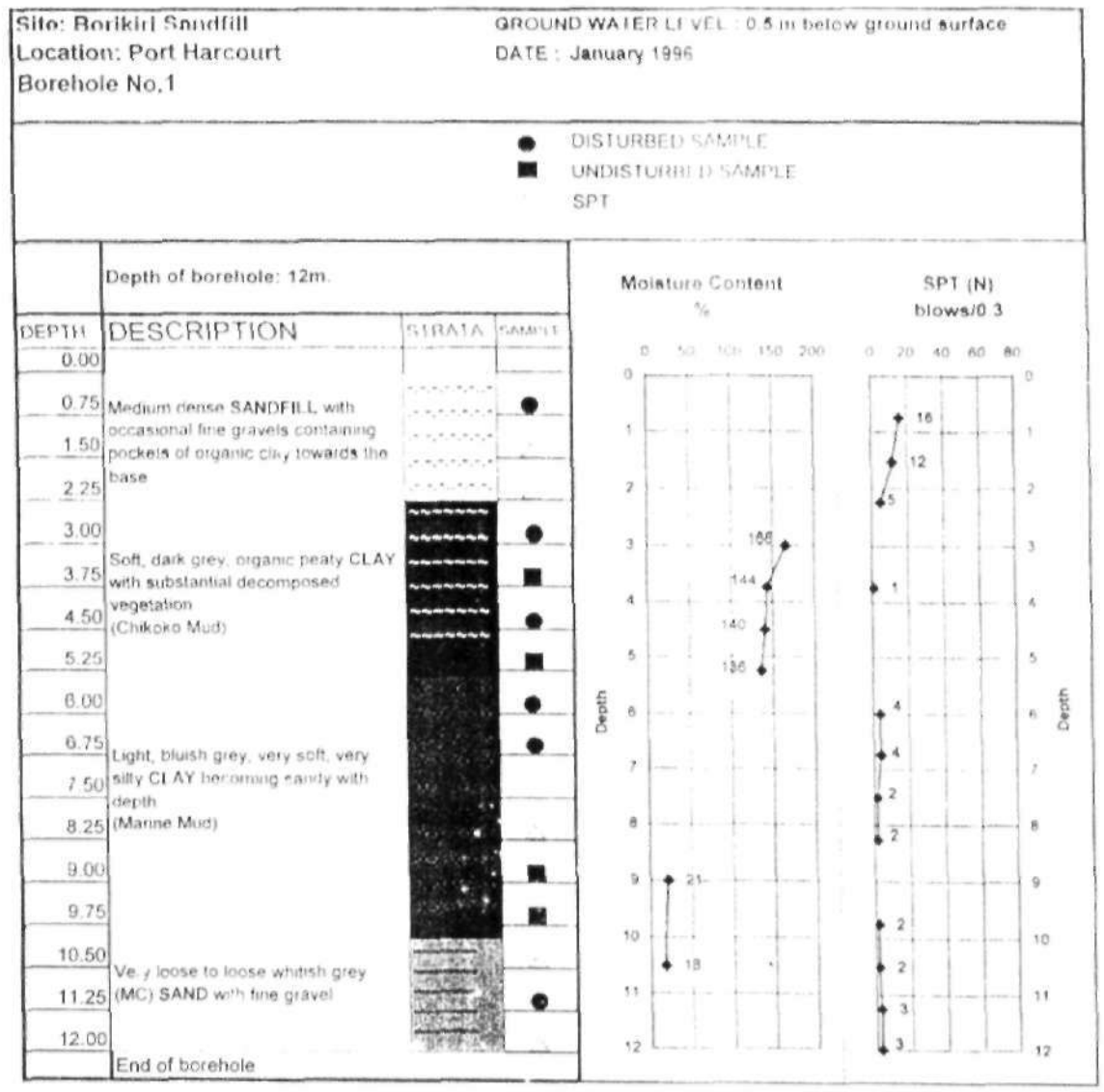


Figure 3.4a: Typical Borehole log for Borikiri

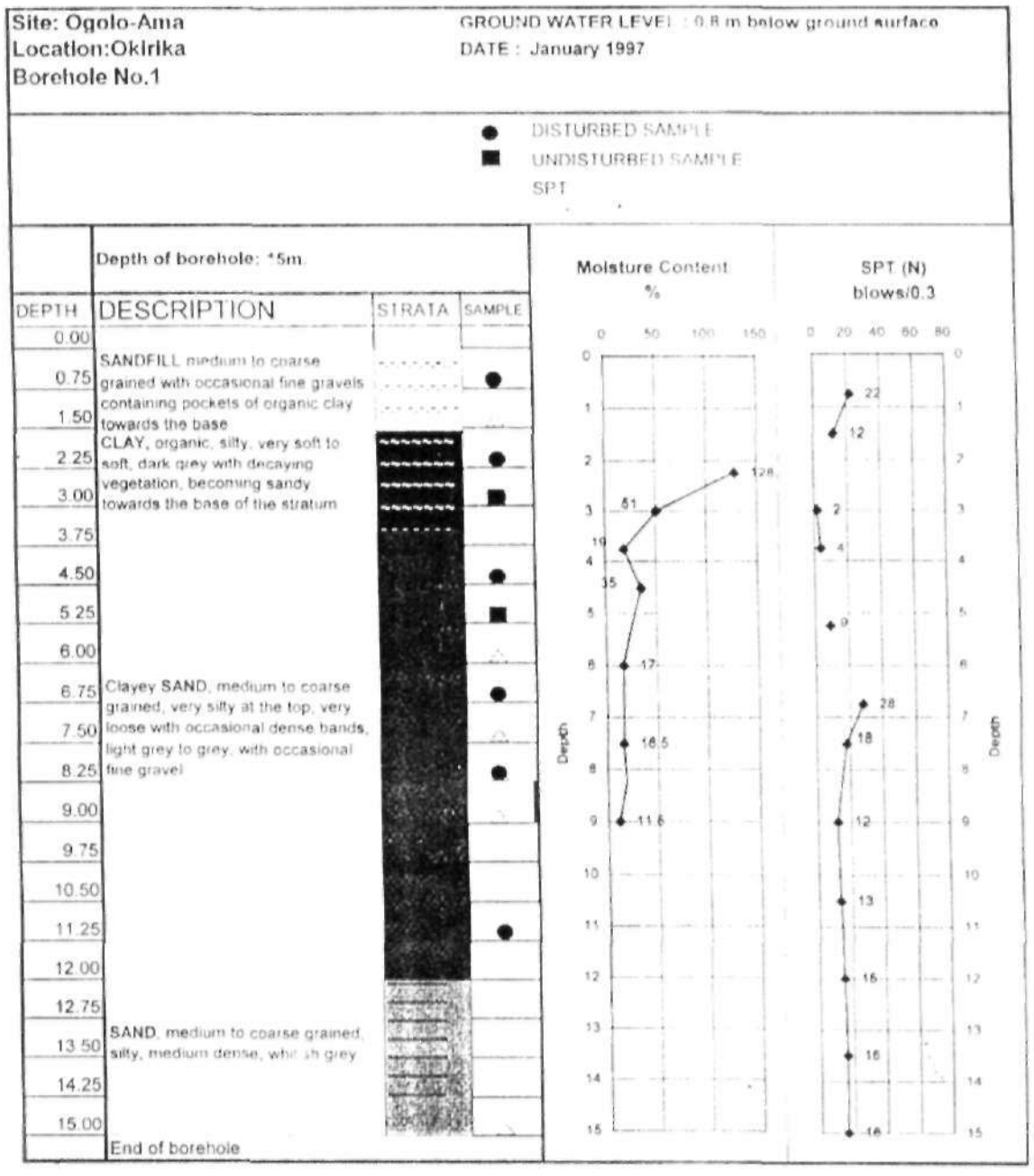


Figure 3.4b: Typical Borehole log for Ogolu-Ama

3.3.3 Cone penetration test (CPT)

The cone penetration tests were performed using the hand-operated light weight static penetrometer 25 kN capacity of the 'Gouda' type. The cone has an apex angle of 60^0, 36 mm diameter and a cone area of 10 cm^2. In the sounding operation, the cone was advanced to the required depth in closed position by exerting pressure on the outer sounding tube. The force on the cone was then measured by hydraulic gauge. In order to obtain the force on the cone and friction jacket, the cone was pushed further down, but with the cone engaging the friction jacket. The measurement this time gives the force on the cone and friction jacket. The process was repeated at 20cm interval into the soil.

Eight cone penetration tests (CPT) were performed on the two investigating sites, four were carried out at Borikiri while the remaining four were carried out at Ogolo-Ama. At Ogolo-Ama the test was terminated at 15.00 m while the depth of investigation at Borikiri was 12.00m. The high depth investigated was to avoid termination of borehole in the weak and highly compressive material. Typical graphs showing the cone penetration resistance against depth for the two sites are presented in Figure 3.5a and 3.5b respectively.

3.3.4 Determination of ground water level

In general, the ground water level measurements were made during boring operation, and from the nearby creeks which are common features in hydraulic sandfilled areas in the Niger delta. But in Borikiri sandfill the ground water table was also measured from the domestic well located about three meters from the distressed building.

Site: Borikiri Sandfill
Location: Port Harcourt
Test No: CPT
Date: 1/2/96

Angle 60°
Area 10 cm²
Rod Ø 35 mm
Penet. depth 20 cm

Point N. 2

Dutch Cone Penetrometer Sounding

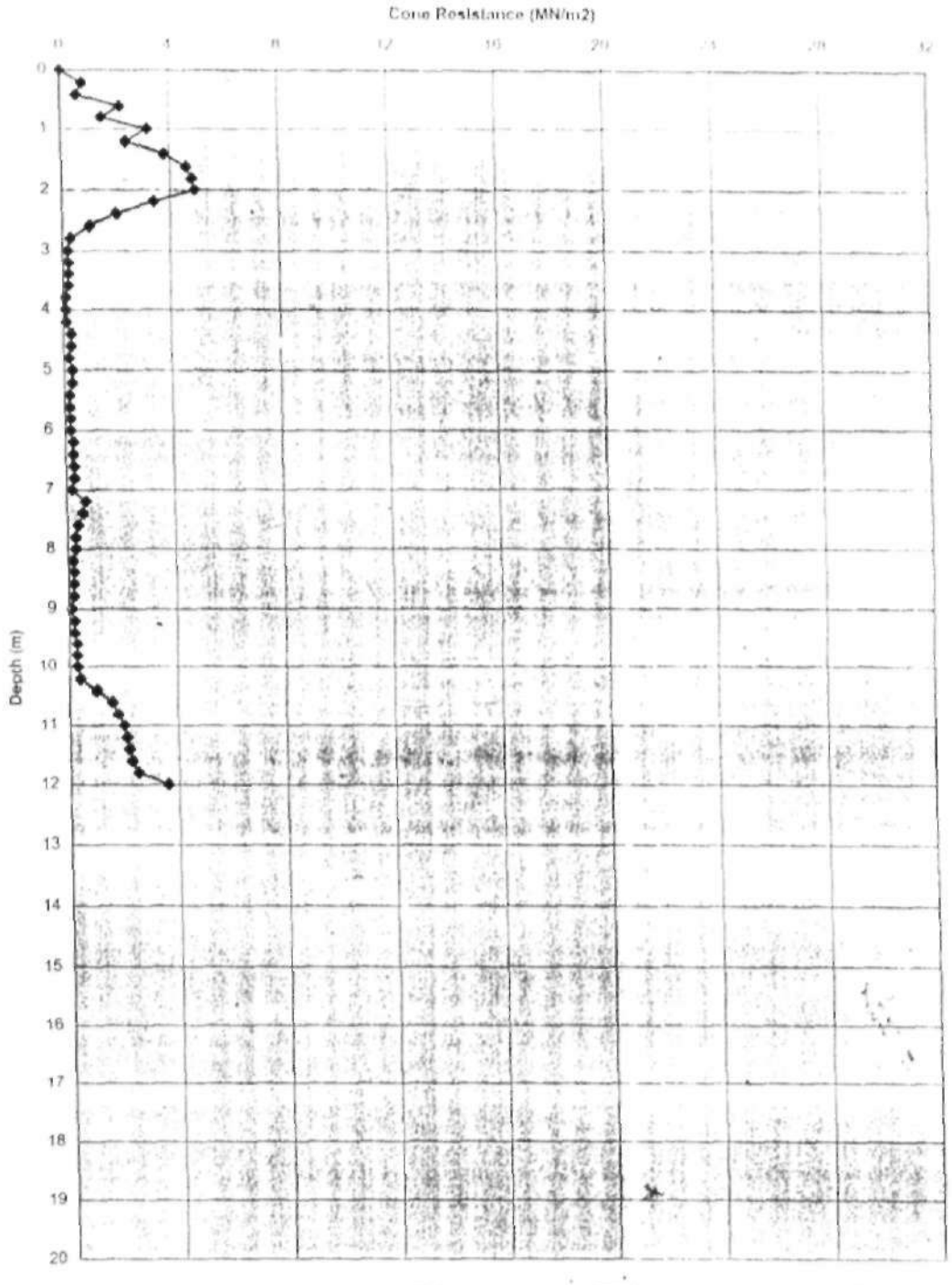


Figure 3.5a: Typical Conc Penetration Resistance for Borikiri

Site: Ogolo Area
Location: Okrika
Test No: CPT
Date: 24/08/93

Angle: 60°
Area: 10 cm²
Rod Ø: 15 mm
Penet. depth: 20 cm

Point N. 3

Dutch Cone Penetrometer Sounding

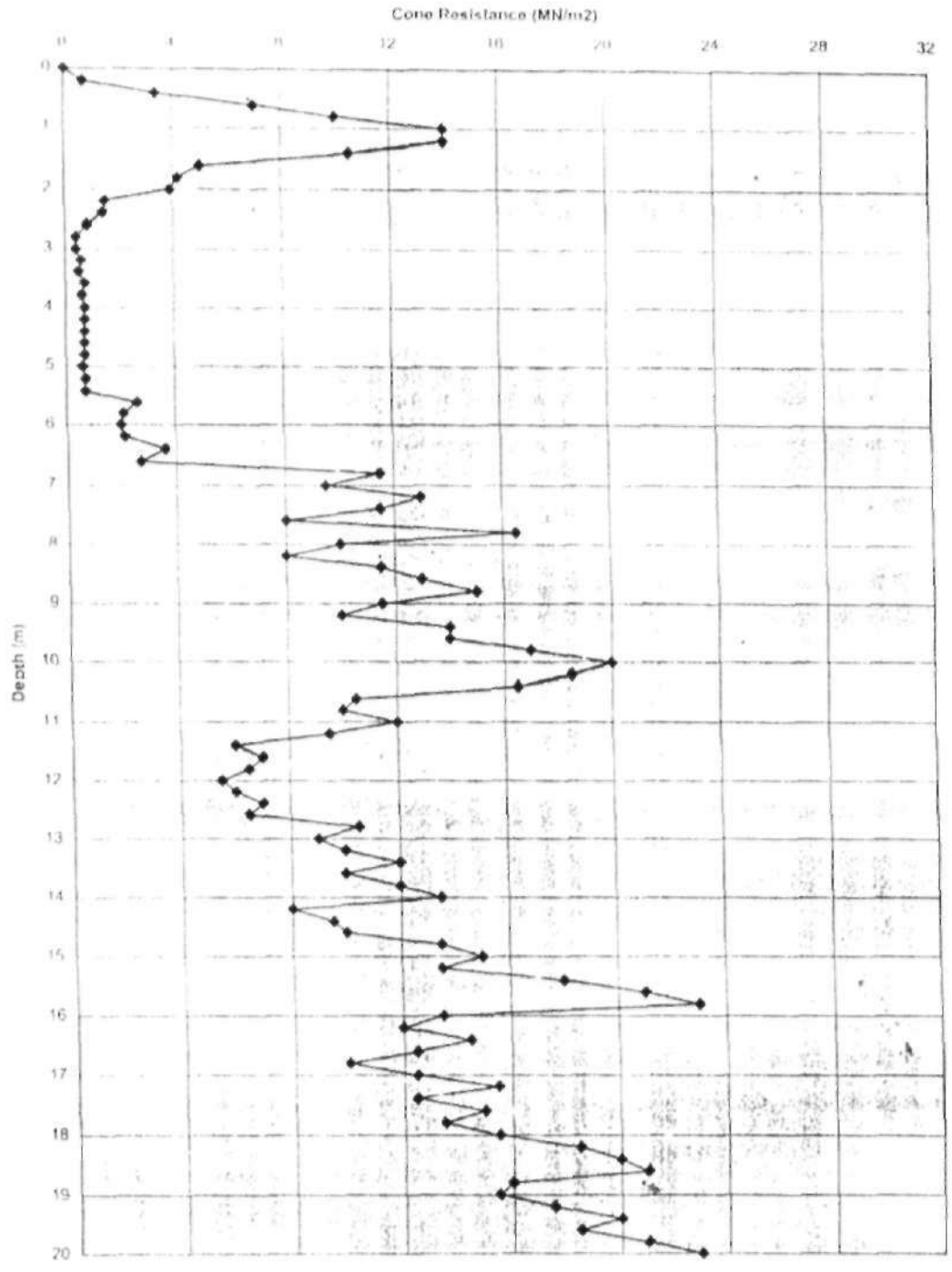


Figure 3.5b: Typical Cone Penetration Resistance for Ogolu-Ama

The difference in the ground water levels recorded from the three locations was negligible. The ground water was, in general, encountered just below the ground surface at depths varying from the surface of the ground to about 2.0m below the ground level.

At Borikiri sandfill area the ground water level recorded during the investigation period, varied between 0.4m and 1.5m while in Ogolo-Ama the range of the ground water levels recorded during the investigation period was between 0.6 to 2.00m. With repect to the observation made during the field investigation at both sites, the following deductions were made. The ground water levels recorded during the investigation is as summarized in table 3.1.

Table 3.1: Ground water conditions for Barikiri and Ogolo-Ama Sandfilled Areas

BORIKIRI			
Borehole Number	Low Tide Depth (m)	Full Tide Depth (m)	Average Depth (m)
1	0.6	0.4	0.5
2	0.7	0.35	0.525
OGOLO-AMA			
1	1.0	0.6	0.8
2	0.8	0.6	0.7

From the results in Table 3.1, the ground water levels in the sandfilled areas are still very high and are in agreement with Enoch George Associates (1986), Abam and Okogbue (1993), and Trevi (1993), etc.

3.4 Laboratory Tests And Results

3.4.1 Preamble

A comprehensive laboratory testing programme was embarked upon in order to further investigate soil characteristics at the site and to evaluate the engineering and physical properties of subsurface materials. The testing programme was designed to provide

sufficient information on the geotechnical characteristics which were used to understand the behaviour of the soils. All tests were carried out in accordance with BS 1377(1990).

The tests performed on the samples collected during the boring operations were broadly classified into three groups, namely the Identification and Classification tests, the Strength tests and the Consolidation tests. The identification and classification tests were carried out on the disturbed or bagged samples collected during the boring operation, while the strength and consolidation tests were performed on the undisturbed samples collected with the U60-tube during boring.

3.4.2 Identification and classification tests

The tests were carried out to facilitate accurate geotechnical classification of the soil. All samples obtained from the borings were visually identified and classified at the site. In order to improve the field classifications, standard laboratory classification tests were performed. These tests include particle-size analysis, moisture content determination, specific gravity of solid particles and Atterberg limits.

The sieve analysis was performed on the sandfill material and the alluvial clayey sand underlying the organic clays, using the wet sieving method of BS 1377 (1990). The typical particle-size distribution curves for the fill material in Borikiri and Ogolo-Ama sandfill areas are as presented in figure 3.6a and 3.6b, respectively.

For the particle size analysis, the grading of the soil is best determined by direct observation of its particle size distribution curve (Smith, 1990). In this study, the grading of the soil was simplify by the use of the grading parameters defined as follows:-

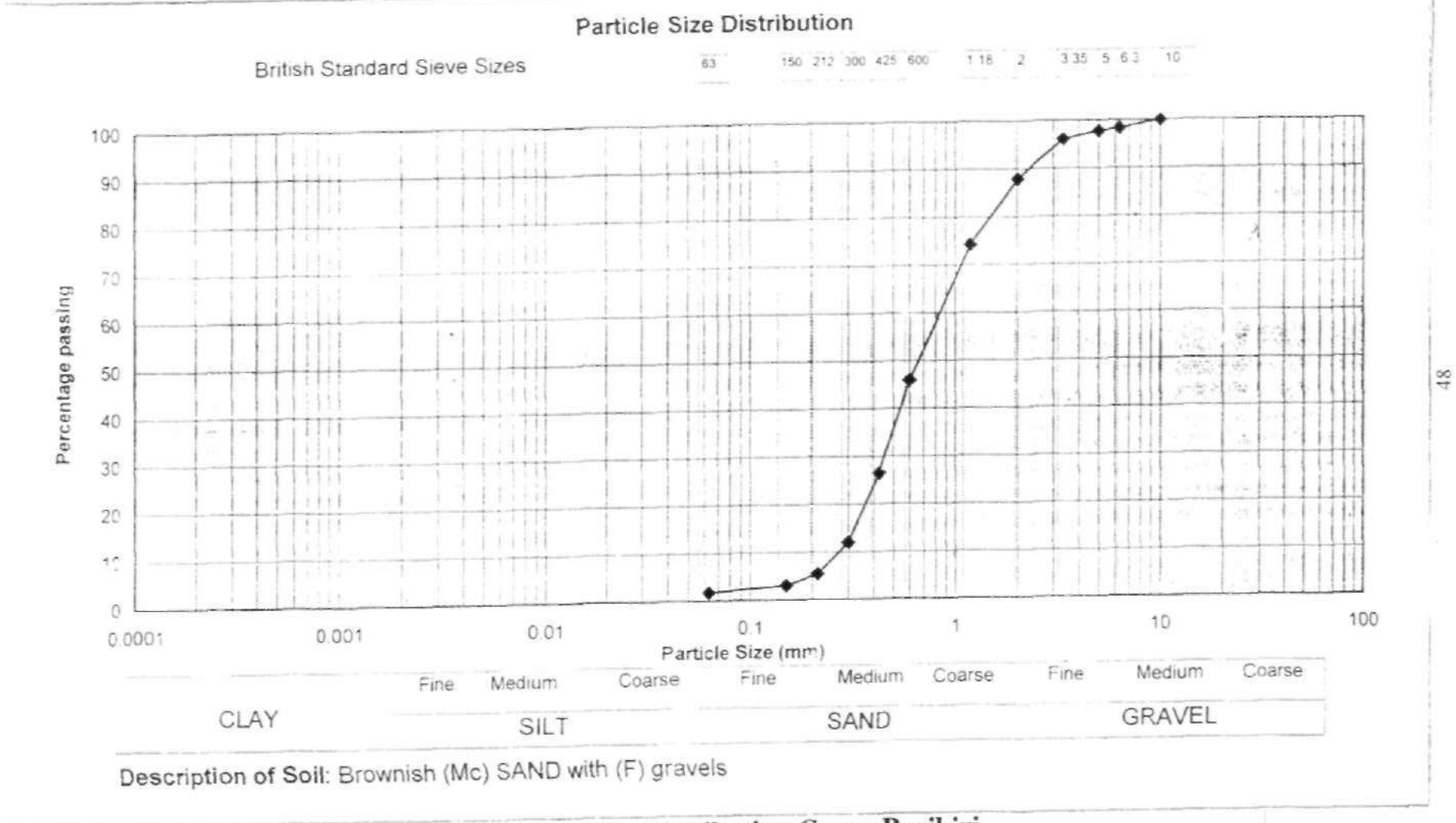
(i) Uniformity coefficient $C_u = D_{60}/D_{10}$

(ii) Coefficient of curative $C_c = d^2_{30}/d_{10} \times d_{60}$

Site: Borikiri Sandfill
Location: Port Harcourt

PROJECT: M.Sc/ENG/04742/93

Borehole No.1
Sample No.1
Depth: 0.75-1.20m



Description of Soil: Brownish (Mc) SAND with (F) gravels

Figure 3.6a: Typical Particle-Size Distribution Curve Borikiri

Site: Ogolo-Ama
Location: Okirika

A.B.U.ZARIA
PROJECT: M.Sc/ENG/04742/93

Borehole No.1
Sample No.1
Depth: 0.75-1.20m

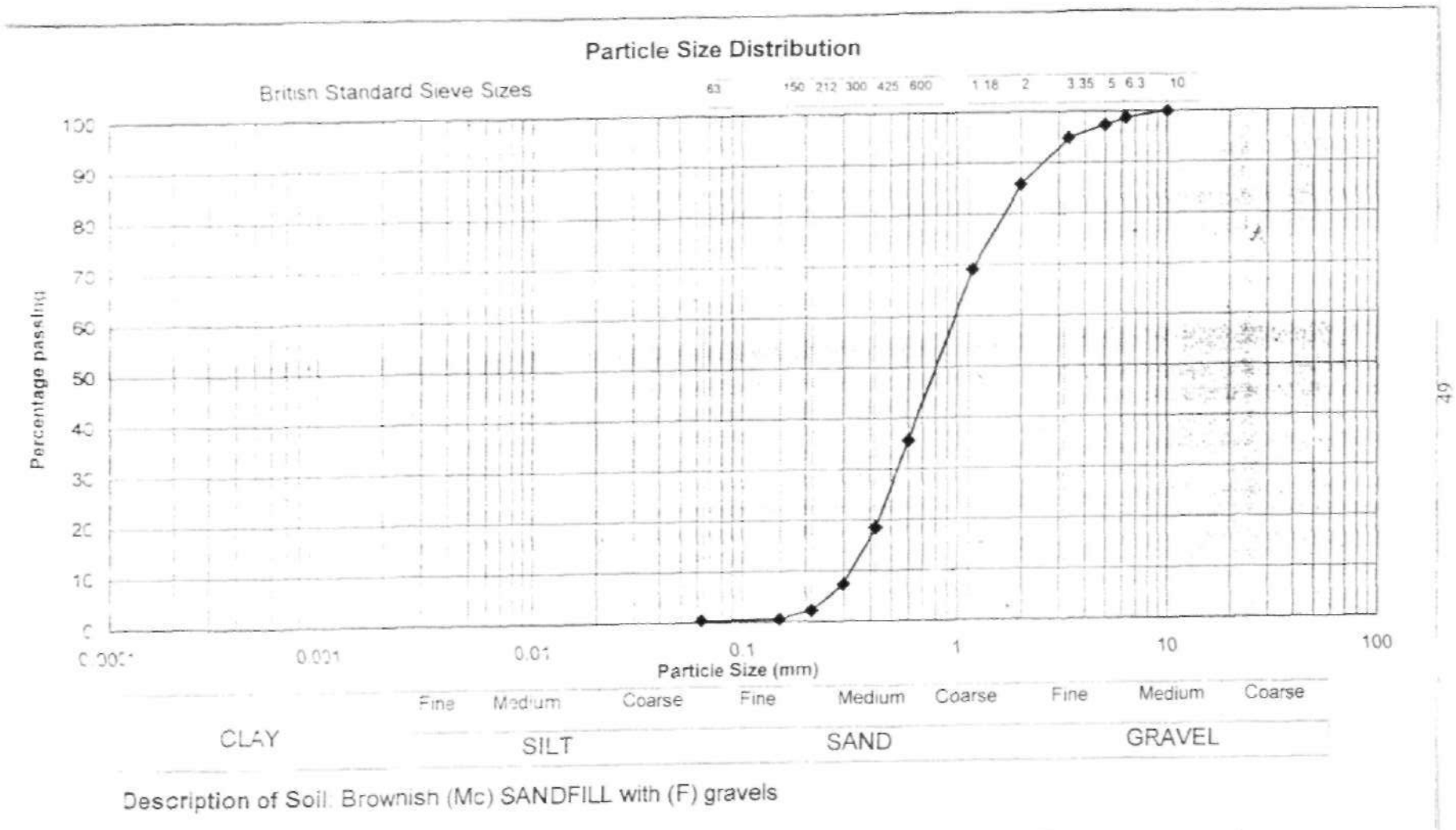


Figure 3.6b: Typical Particle-Size Distribution Curve Ogolo-Ama

The application of the grading parameters and the results obtained from analyses of the particle size distribution curves are summarized and presented in table 3.2a and 3.2b for the two sites. Results obtained from the particle size analysis are summarised in Tables 3.2a and 3.2b for the two sites investigated sites.

3.4.3 Atterberg limits

The three main Atterberg limits, liquid limits, plastic limit and shrinkage limit are widely used for soil identification and classification test. The shrinkage limit is particularly useful in areas in which the soils undergo large volume charges when going through wet and dry cycles. With the soil condition under consideration, only the liquid and plastic limits were carried out on the samples. The method employed in the determination of the liquid limits of the fine grained soils was based upon the Casagrande apparatus, using the procedure specified in BS 1377 (1990).

However, there were modifications in the sample preparations, in order to cope with the soil condition at certain depths especially the organic clay just below the sandfill material that contained substantial amounts of decomposed plant debris that may sometimes form more than 50% of the soil sample collected. Thus for the cohesive samples that were affected by the high organic content, the liquid limit and plastic limit were performed on the natural sample from the site without drying and sieving through the 425mm test sieve specified by BS 1377 (1990).

The laboratory Atterberg limits test results plotted for Borikiri and Ogolo-Ama sandfilled areas are presented in Appendices A_1 and A_2, respectively. In addition, the results obtained from the analysis of Atterberg limits curves, are presented in Table 3.3 for the two investigation sites.

Table 3.2a : Soil Classification (Grain Size Analysis), Borikiri

Borehole N	Depth of Sample	d_{10} (mm)	d_{30} (mm)	d_{50} (mm)	d_{60} (mm)	$C_u = d_{60}/d_{10}$	C_c	USCS	N (blows/0.3m)	Percentage of gravel	Percentage of SAND	Percentage of Fines
1	0.75-0.95	0.28	0.45	0.72	0.80	2.9	0.9	SP	16	12	85	3
	1.50-1.95	0.28	0.54	0.76	0.96	3.4	1.1	SP	12	2	95	3
	2.25-2.70	0.25	0.37	0.47	0.51	2.0	1.1	SP	5	7	63	30
										3	62	35
	10.50-10.95	-	-	0.45	0.59	-	-	SP-SC	2	8	70	22
	12.00-12.45	-	0.24	0.54	0.72	-	-	SP-SC	3			
2	0.75-0.95	0.26	0.34	0.43	0.52	2.0	0.9 -	SP	14	15	84	1
	1.50-1.95	0.28	0.43	0.56	0.68	2.4	1.0	SP	8	4	93	3
	10.50-10.95	-	0.13	0.39	0.70	-	-	SP-SC	3	2	85	13
	12.25-12.45	-	1.18	0.48	0.78	-	-	SP-SC	6	2	76	22

NOTE: Effective Particle Size = d_{10} Unified Soil Classification = (USCS) Fines = Silt and Clays
Mean Particles Size = d_{50} SPT Penetration Resistance (N)

Table 3.2b: Soil Classification (Grain Size Analysis), Ogolo - Ama

Borehole N	Depth of Sample (:n)	d_{10} (mm)	d_{30} (mm)	d_{50} (mm)	d_{60} (mm)	$Cc=d_{60}/d_{10}$	$Cc=(d_{30})^2/(d_{10}*d_{60})$	(USCS)	N (blows/0.3 m)	Percentage of Gravel (%)	Percentage of Sand (%)	Percentage of Fines
	0.75 – 1.20	0.34	0.54	0.8	0.9	2.6	0.95	SP	22	14	85	1
	1.50-1.95	0.21	0.41	0.53	0.62	3.0	1.3	SP	12	6	79	15
	6.00-6.45	-	0.41	0.72	0.92	-	-	SP-SC	9	-	-	-
	7.50-7.95	0.17	0.39	0.42	0.44	2.6	2.0	SP	18	-	-	-
	9.00-9.45	0.33	0.62	0.79	0.84	2.5	1.4	SP	25	2	74	24
	10.50-10.95	0.44	0.79	1.20	1.50	3.4	0.9	SP	18	9	87	4
	13.50-13.95	0.12	0.38	0.60	0.80	6.7	1.5	SP	10	8	89	3
	15.00 – 15.45	0.1	0.39	0.6	0.8	8	1.9	SP	16	10	86	4
	0.75 – 1.20	0.3	0.47	0.67	0.8	2.7	0.9	SP	13	7	92	1
	1.50-1.95	0.32	0.54	0.72	0.81	2.5	1.3	SP	13	4	94	2
	6.00-6.45	0.20	0.27	0.29	0.30	1.5	1.2	SP	12	-	-	-
2	7.50-7.95	0.21	0.30	0.34	0.38	1.8	1.1	SP	18	-	-	-
	9.00-9.45	0.32	0.50	0.60	0.66	2.1	1.2	SP	35	-	-	-
	10.50-10.95	0.38	0.57	0.72	0.83	2.2	1.0	SP	19	2	84	14
	12.00-12.45	0.30	0.63	0.93	1.10	3.7	1.2	SP	16	16	81	3
	15.00-15.45	0.12	0.37	0.60	0.79	6.6	1.4	SP	11	30	69	1

NOTE: Effective Particle Size = d_{10} Unified Soil Classification = (USCS) Fines = Silt and Clays SP poorly graded sand.
Mean Particles Size = d_{50} SPT Penetration Resistance (N) SP – SC poorly graded sand with clay

Table 3.3 : Atterberg Limits Results

Soil Classification (Atterberg limits), Borikiri

Borehole N	Depth of Sample (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unified Soil Classification (USCS)	British Soil Classification (BSSCS)
1	3.00-3.45	-	NP	-	-	-
	4.50-4.95	175	68	107	CH	CE
	5.25-5.70	203	62	141	CH	CE
	6.00-6.45	29	10	19	CL	CL
	8.25-8.70	32	11	21	CL	CL
	9.75-10.20	24	NP	-	CL-ML	CL
2	3.00-3.45	21	NP	-	-	-
	4.50-4.95	170	61	109	CH	CE
	5.25-5.70	259	46	213	CH	CE
	6.00-6.45	67	16	51	CH	CH
	8.25-8.70	55	19	36	CH	CH
	9.75-10.20	28	9	19	CL-ML	CL

Soil Classification (Atterberg limits), Ogolo - Ama

Borehole N	Depth of Sample (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unified Soil Classification (USCS)	British Soil Classification (BSSCS)
1	3.00-3.45	-	NP	-	-	-
	4.50-4.95	175	68	107	CH	CE
	5.25-5.70	203	62	141	CH	CE
	6.00-6.45	29	10	19	CL	CL
	8.25-8.70	32	11	21	CL	CL
	9.75-10.20	24	NP	-	CL-ML	CL
2	3.00-3.45	21	NP	-	-	-
	4.50-4.95	170	61	109	CH	CE
	5.25-5.70	259	46	213	CH	CE
	6.00-6.45	67	16	51	CH	CH
	8.25-8.70	55	19	36	CH	CH
	9.75-10.20	28	9	19	CL-ML	CL

3.4.4 Strength tests

The strength tests were performed on the undisturbed clay samples extracted from the boreholes with U60-tubes. Considering the soil condition in the sandfill area, two types of strength tests were carried out on the clays.

Generally, single stage unconsolidated undrained triaxial compression tests were carried out on the clay samples, and the laboratory vane tests were also performed on some of the clay

samples. The method adopted was based on the procedures of BS 1377 (1990). The laboratory triaxial test readings recorded for the two sites were relatively low, thus confirming the weak consistency of the clays underlying the fill. These reading were used to plot the 'stress versus strain curves'. **Appendix** B_1 and B_2 show graphs obtained for the weak clays of Borikiri and Ogolo-Ama respectively.

3.4.5 Consolidation tests

Oedometer tests were performed on the selected clay soil from all U60-tube samples to determine the compressibility characteristics of the organic silty clays. The test generally followed the method prescribed in BS 1377 (1990). The values of the coefficient of volume compressibility (m_v) were obtained from the stress range of effective overburden pressure plus the working stress.

Results obtained from the analysis of the strength tests and consolidation tests data will be highlighted in the relevant chapters where detailed analysis of the strength test data and compressibility of the weak soils are considered. However, as pointed out in the introduction, in this study emphasis is on settlement analysis using the various methods of compressibility analyses. Thus, details of the analysis and the results obtained will remain the focus of the work.

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Chapter 4

ANALYSIS OF STRENGTH AND COMPRESSIBILITY OF WEAK SOILS OF THE SANDFILLED AREAS

4.1 General Approach to Shear Strength Analysis

Shear strength parameters are required for the design of all types of foundations, retaining walls, and for calculating the stability of embankments, cuttings and natural slopes. The prevailing method of shear strength analysis is based on the work of Coulomb (1766) who suggested that the strength envelope of soil could be expressed by a straight line Equation:

$$\tau_f = C + \sigma \tan\phi \qquad (4.1)$$

Where τ_f = shear stress at failure, i.e. shear strength

C = unit Cohesion

σ = total normal stress on failure plane

A complication arises when the normal stresses and angle of shearing resistance are carried partly by the soil skeleton, thus Equation 4.1 is modified to:

$$S = C' + (\sigma - u) \tan\phi \qquad (4.2a)$$

$$S = C' + \sigma \tan\phi \qquad (4.2b)$$

where **u** is the pore water pressure, $\sigma' = (\sigma - u)$ the effective normal stress and $'$ and ϕ' shear strength parameters related to effective stresses.

Thus, when considering the shear strength of soils, there is a choice: either the total combined response of the soil and pore water can be considered, or the specific response of the soil skeleton can be separated from the pore water pressure by considering effective stress (Equations 4.2).

Practically, the actual soil strength that applies in situ is dependent upon the type of soil, its previous stress history, the drainage conditions, the forms of construction and loading. Hence, the shear tests chosen to determine the soil strength parameters to be used in a design should reflect the conditions that will actually prevail during and after the construction period.

The weak soils underlying the sandfill material in all the sandfilled areas investigated, are soft to very soft in consistency (shown in Appendix B1 and B2) and are fully saturated as a result of the high ground water levels associated with the reclaimed areas of the Niger delta. For these reasons, the analysis of shear strength in this work are mainly based on the undrained parameter Cu and ϕ_u.

4.1.1 Shear strength from quick undrained triaxial tests

The shear strength of a soil expressed in terms of total stress, corresponds to Coulomb's law:

$$\tau_f = C_u + \sigma \tan \phi_u \qquad 4.3$$

Where C_u = unit cohesion of the soil, with respect to total stress, kN/m^2

ϕ_u = angle of shearing resistance of soil, with respect to total stress, degree

σ = total normal stress on the plane of failure, kN/m^2

For saturated cohesive soils tested in undrained shear, it is generally found that τ_f has a constant value being independent of the value of the cell pressure σ. The main exception to this finding is in fissured clay (Smith, 1990). Hence, it is generally assumed that $\phi_u = 0$ when a saturated cohesive soil is subjected to undrained shear, this gives

$$\tau_f = C_u = 0.5\ (\sigma_1 - \sigma_3) \qquad 4.4$$

where the term C_u is the undrained shear strength of the soil.

To obtain the undrained shear strength parameter of Equation (4.3), an attempt was made to plot the Mohr circle for each set of sample tested for the two sites, and the best common tangent to the circles were taken as the strength envelope. Typical Mohr circle plotted for the weak organic clay immediately underlying the fill material for the reclaimed areas is shown in figure 4.1a and 4.1b.

Generally, the Mohr circle drawn for the organic peaty clay sample tested portrays that of a saturated cohesive soil. The angle of shearing resistance ϕ_u of these clay soils are very small (range 0°-5°) and can be neglected or assumed to be zero without any markable effect on the strength parameters. Furthermore, the low value of the dial readings, made the construction of the Mohr circle tedious and with little success. Thus, with regards to the characteristics of the weak organic clays of the sandfilled areas, a more straight forward analysis of the shear strength values was conducted using Equations (4.4) provided an easy and accurate method of shear strength analysis. In addition, the results obtained were in agreement with the ones obtained from the Mohr circle envelopes.

The result of the undrained shear strength parameters, obtained from the analysis of triaxial test results and the laboratory vane test are summarised in Tables 4.1a and 4.1b for the sandfilled areas investigated.

4.1.2 Undrained shear strength from SPT N-values

Attempts have been made to correlate the unconfined compressive strength or the undrained shear strength of clays with the results of Standard Penetration Test with varying degrees of success. In the sandfilled areas investigated, due to the difficulties of obtaining the undisturbed sample of organic clays at certain depths, basically because of the high water

A.B.U. ZARIA

PROJECT: M.Sc./ENG/04742/93

Mohr Circle Parameters

OPERATOR: Ekpo, A.U.

Site : Borikiri Sandfill
Borehole No. 1
Depth 3.00-3.45m

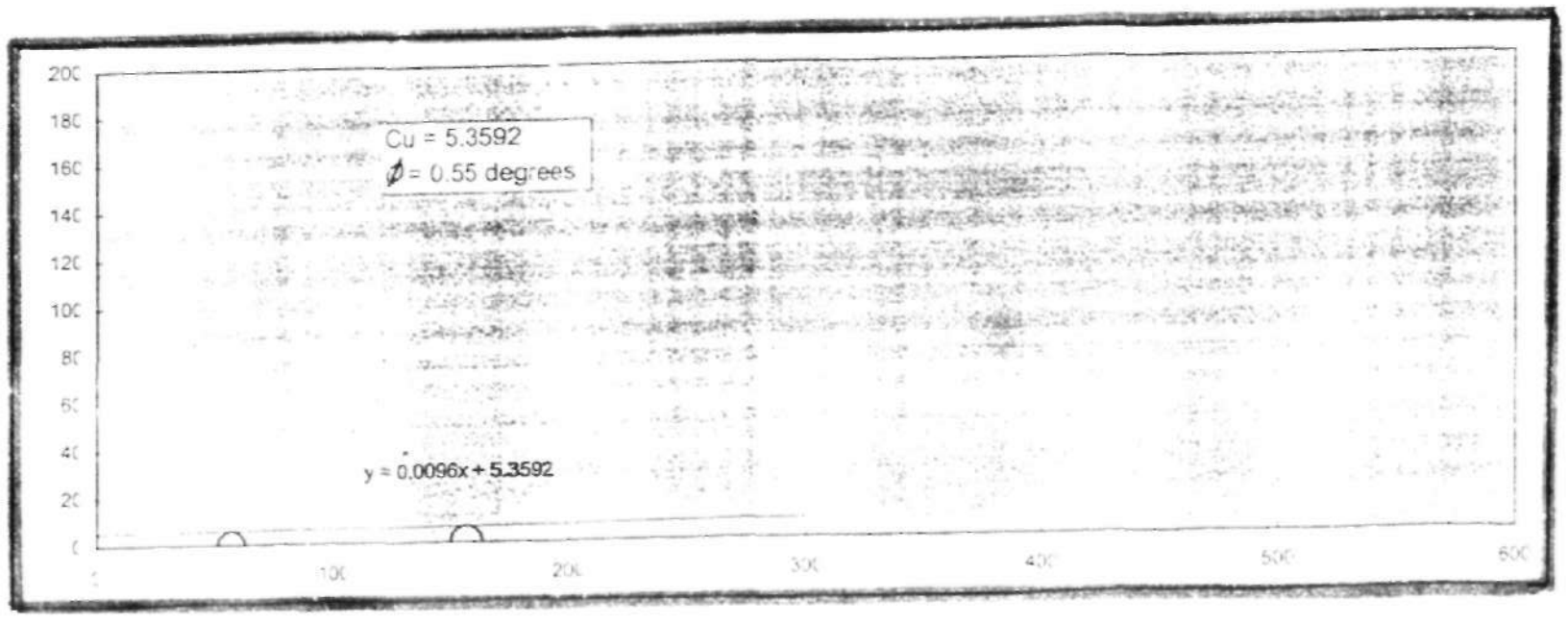


Figure 4.1a: Typical Mohr Circle for Organic Peaty Clay for Borikiri

OPERATOR: Ekpo, A.U

A.B.U. ZARIA

PROJECT: M.Sc./ENG/04742/93

Mohr Circle Parameters

Site: Ogolo-Ama
Borehole No: 1
Sample No: 3
Depth: 2.25-2.50m

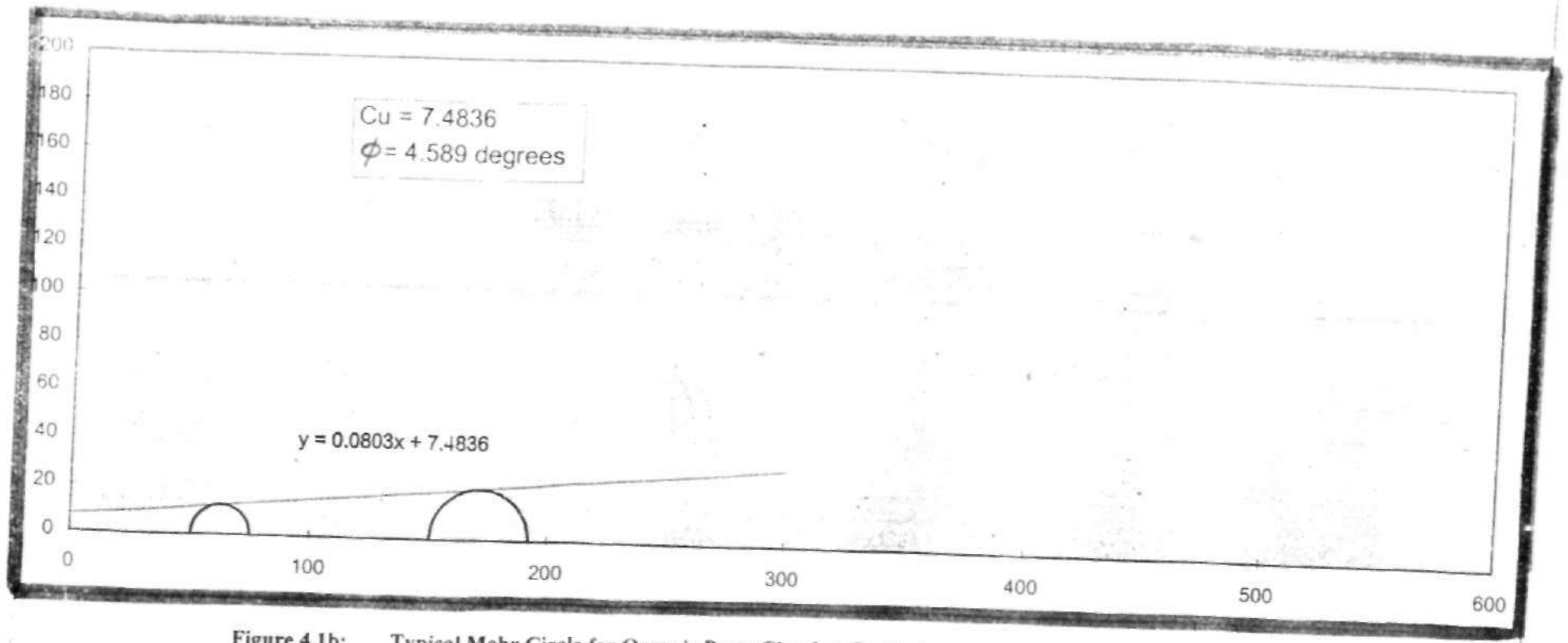


Figure 4.1b: Typical Mohr Circle for Organic Peaty Clay for Ogolu-Ama

LOCATION: Borikiri Sandfill

Table 4.1a

STRENGTH TEST (Unconsolidated Undrained Triaxial Compression Test)

Borehole No.	Depth of Sample (m)	Natural Moisture Content (%)	Bulk Unit Weight (kN/m^3)	Undrained Cohesion (kPa)	Angle of Internal Friction (°)	Lab. Vane Test $Cu(kN/m^2)$	Description of Soil
1	3.00-3.45	144	12.7	23.6	1.175	-	Black soft silty organic peaty CLAY with substantial decomposed plant Debris (Chikoko Mud).
	5.25-5.70	133	12.7	23.8	1.576	-	Dark grey, soft silty organic peaty CLAY with decomposed plant Debris.
	9.00-9.45	20	20.7	17.74	0.785	-	Light bluish grey, very soft, sandy silty CLAY (Marine Mud)
2	2.25-2.70	149	13.0	5.4	0.550	10	Black soft silty organic peaty CLAY with substantial decomposed plant Debris (Chikoko Mud).
	4.40-4.95	61	15.11	13.28	3.595	12	Dark grey, soft silty organic peaty CLAY with decomposed plant Debris.
	9.00-9.45	-	-	-	-	-	Light bluish grey, very soft, sandy silty CLAY (Marine Mud)

LOCATION: Ogolo-Ama Sandfill

Table 4.1b

STRENGTH TEST (Unconsolidated Undrained Triaxial Compression Test)

Borehole No.	Depth of Sample (m)	Natural Moisture Content (%)	Bulk Unit Weight (kN/m^3)	Undrained Cohesion (kPa)	Angle of Internal Friction (^{0})	Lab. Vane Test Cu(kN/m^2)	Description of Soil
1	2.25-2.70	102	13.9	7.5	4.589	6	Black soft silty organic peaty CLAY with substantial decomposed plant Debris (Chikoko Mud).
	4.50-4.95	21	19.7	35.1	0.951	13	Light grey sandy CLAY
	6.00-6.45	16	20.8	54.3	8.737	24	Loose light bluish grey clayey SAND
2	2.25-2.70	148	13.0	5.8	1.662	8	Black soft silty organic peaty CLAY with substantial decomposed plant Debris (Chikoko Mud).
	4.50-4.60	166	-	-	-	-	Light grey, very soft, sandy CLAY
	6.00-6.45	-	-	-	-	-	Firm, light bluish grey clayey SAND

and sand contents, the SPT was carried out both in the cohesionless and the cohesive materials.

To further investigate into the undrained shear strength of the clayey materials, the SPT N-values were used to estimate the range of undrained shear strength of the clays using the correlation curves suggested by Terzaghi and Peck (1967) and Sowers (1979) The undrained shear strength obtained with depth, using the mentioned correlation curves are shown in Table 4.2.

Table 4.2: Undrained Shear Strength Values Using Approximate Correlation between Undrained Shear Strength and Standard Penetration Test N-value (after Terzaghi and Peck,1967 and Sowers, 1979)

Borehole No.	Depth of Sample (m)	Average SPT N-values (N)	Undrained shear strength Cu (kN/m²)	Ranges of undrained shear strength Cu (kN/cm²)	Soil Type
Site: Borikiri			**(Terzaghi & Peck, 1967)**	**(Sowers, 1979)**	Correlation Grading
1	1.50-2.25	4	30.0	10-20	SC
	3.75-4.50	2	5.0	10	CH
	6.00-8.25	4	10.0	4-10	SC-ML
	9.75-12.00	3	15.0	7-20	SC-ML
2	3.00-4.40	2	10.0	5-10	CH
	5.25-8.25	3	15.0	7-15	SC-ML
	9.75-13.00	6	35.0	15-25	SC-ML
Site: Ogolo-Ama			**(Terzaghi & Peck, 1967)**	**(Sowers, 1979)**	
1	3.00-4.50	1	5.0	10	CH
	5.25-6.00	2	10.0	4-10	SC
	6.75-10.50	9	55.0	20-40	SC-ML
	10.50-15.00	12	75.0	30-55	SC-ML
2	3.00-4.50	2	10.0	4-10	SC
	4.50-5.25	4	25.0	10-20	SC-ML
	6.75-10.50	9	55.0	20-40	SC-ML
	10.50-15.00	13	80.0	30-60	SC-ML

4.1.3 Evaluation of shear strength from Atterberg limits

he consistency limits as indicators of soil behaviour can be used for the estimation of the shear strength of weak clay soils. This has been investigated by many researchers {Tomlison, 1986; Smith, 1990; Carter and Bentley, 1991 etc} and is easily justified from the Alterberg limit tests methods. For instance, the Casagrande procedure models a slope failure due to dynamic loading under quick undrained conditions (Carter and Bentley, 1991).

The authors also pointed out that the shear strength of the specimen is progressively reduced by increasing its moisture content until a specific energy input in the form of standard taps, causes a failure of standard slope in a defined manner. Also, the alternative cone method, is also an indirect shear strength test which models bearing failure under quick undrained conditions. The consequence of the test procedures is that all soils at their liquid limit exhibit the same value of undrained shear strength. Casagrande (1932) estimated this value as 2.6 kN/m^2, and later work by Skempton and Northey (1952) indicated values of 1-2 kN/m^2. It has been reported by a number of researchers(Smith, 1990, Skempton and Northey (1952) Carter and Bentley, 1991 etc) that all soils at the plastic limit exhibit similar values of undrained shear strength as being 100-200 kN/m^2.

Generally, the liquid and plastic limits are moisture contents at which soil has specific values of undrained shear strength. It therefore follows that for a remoulded soil, the shear strength depends on the values of the natural moisture content in relation to the liquid and plastic limit values. This can be conveniently expressed by using the concept of liquidity index defined by:

$$\textit{Liquidity Index } I_l = \frac{w - PL}{LL - PL} = \frac{w - PL}{PI} \qquad 4.5$$

Where w = Natural Moisture Content

PL = Plasticity Limit

LL = Liquid Limit

PI = Plasticity Index

Based on the definition of the liquidity index, consistency limits are all incorporated in the formula. Thus in selecting the correlation between the undrained shear strength of

the weak clays and the Atterberg limits, the utilisation of the liquidity index presents the best option. In the analysis, the liquidity index calculated for the weak organic clays were used to obtain the undrained shear strength of the weak soils from the correlation between shear strength and liquidity index (after Skempton and Northey, 1952). The results obtained from the analysis of the undrained shear strength using the mentioned correlation curves are summarised in Table 4.3a and Table 4.3b for Borikiri and Ogolo-Ama sand filled areas, respectively.

4.1.4 Choice of undrained shear strength for design

A study of the results obtained from the analysis of the undrained shear strength from the various methods, shows a close relationship in the strength characteristics of the organic clays. From the results presented on the relevant tables, the undrained shear strength of the organic peaty clay immediately underlying the fill is less than 40 kN/m^2. This classified the clay as very soft to soft in consistency (Carter and Symons, 1989; Smith, 1990). Smith (1990), proposed that a clay with an undrained shear strength, C_u, of not more than 40 kN/m^2 is classified as a soft clay and will be normally consolidated (or slightly overconsolidated). Nevertheless, it is essential to note that although the three methods of analyses reflect the weak strength of the organic clays underlying the sandfill material. For design purpose clay the undrained shear strength obtained from quick undrained triaxial test and laboratory vane test were utilized due to the limitation of the SPT results and the Alterberg limits test.

Basically, in the SPT test, the actual behaviour of the soil is not simulated, the results reflect only the state of compaction or densification of the soil and this can only give a rough estimate of the soil strength. Practically, there are always limitations on the use of consistency limits. The Atterberg limits tests are performed on the material finer than

Table 4.3a: Evaluation of Undrained Shear Strength using the correlation between Cu and Liquidity Index (after Skempton and Northey, 1952)

Borikiri Sandfill							
Borehole No.	Depth of Sample (m)	Natural Moisture Content (Wn) (%)	Liquid Limit LL (%)	Plastic Limit PL (%)	Plasticity Index PI (%)	Liquidity Index (Wn-PL)/PI	Range of Undrained Shear Strength Cu (kN/m2)
1	2.25-2.70	167	-	-	-	-	-
	3.00-3.45	184	182	68	114	1.02	0.9-1.7
	5.25-5.70	205	203	62	141	1.0	1.0-2.0
	8.25-8.70	30	29	10	19	1.1	0.8-1.5
	9.00-9.75	26	32	11	21	0.7	3.0-4.0
	9.75-10.20	32	24	NP	-	-	-
2	2.25-2.70	170	173	61	112	1.0	0.9-1.7
	3.00-3.45	166	170	61	109	1.0	0.9-1.7
	4.50-4.95	267	268	46	222	1.0	0.9-1.7
	6.00-6.75	70	70	16	54	1.0	0.9-1.7
	9.00-9.45	31	29	9	20	1.1	0.8-1.5
	9.75-10.20	19	28	9	19	0.5	7.0-10.3

Table 4.3b: Evaluation of Undrained Shear Strength using the correlation between Cu and Liquidity Index (after Skempton and Northey, 1992)

Ogolo-Ama							
Borehole No.	Depth of Sample (m)	Natural Moisture Content (Wn) (%)		Plastic Limit PL (%)	Plasticity Index PI (%)	Liquidity Index (Wn-PL) PI	Range of Undrained Shear Strength Cu (kN/m2)
1	2.25-2.70	116	166	37	123	0.6	4.0-7.0
	4.50-4.95	46	44	17	27	1.1	0.8-1.5
	6.00-6.45	42	32	14	18	1.5	0.2-0.7
	7.50-8.20	120	136	58	78	0.8	2.0-3.0
2	2.25-2.70	170	173	61	112	0.9	1.6-2.7
	4.50-4.95	57	58	19	39	0.9	1.6-2.7
	6.00-6.45	40	40	18	22	1.0	1.0-2.0

425 micron, and the degree to which this fraction reflects the properties of the soil depends on the proportion of coarse material present and on the precise grading of the soil. Another limitation is that the limit tests are performed on remoulded soils and the correlation are not generally valid for undisturbed soils unless the soil properties do not change substantially during remoulding. This is the case with many normally consolidated clays. Sensitive clays and cemented soils often differ markedly from those predicted from Atterberg limit tests. Besides, the undrained shear strength results from correlations are usually considered as rough estimate, .thus are only useful for preliminary design.

Generally, from the results presented so far on the strength characteristics of the weak soils in the sandfilled areas, the weak organic clays are soft, either normal or slightly over-consolidated, and from their geologic location are always saturated. Thus, for any foundation design problem in the reclaimed areas of the Niger delta, the ideal undrained shear strength parameters are those obtained from the quick undrained triaxial test and the laboratory vane test results. The slight difference in the values of shear strength obtained from the unconfined compression testing or triaxial testing and the laboratory vane test results can be attributed to the influence of geological history and the effect of stress history during testing on the triaxial test machine.(Wroth, 1984).

4.2 Compressibility Of Weak Soils

4.2.1 Theoretical background

The compressibility characteristics of weak soils due to application of foundation load is generally considered from the consolidation point of view. Published work on consolidation of weak soils are numerous as it forms an integral part of the design of

foundations of structures. For the purpose of this study, only those work related to the soil condition in the reclaimed areas of the Niger delta are considered.

Cargrill (1984) stated that there are basically two alternatives to the geotechnical engineer faced with the problems of predicting the time dependent one dimensional 1-D primary consolidation behaviour of saturated thick layers of soft fine-grained natural soils:

- Solution chart, i.e percentage consolidation versus dimensionless time factor

- Computer programme which solves the consolidation governing equation with some specified initial boundary conditions.

It was pointed out that, the derivation of the solution chart relationship involves a dimensionless solution of the conventional Terzaghi equation

$$\frac{C_v \partial u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad 4.5$$

The original theory of consolidation (Terzaghi, 1943) is based upon the principle of consolidation of mass continuity of a porous medium in which the volume changes as 1-D and strains and flow velocities are infinitesimal. Despite its deficiencies and lack of credibility (Lambe, 1973; Peck, 1967), this theory has been applied in many ways, and is widely used in geotechnical engineering practice.

Usually, consolidation characteristics of a soil are defined in terms of the coefficient of volume compressibility, m_v, which is a measure of the rate of consolidation (Carter and Symons, 1989). They further pointed out that some soils such as fissured clays and stratified estuarine silts have a macrostructure which allows water to drain away relatively free. As a result of this, consolidation takes place at a much higher rate than is predicted by C_v values obtained in the laboratory tests; time factors of 10:1 are common whereas factors as high as 100:1 are not unknown. In such soils it is better to calculate C_v

from the results of field test using the formula:

$$C_v = \frac{k}{\gamma_w m_v} \qquad 4.6$$

where κ is the field coefficient of permeability
γ_w is the unit weight of water
m_v is the coefficient of volume compressibility obtained from the laboratory test

Hydraulic sandfill areas in the Niger delta are characterised by stratified estuarine silts and clays with macrostructures which make them highly permeable. For instance, as confirmed from the field investigation, the ground water levels vary with the tidal variations. This implies that the deposits including the organic clays are highly permeable (Enoch Associates, 1989). Hence Equation 4.6 was adopted for the calculation of the coefficient of permeability. However, it was impossible to obtain the field coefficient of permeability due to limited equipment available for the research work. K was made the subject of the Equation 4.6, and was obtained after the evaluation of other compassibility parameter (C_T, m_v and γ_w) as:

$$k = m_v \gamma_w / C_v \qquad 4.7$$

4.2.2 Determination of compressibility by parameters

As expected with advancement in soil mechanics and geotechnical engineering activities, there are numerous contributions for and against the existing method of compressibility analysis of weak soils based on the 1 - D Consolidation theory (Rowe 1968 and 1972; De Beer 1965; etc). Nevertheless, in our present stage of development, the compressibility of clays is usually measured by means of oedometer tests or similar methods as presented by Tavenas and Leroueil(1987). Results may be expressed in a number of ways, leading to sometimes confusing variety of compressibility parameters, but the basic principle is, from the consolidation test results the specimen thickness is usually plotted (to a linear

scale) against the effective pressure (to a logarithmic scale) or e/pressure; the two methods are exactly equivalent, since height is proportional to volume (Carter and Symons, 1989).

The author further pointed out that the e/p plots express the relationship in a more fundamental form but the direct plot of thickness against pressure avoids unnecessary calculations and the need to measure or assume a value for Gs, the specific gravity of the soil solids, which is needed to calculate the void ratio (e). Values of M_v can be obtained from the plot. The value of m_v obtained will depend on the pressure range that corresponds to field conditions. Beneath a proposed foundation the initial pressure of the soil at a given depth will be the overburden pressure plus the extra pressure due to the foundation loading which is the pressure range used when calculating M_v values. In deciding on the choice of method of analysis, it was essential to consider:

(i) the requirement of BS 1377 (1990) which emphasises the use of e/logP instead of the direct method of height/pressure. Consequently, this method was adopted for the analysis of C_c.

ii) For the sandfilled areas investigated, the weak soils are often saturated due to the high water table; thus in the analysis of the consolidation test results, the clay specimen were assumed to be fully saturated ($S_r = 100\%$) for all calculations.

The oedometer test results obtained for the two investigated sites were analysed in similar ways. However, for clarity, the compressibility analysis was executed in stages, based on the one-dimensional consolidation theory. In the first stage of the analysis, the coefficient of consolidation (C_v) was obtained directly from oedometer test results using various methods of C_v analyses while in the second stage of the analysis, other compressibility parameters (M_v, k etc.) were evaluated. The method of compressibility analysis adopted,

the steps taken and the results obtained are as described in the subsequent sections.

4.2.2.1 Coefficient of consolidation (C_v)

Frequently, for a given load increment on a sample, there are two commonly used graphic methods for determining C_v from laboratory one-dimensional consolidation test. These include the logarithm-of-time method proposed by Casagrande and Fadum (1940) and the other is the Square-root-of-time method suggested by Taylor, 1942. The two methods form the basis of C_v analysis and the result have proved very reliable over the years. However, with progress in geotechnical engineering works, other methods have been proffered by many authors (Scott R.F., 1962; Sridharan and Prakash, 1985; etc.). For this research work, in addition to the square - root - of - time method and the logarithm - of - time method, the "Hyperbola" method of C_v analysis was utilized for comparison of results.

Once a load increment was applied to the soil in the consolidation test, values of specimen thickness were taken at set times so that a plot could be made of thickness against time. In the Square-root-of-time method, specimen thickness was plotted against the square root of time, while is the logarithm of time method, the specimen thickness was plotted against the logarithm of time. All the graphic construction required to evaluate the C_v values from the graphs are as presented in the various soil mechanics and geotechnical engineering textbooks (Tomlinson, 1986; Das, 1989; Smith, 1990). The C_v values obtained using the square root of time fitting method are given in Table 4.4., while the C_v values obtained from the log-of-time method of analysis is summarised in Table 4.5 for the two investigated sites.

Table 4.4 - C_v Values using Square – root – of – time method

Location: Borikiri							
B/H No.	Depth of Sample (m)	Pressure range (kN/m^2)	Coefficient of consolidation C_v (m^2/yr)	B/H No.	Depth of Sample (m)	Pressure range (kN/m^2)	Coefficient of consolidation C_v (m^2/yr)
1	3.00-3.45	0.25	0.01	1	2.25-2.70	0-12.50	4.50
		25-50	0.30			12.50-25	6.72
		50-100	0.22			25.50	5.69
		100-200	0.28			50-100	2.90
		200-400	0.29			100-200	4.34
		400-800	0.21			200-400	2.62
		800-1600	0.29				
	5.25-5.70	0-25	0.59	2	4.50-4.95	0-6.25	6.52
		25-50	0.33			6.25-12.50	2.36
		50-100	0.40			12.50-25	0.12
		100-200	0.22			25-50	0.14
		200-400	0.35			50-100	0.21
	9.00-9.45	0.25	3.81		9.00-9.45	0-25	7.84
		25-50	2.47			25-50	18.63
		50-100	1.46			50-100	10.32
		100-200	1.83			100-200	20.65
		200-400	2.08			200-400	23.12
		400-800	3.17			400-800	21.25
		800-1600	3.11			800-1600	20.70
Location: Ogolo-Ama							
1	2.25-2.70	0-25	1.14	2	2.25-2.700	0-12.50	6.69
		25-50	1.87			12.50-25	5.65
		50-100	0.13			25-50	2.89
		100-200	0.14			50-100	4.32
		200-400	0.29			100-200	3.6
	4.50-4.95	0-25	1.82			200-400	2.22
		25-50	1.31		6.00-6.45	0-25	7.86
		50-100	6.18			25-50	18.71
		100-200	10.17			50-100	10.37
		200-400	15.36			100-200	20.75
		400-800	4.24			200-400	23.59
		800-1600	7.84			400-800	21.33
	6.00-6.45	0-25	3.09			800-1600	20.77
		25-50	4.13				
		50-100	8.22				
		100-200	7.05				
		200-400	15.39				
		400-800	1.536				
		800-1600	24.68				

Table 4.5 - Log-of-time method of C_v Analysis

B/H N	Depth of Sample (m)	Pressure range (kN/m²)	Coefficient of consolidation C_v (m²/yr)	B/H N	Depth of Sample (m)	Pressure range (kN/m²)	Coefficient of consolidation C_v (m²/yr)
Location: Borikiri							
1	3.00-3.45	0-25	0.359	3	2.25-2.70	0-12.50	1.304
		25-50	0.208			12.50-25	0.292
		50-100	0.205			25-50	0.519
		100-200	0.284			50-100	0.351
		200-400	0.221			100-200	0.413
		400-800	0.226			200-400	0.176
		800-1600	0.350				
	5.25-5.70	0-25	0.768		4.50-4.95	0-6.25	0.176
		25-50	0.524			6.25-12.5	0.100
		50-100	0.353			12.50-25	0.130
		100-200	0.161			25-50	0.207
		200-400	0.123			50-100	
	9.00-9.45	0-25	2.32		9.00-9.45	0-25	4.128
		25-50	2.52			25-50	4.123
		50-100	3.92			50-100	4.317
		100-200	3.25			100-200	4.128
		200-400	1.52			200-400	5.565
		400-800	4.13			400-800	2.92
		800-1600	2.46			800-1600	0.582
Location: Ogolo-Ama							
1	2.25-2.70	0-25	1.036	2	2.25-2.70	0-12.50	2.320
		25-50	0.200			12.50-25	0.584
		50-100	0.189			25-50	1.038
		100-200	0.208			50-100	0.702
		200-400				100-200	0.826
	4.50-4.95	0-25	0.733			200-400	0.352
		25-50	0.654		6.00-6.45	0-25	4.256
		50-100	3.286			25-50	4.256
		100-200	2.604			50-100	4.634
		200-400	3.277			100-200	4.256
		400-800	3.852			200-400	5.574
		800-1600	3.677			400-800	5.840
	6.00-6.45	0-25	2.318			800-1600	1.164
		25-50	3.068				
		50-100	5.194				
		100-200	3.675				
		200-400	2.920				
		400-800	3.280				
		800-1600	4.220				

Hyperbola" method: One of the most recent empirical methods of C_v analysis using the consolidation test results is the hyperbola method. The procedure for the determination of the C_v values by hyperbola method adopted in this work is based on the method investigated and presented by Das (1989). In the hyperbola method, the following procedure is recommended for determination of C_v:

(i) The time (t) and specimen deformation ($_{\Delta}H_1$) from the laboratory oedometer consolidation test results were computed.

(ii) The graph of $t/_{\Delta}H$ against t, as shown in fig 4.2 below were plotted.

(iii) The straight line portion of the graph (bc) was identified, and projected back to cut the $t/_{\Delta}H$ axis at a point D; the intercept D was thus determined

(iv) Slope (m) of the line bc were determined

(v) From the parameters obtained, the computation of C_v values was done using the formula:

$$C_v = \frac{0.3mH_{dr}^2}{D} \qquad 3.7$$

where H_{dr} = the length of the drainage path.

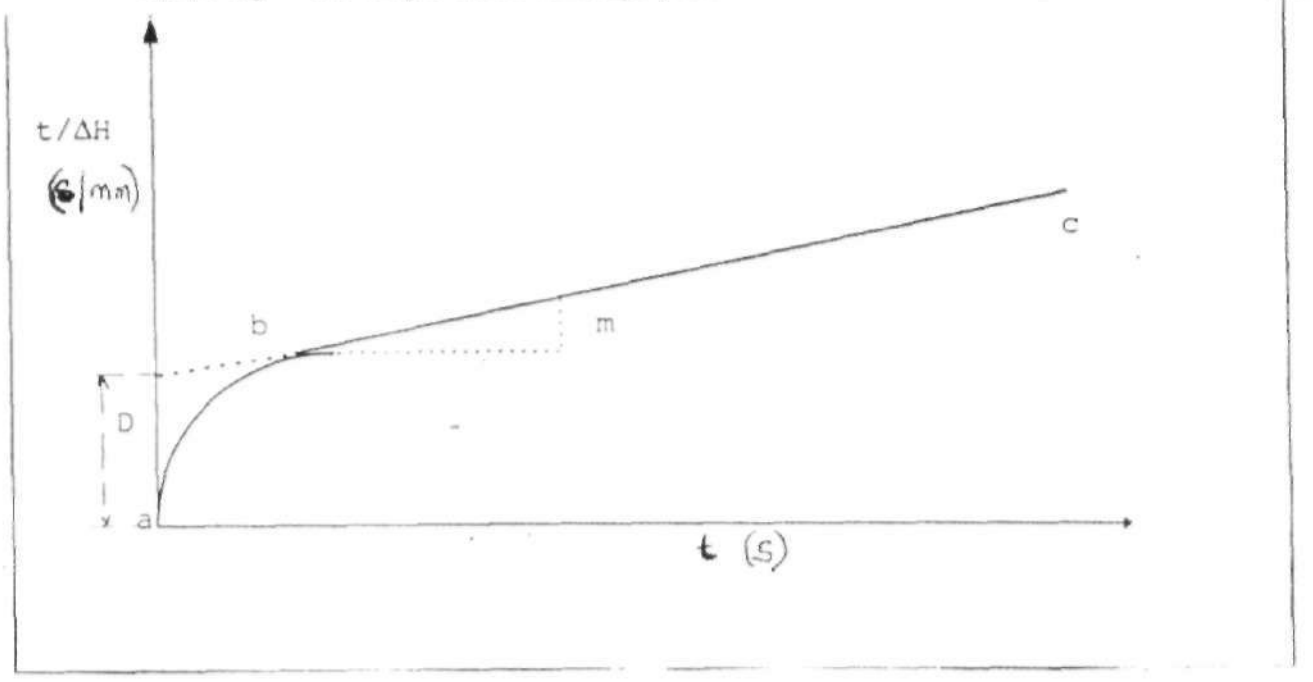


Figure 4.2: Hyperbola Method for determination of Coefficient of Consolidation

The summary of C_v values obtained using the Hyperbola Method of analysis is presented in Table 4.6 for two locations.

Table 4.6 - C_v Values using Hyperbola method

Location: Borikiri							
B/H No.	Depth of Sample (m)	Pressure range (kN/m^2)	Coefficient of consolidation C_v (M^2/yr)	B/H No	Depth of Sample (m)	Pressure range (kN/m^2)	Coefficient of consolidation C_v (M^2/yr)
1	3.00-3.45	0-25	0.396	2	2.25-2.70	0-12.50	0.764
		25-50	0.104			12.50-25	0.371
		50-100	0.174			25-50	0.263
		100-200	0.188			50-100	0.174
		200-400	0.248			100-200	0.208
		400-800	0.200			200-400	
		800-1600	0.261				
	5.25-5.70	0-25	0.972		4.50-4.95	0-6.25	0.167
		25-50	0.288			6.25-12.50	0.134
		50-100	0.212			12.50-25	0.177
		100-200	0.081			25-50	0.220
		200-400	0.149			50-100	-
		400-800	2.478				
	9.00-9.45	0-25	3.372		9.00-9.45	0-25	1.248
		25-50	1.628			25-50	1.790
		50-100	5.001			50-100	1.216
		100-200	2.535			100-200	1.045
		200-400	2.935			200-400	1.780
		400-800	4.169			400-800	3.221
		800-1600	2.675			800-1600	1.444
Location: Ogolo-Ama							
1	2.25-2.70	0-25	1.420	2	2.25-2.700	0-12.50	0.541
		25-50	0.400			12.50-25	0.466
		50-100	0.237			25-50	0.327
		100-200	0.193			50-100	0.174
		200-400	-			100-200	0.207
	4.50-4.95	0-25	1.118			200-400	-
		25-50	0.669		6.00-6.45	0-25	5.367
		50-100	1.001			25-50	2.908
		100-200	1.742			50-100	1.053
		200-400	2.873			100-200	0.784
		400-800	1.218			200-400	3.165
		800-1600	1.899			400-800	0.989
	6.00-6.45	0-25	2.713			800-1600	0.570
		25-50	0.761				
		50-100	0.460				
		100-200	3.028				
		200-400	0.871				
		400-800	1.536				
		800-1600	1.179				

4.2.2.2 Evaluation of other compressibility parameters

To obtain the other compressibility parameters, m_v and k, the process is often based on the "Model Soil Sample analysis" as described in the various Soil Mechanic and Geotechnical Engineering textbooks. The procedure was divided into simple calculation steps. For each consolidation test result, after obtaining the C_v values, the steps taken in

typical oedometer test data. The summary of the compressibility results obtained are presented in **Appendices D**1 and **D**2 for the two sites respectively.

4.3 Choice of Compressibility Analysis for Design

The choice of compressibility parameters to be adopted for the foundation design of structures considered in this work is essential as it will enhance a more precise estimation of the foundation settlement and the rate at which the settlement will occur. In deciding on which compressibility parameter to be utilised, the following factors were examined:

(a) the degree of involvement of the soil properties in the derivation of the compressibility parameters;

(b) the accuracy of the method which can be assessed from the performance of the structures from existing soil investigation reports and the actual results obtained from the analysis (**Appendices D**1 and **D**2). The ability of any given method to reflect change of layers of material from the C_v values obtained will also be considered; the ease of application of each method of analysis which also include the time required for the evaluation of the compressibility parameters etc.

Most of these factors have been thoroughly scrutinized by the various authors on compressibility analysis. Nevertheless, the choice of method of analysis for any foundation design purpose is often dependent on the soil in question due to the heterogeneous nature of soils. For instance, the weak organic clays of the reclaimed areas of the Niger delta may have certain unique properties due to the high moisture content and the substantial amount of decaying vegetation in the soil. These unique properties may account for the inconsistent results of a given method of compressibility analysis that may be adequate for other types of clays.

However, based on the compressibility results presented in **Appendices C**1 and **C**2 and

However, based on the compressibility results presented in **Appendices C_1 and C_2** and the existing published work on the compressibility characteristics of the weak soils of the sandfilled areas of the Niger delta, a critical assessment of the soils parameters show a very close relationship in the C_v values obtained from the Square-root-of-time method, the Log-of-time fitting method and the Hyperbola method of analysis. The m_v and k values evaluated from these three methods are also similar. But in considering the ease of application, the time involved in the evaluation of the compressibility parameters from the graphic constructions and the variability of the C_v values in the peaty clays and sandy clays of the sandfilled areas, the Square-root-of time method presents the best option.

For the hyperbola method of C_v analysis, apart from the time consuming method of evaluating $_{\Delta}H$ and $t/_{\Delta}H$, it is considered an empirical method and thus can only be used for preliminary design.

A survey of existing work on the compressibility parameters obtained from the weak soils of the sandfilled areas shows that the Square-root-of-time fitting method of C_v analysis has proved very reliable. Hence, for the foundation design that will be considered in the next chapter, the compressibility parameters based on the C_v values derived from the Square-root-of-time method of analysis will be used for the estimation of the foundation settlement.

Generally, the C_v values obtained from the three methods of analysis utilised in this work are in agreement with the compressibility characteristic of the reclaimed areas (George and Abam (1990); Enoch Associates, 1989; Trevi, 1993 etc). Usually, the choice of any particular method of analysis for the evaluation of design parameters is in most cases a matter of preference, depending on the objectives and the discretion of the engineer in charge of the design.

Chapter 5

FOUNDATION SETTLEMENT ANALYSIS

5.1 General Consideration

One of the problems a geotechnical engineer is often faced with in the design of engineering structures involve the estimation of the expected settlement. This problem is essentially due to the heterogenous nature of soils. For this reason, settlement results are often given in ranges but never an exact value. The problem is more complicated for a reclaimed land such as the sandfilled areas under consideration, where environmental and developmental factors contribute immensely to the settlement problems.

Osinubi (1992) observed the fact that in spite of our present advanced analytical capabilities, our ability to predict settlement of buildings is in many cases inadequate. Settlements are often erroneously separated into two phases,'construction and occupancy period. The author looked into the mentioned inadequacies on the basis of actual field observation of different 5-9 storey building founded on hydraulic sand fills with underlying weak strata. It was pointed out that for shallow foundation on hydraulic sandfill with underlying weak soil, the best bearing layer is the fill material, since the underlying weak soil is often very weak and highly compressible for any structural load.

Similarly, majority of the soil investigation reports covering the hydraulic sandfilled areas of the Niger delta (Genatec, 1988; OAIM, 1986 and 1988; and Enoch George-Associates, 1986 and 1988 etc.) jointly pointed out that the computations employed to predict settlement are at best, only approximate values because of several limitations. The soil reports strongly advised on the monitoring of actual settlement after the placement of the hydraulic sand fill. Furthermore, soil reports also revealed that the settlement of the unloaded reclaimed area may have two components to be considered:

(i) the settlement of the fill material (sand) under self weight

(ii) the settlement of the soil supporting the fill material (peaty clays).

Besides, the peaty clay would settle considerably over many years as result of

(i) consolidation

(ii) lateral plastic yielding

(iii) decomposition of organic matter

According to the soils reports, the total settlement of the peaty clay under load may be up to 66% of its thickness. The differential settlement would also occur in view of the difference in the peaty clay thickness as observed within any given site.

However, due to the timing of this project, and the ignorance of the landowners in the vicinity of the distressed buildings investigated for this work, the actual field monitoring of settlement in the sandfilled areas was not possible. Pegs erected and the surveyors marks at the base of the distressed buildings were removed by the landlords irrespective of explanations given. For these reasons, settlements results obtained were based on the penetration test results and the analysis of the samples' collected during the site investigation.

With regards to the two layered foundation bases under consideration for shallow foundation on the sandfilled areas (sandfill and the organic peaty clays) the various approaches adopted and the results obtained are as described below:

5.1.1 Evaluation of foundation settlement on sandfilled areas

In evaluating the foundation settlement in the sandfilled areas investigated, the following factors were considered:

(i) the type of foundation used for the distressed buildings

(ii) expected foundation load from the distressed buildings

(iii) the ultimate bearing strength of the subsoil

(iv) the thickness of the sandfill material

(v) the thickness of the compressible weak soils affected by the structural load the depth of the soil affected by the foundation load the depth of the foundation

For the residential building at Borikiri, and the classroom blocks at Ogolo-Ama, strip foundation was used. The structural load for the residential building is about 36 kN/m^2 while that of the classroom block was estimated as 45 kN/m^2. The load were on the high side with respect to the size of the buildings. This is due to the fact that solid blocks were used for the construction of the load bearing walls instead of the usual hollow blocks. The foundation depths for the two buildings as revealed by excavation and the structural drawing are 0.7 m and 1.0 m for the residential building and the classroom blocks, respectively.

In the sandfilled areas investigated, although the foundations were placed on the fill material (sand), the distance between the foundation base and the peaty clays is relatively small thus, substantial part of the foundation load are on the highly compressible organic peaty clays. The terminating depths of the fill material as well as the thickness of the underlying weak clays were highly variable around any given building.

5.1.2 Estimation of the ultimate bearing capacity

A number of methods exist for calculating the ultimate bearing capacity but the most commonly used are those proposed by Terzaghi and Peck (1967); Meyerhof (1956) and Hansen (1978). However, for this work, the Terzaghi's bearing capacity formula is used for the computation of the bearing capacity of the subsoil layers. From the basic premise of shear failure along an assumed surface Terzaghi developed the following expressions

for the net ultimate bearing capacity, q_{nu}, of a shallow strip foundation

$$q_{nu} = CN_c + P_o(N_q - ?) + \frac{1}{2}\gamma BN_\gamma \qquad 5.1$$

Where γ is the unit weight density of the soil below the foundation level
C is the cohesive strength of the soil
P_0 is the effective overburden pressure at foundation level
B is the foundation width
N_c, N_q and N_γ are the Terzaghi bearing capacity factors.

Meyerhof's approach is similar to Terzaghi's but it takes into account the shear strength of the soil above the founding depth and the different modes of failure of shallow and deep foundations. The bearing capacity factors also take into account the shape of the foundation. This makes the method more accurate and more comprehensive than Terzaghi's method but less simple to use. The basic equation for bearing capacity is:

$$q_{nu} = CN_c + P_o(N_q - 1/2) + \frac{1}{2}\gamma BN_\gamma \qquad 5.2$$

where N_c, N_q and N_γ are Meyerhof's bearing capacity factors. Other parameters is as defined in Equation 5.1.

For saturated clays, using a total stress analysis, the soil typically behaves as a purely cohesive material. In this case, the bearing capacity equation reduces to:

$$q_{nu} = CN_cS \qquad 5.3$$

where bearing capacity factor N_c is obtained from values of N_c for foundation on a purely cohesive soil (Meyerhof, 1956).

Meyerhof's method is usually preferred for foundations on clay since it is easy to use and models the conditions more accurately than Terzaghi's method.

In utilising these two bearing capacity formulae, the following notes were considered:

(i) since in the sandfilled areas, the water table is above the founding level, the values of γ used in the bearing capacity calculation is the submerged unit weight (γ_{sub})

(ii) before the bearing capacity coefficients were obtained, values for the angle of shearing resistance were determined. For the fill material (sand) the values were estimated from correlation curve between relative density, N – values and ϕ for sands (Peck, *et al* 1967) while for the weak clays the values of ϕ_u were obtained from the Mohr circle results.

5.1.3 Calculation of allowable bearing pressure

(a) Borikiri Sandfill

1.0 Soil Conditions: considering the 2 boreholes and 8 cone penetration tests the soil properties are as follows:

GL-1.50m :Medium to Dense SAND FILL

1.50-5.50m :Soft Organic peaty CLAY with substantial decomposed plant debris

5.50-12.00m :Very Soft Very Silty CLAY becoming sandy with depth

1.1 Soil Properties:

GL-1.50 m N = 13

γ = 20.0 kN/m^3

C_u = 0 kN/m^2

n = 31^0

1.50-5.70m γ = 13.38 kN/m^3

C_u = 16.52 kN/m^2

n = 2^0

5.70-12m γ = 20.85 kN/m^3

C_u = 63.48 kN/m^2

n = 3^0

2.0 Bearing Capacity

General average ground water level for Borikiri is GWL = 0.6 m below ground surface.

Foundation Dimension (strip) = LxB = 12.00x1.0m

2.1 Layer 1 - Medium Dense SAND

Terzaghi's equation for the net ultimate bearing capacity:

q_{nu} of the shallow strip foundation is given by

$$q_{nu} = C N_c + P_o (N_q - 1) + 1/2 \gamma B N_\gamma$$

From the data given above, C = 0

$$q_{nu} = P_0 (N_q - 1) + 1/2 \gamma B N_\gamma$$

$$P_0 = \gamma z - \gamma_w z_w = (20x1 - 9.8x0.6) = 14.12 \text{ kN/m}^2$$

From Chart, Terzaghi bearing capacity factors for

$\phi = 31^0$ are: Nc = 50 Nq = 26 N_γ = 25

$$q_{nu} = 14.12(26-1) + 1/2x20.0x1.0x25 = 353 + 250 = 603 \text{ kN/m}^2$$

Allowable bearing pressure $q_a = q_{nu}/Fs$

Fs = 3 for foundation works

$$q_a = 603/3 = 201 \text{ kN/m}^2$$

2.2 Layer 2

For saturated clays, using a total stress analysis, the soil typically behaves as a purely cohesive material. In this case, the Meyerhof bearing capacity equation reduces to:

$$q_{nu} = C N_c$$

Where bearing capacity factor N_c is obtained from relevant chart.

$C_u = 16.52 \text{ kN/m}^2$ $N_c = 6.5$

$q_{nu} = 16.52x6.5 = 107\ kN/m^2$

$q_a = 107/3 = 36\ kN/m^2$

2.3 Layer 3

$C_u = 63.48\ kN/m^2$ $N_c = 6.5$

$q_{nu} = 63.48x6.5 = 409.5\ kN/m^2$

$q_a = 409.5/3 = 136.5\ kN/m^2$

(b) Ogolo-Ama Sandfill

1.0 Soil Conditions: considering the 2 boreholes and 8 cone penetration tests the soil properties are as follows:

GL-1.50m	Medium Dense SAND FILL
1.50-5.00m	Soft Organic Peaty Clay with Substantia Decomposed Plant Debris
5.00-7.50m	Very Soft Very Silty CLAY becoming sandy with depth
7.50-15.00m	Very loose, Clayey (MC) SAND with Clay Content Decreasing with Depth

1.1 Soil Properties:

GL-1.50 m	$\bar{N} = 14$
	$\gamma = 22.0\ kN/m^3$
	$C_u = 0\ kN/m^2$
	$\phi = 31.3^0$
1.50-5.00m	$\gamma = 15.53\ kN/m^3$
	$C_u = 16.13\ kN/m^2$
	$\phi = 2^0$

5.00-7.50m γ = 20.13 kN/m^3

C_u = 81.15 kN/m^2

$\phi = 6^0$

7.50-15m N = 17

γ = 22 kN/m^3

C_u = 0 kN/m^2

$\phi = 32^0$

Bearing Capacity

General average ground water level for Ogolo-Ama is GWL = 0.8m below ground surface.

Foundation Dimension (strip) = LxB = 10.00x0.7m

2.1 Layer 1 - Medium Dense SAND

Terzaghi's equation for the net ultimate bearing capacity:

q_{nu} of the shallow strip foundation is given by

$$q_{nu} = C\,N_c + P_0\,(N_q-1) + 1/2\,\gamma\,B\,N_\gamma$$

From the data given above, C = 0

$$q_{nu} = P_0\,(N_q-1) + 1/2\,\gamma\,B\,N_\gamma$$

$P_0 = \gamma z - g_w z_w$ = (22x1-9.8x0.6) = 15.14 kN/m^2

From figure (4.2) Terzaghi bearing capacity factors for ϕ = 31.3^0 are: Nc = 40, Nq = 26, N_γ = 25

q_{nu} = 15.14(26-1) + 1/2x22.0x0.7x25 = 378.5+192.5 = 571 kN/m^2

Allowable bearing pressure $q_a = q_{nu}/Fs$

Fs = 3 for foundation works

$q_a = 571/3 = 190\ kN/m^2$

2.2 Layer 2 - Organic Peaty Clay

Meyerhof bearing capacity equation:

$q_{nu} = C\ N_c$

Where bearing capacity factor N_c is obtained from figure 4.3

$C_u = 16.13\ kN/m^2 \qquad N_c = 6.5$

$q_{nu} = 16.13x6.5 = 105\ kN/m^2$

$q_a = 105/3 = 35\ kN/m^2$

2.3 Layer 3 - Silty Sandy Clay

$C_u = 81.15\ kN/m^2 \qquad N_c = 6.5$

$q_{nu} = 81.15x6.5 = 527.5\ kN/m^2$

$q_a = 527.5/3 = 178\ kN/m^2$

The bearing capacity results presented above give reasonable values especially the sandfill material. The values from the fill are based on the penetration test results, which may not be very reliable due to many factors. For instance, the sandy material compacts during the penetration testing and N-values are high under this condition. Besides, experience from the subsoil characteristics of the sandfilled areas have revealed that, the foundation failures are generally due to the consolidation of highly compressible clays accelerated by the developmental and environmental factors. Thus, the detailed settlement analysis is focused on the consolidation settlement of the weak clays underlying the distressed buildings.

5.2 Settlement Analysis

5.2.1 Settlement on the fill material (sand)

In sands and gravels the allowable bearing pressure is usually limited by settlement considerations rather than by ultimate bearing capacity. Since for most soil investigations it is impractical to obtain and test undisturbed samples of granular soil, the procedure used was to estimate the relative density from standard penetration tests or cone penetration tests. Settlement was estimated from SPT N-values using the chart given for estimating allowable bearing pressures on sands using standard penetration tests results (Terzaghi and Peck, 1967). This chart gives the allowable bearing pressure for 25 mm settlement. The values obtained from the chart are for dry conditions. In the reclaimed areas investigated, since the water table is above foundation base, the bearing pressure obtained from the chart are halved.

For the Borikiri sandfill area, using the soil data presented in the previous section, the average N-value of 13, gives allowable bearing pressure of about 100 kN/m^2 which is equivalent to 50 kN/m^2 considering the effect of the ground water level above the foundation level.

In the Ogolo-Aba, the allowable bearing pressure obtained from this method of analysis is about 70 kN/m^2 at the foundation depth of 1.0 m. This also gives a settlement of 25 mm. As mentioned earlier, design values obtained from penetration test results are considered as rough estimates of the actual design values and are only utilised for comparison and for preliminary design where necessary.

In addition, the detailed settlement analysis considered in this work, is the "Consolidation Settlement" due to the underlying organic peaty clays. This is based on the fact that the fill material is SAND, fine to medium coarse which is usually free draining material. Obviously, if the excessive settlement of the buildings were due to the fill material, the

settlement would have been completed soon after the construction period. However this is not the case of the distressed buildings of the sandfilled areas of the Niger delta. For instance, the two buildings under consideration were built more than seven years ago, but the settlement of the soil under the buildings has not ceased. This is confirmed from the continuous opening of the cracks on the residential building at Borikiri even after several repairs (filling of the cracks with concrete or similar materials).

5.2.2 Consolidation settlement

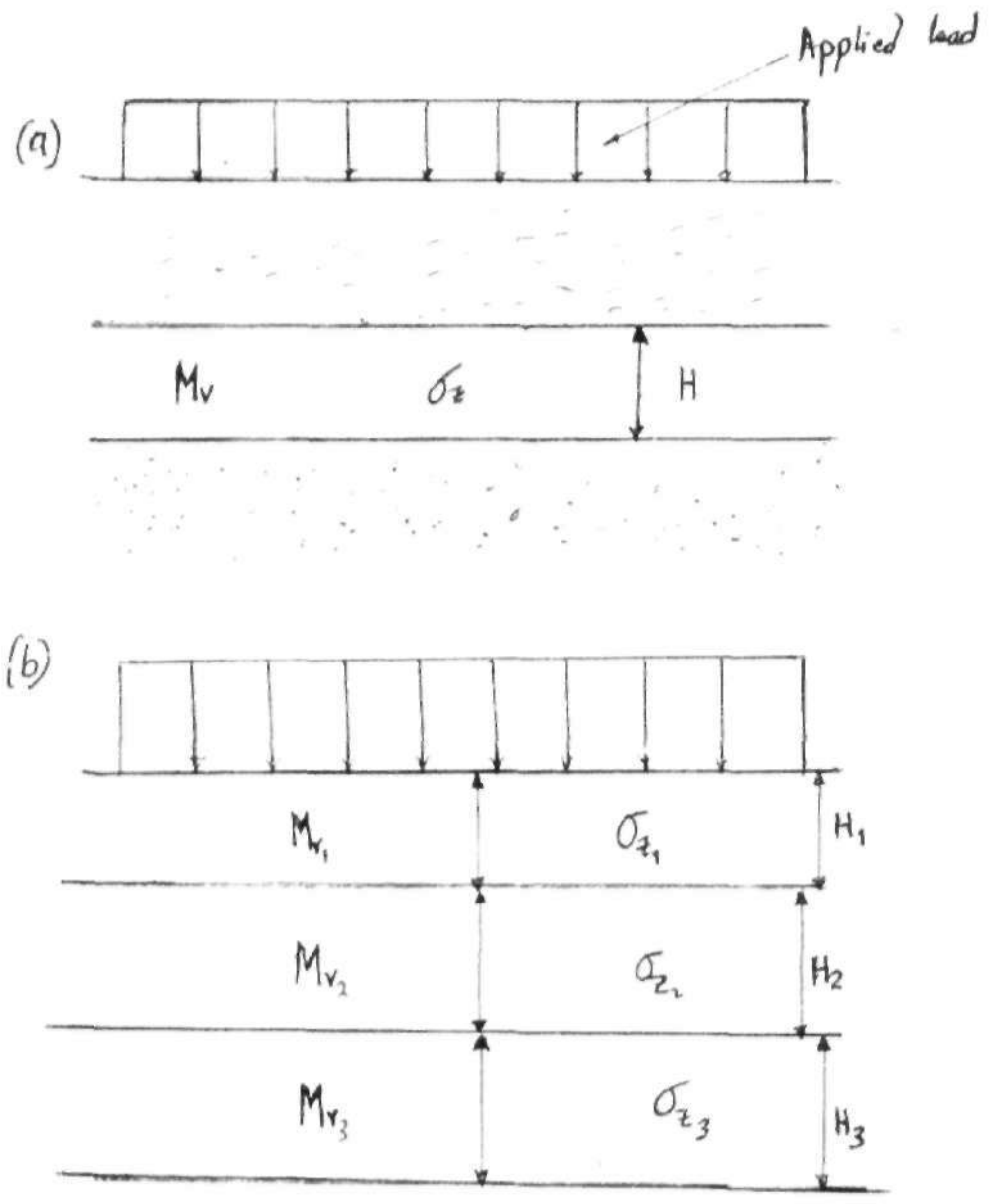
Consolidation settlement is often estimated using the Coefficient of Volume Compressibility (m_v). The m_v, defined as

$$m_v = 1/h * dh/dp \qquad 5.4$$

where h is the specimen thickness in a consolidation test and dh is the change in thickness due to a pressure change dp. This definition was used to calculate consolidation settlement due to the compressible layers of clays beneath the foundation, as illustrated in Figure 5.1 The specimen thickness h is replaced by the layer thickness H and the change in thickness dh becomes the settlement S. The stress increase dp is now equal to the vertical pressure in the layer, s_v, due to the applied loading. By substituting these terms and rearranging the equation gives:

$$S = H * M_v * s_v \qquad 5.5$$

The value of s_v is usually obtained from elastic analysis. The stress across the layer will obviously vary with depth but for this layers it is usually sufficient to calculate the value of s_v at mid-depth. Alternatively values of the top and bottom of the layer may be



5.1: Soil profile for estimating settlement of different layers of subsoils

calculated and the average value used. Where several compressive layers are present, as shown in Fig.5.1b, the value of s, are calculated for each of these and the settlement is obtained by adding together the settlement of the layers:

$$S = H_1 m_{v\sigma 1} + H_2 m_{v2} \sigma_{z2} + \qquad 5.6$$

However, for clarity, the consolidation settlement analysis described above was achieved by using two methods of consolidation settlement calculation:

- settlement calculation using consolidation theory, and
- settlement calculation using elasticity theory

The two methods are closely related, but the only difference is in the application of the compressibility parameters.

(i) Settlement Calculations using Consolidation Theory

In this method of analysis, settlement estimation for the distressed buildings were obtained by using the following formulae:

$$m_v = C_c/(1 + e) * \log(P_2/P_1)/(P_2-P_1) \qquad 5.7$$

From Equation 5.7 above, this implies that:

$S = H_{\sigma z} m_v = H*(p_2-p_1) * C_c/1 + e *\log(p_2/p_1)/1 + e (p_2-p_1)$

this gives

$$S = H* C_c \log(P_2/P_1)/1 + e \qquad 5.8$$

where H is the thickness of the compressive layer, C_c is the compression index, and p_2, p_1 are the two pressure ranges.

The compression indices Cc were obtained from e-log P curves of **Appendix E.** The e-log P curves were plotted using the appropriate values of consolidation pressure and void ratios from the consolidation test results. The ranges of settlement results obtained using the consolidation calculation theory is as summarized below:

Table 5.1 : Summary of Settlements Results for Obtained Consolidation Theory

1. LOCATION	DEPTH (m)	RANGE OF ESTIMATED SETTLEMENT (S=S1+S2+...) (mm)
2. Borikiri	2.25-9.45	202-458
3. Ogolo-Ama	1.5o-7.5	189-586

(ii) Settlement calculation using Elasticity Theory

The approach is to calculate displacement (settlement) directly using elasticity theory, thus reducing the two separate stages in the settlement calculation to one, and avoiding the need to calculate average values of consolidation pressure σ_v across soil layers. Numerous solutions, for both stresses and displacements, have been produced, many of which have been presented by Poulos and Davis (1974). The problem often encountered using elastic solutions to calculate settlements is that it requires the evaluation of Young's modulus, E and Poisson's ratio v neither of which are measured or are strictly meaningful for consolidation problems. This problem is partially solved by using the relationship between E and mv, given as:

$$M_v = 1/E * (1+n)(1-2n)/(1-n) \qquad 5.9$$

where n is the Poisson's ratio and E is defined as the "deformation modulus".

The values of E used for the evaluation of m_v from equation 4.8 is given as:

$$E = 0.578N + 7.17\,mPa \qquad 5.9$$

where N is the average SPT N-values for the layer of soil considered.

The values of the Poisson's ratio used were obtained from the equation:

$$v = (1\text{-}sin\phi)/(2\text{-}sin\phi) \qquad 5.10$$

The estimation of m_v values obtained from Equation 4.8 applies only to the sandfill material and alluvial sands, where it was impossible to obtain the undisturbed sample of the sand for consolidation test. For the clayey layers, under the foundations, the m_v values used were the values obtained from consolidation tests results.

The settlement calculation by elasticity theory was simplified in this study by the use of Influence Factors .The derivation of the Influence Factors formula can be appreciated by considering the settlement, *ds*, of a thin layer of soil, thickness d_z, at depth z beneath a foundation. This gives

$$ds = m_v * \sigma_z * dz \qquad 5.11$$

After integration of Equation 5.11, the settlement becomes:

$$S_f = m_v * B * P(I_2 - I_1) \qquad 5.12$$

where I_1 and I_2 are values of the influence factor at depth z_1 and z_2, respectively. Influence factors chart for both the Center and Corner of a rectangular foundation are presented by (Tomlinson, 1986; Carter and Symons, 1989 etc.), for the settlement design calculation in this work, the chart presented by Carter and Symons 1989 was adopted. Settlement was thus calculated beneath the centre and corner of the strip foundation for the compression layers of any thickness and depth, without the need to evaluate stresses.

Since the m_v values obtained in the previous chapter varies with depth, the soil profile under the foundation was divided into several layers and settlement obtained as the sum of the individual layers. Stress and displacement calculations are usually based on the assumption that the loaded area is completely flexible, resulting in settlements at the centre of the area being greater than those at the edges; this point is particularly highlighted by

the influence charts. Also, in many soils, calculated settlement values overestimate the amount of consolidation settlement. A correction is applied to give a more accurate estimate of the true consolidation settlement, thus.

$$S_{field} = \mu * S \qquad 5.13$$

where S_{field} is the estimated settlement taking into account the differing pore pressure response of the soil in the field to that in the oedometer test. S is the settlement estimate obtained by the simple application of consolidation theory, and m is the factor which depends on the soil type and thickness of the compressible layer compacted with the foundations width. Value of consolidation factor μ for various soil types are presented in many soil mechanics and geotechnical engineering textbooks (Das, 1989; Carter and Bentley, 1991 etc).

In addition, experience has shown that, in normal foundation practice, rectangular foundations are assumed to influence the soil to a depth of 1.5 multiplied by foundation width. Whilst the majority of settlement does take place within this depth, experience has proved that some measurable compression of the soil down to depths greater than 1.5 foundation width can take place. Also, in terms of stress analysis, the depth of soil influenced by a foundation load is greater than 1.5 foundation widths below the base of the foundation (Carter and Bentley, 1991). Although for normal foundation soil, allowing for the fact that soil also generally become stiffer with depth, most of the compression can usually be expected to take place within a depth equal to about three footing width below the base of a foundation. But in the case of the reclaimed areas under consideration, where the soils below the foundations becomes weaker with depth, due to the high moisture content and its variations, as well as the decaying process of the organic materials, thus, the depth of influence of the foundations load will likely increase. For this work, the depth of influence of the foundation load was assumed to be greater than the usual 1.5 foundation depth based on the poor subsoils encountered in the sandfilled areas investigated.

The design calculations made to estimate the settlement of the distressed buildings are as illustrated and presented in **Appendices** F_1 and F_2 respectively, for the two investigated sites.

5.3 Tolerable Differential Settlement

Differential settlement is usually assumed to be equal to half of the total settlement; this assumption was utilised to estimate the differential settlement of the distressed buildings.

In evaluating the tolerable differential settlement, the limiting factor is usually the amount of angular distortion that a structure, or any installed equipment, can accommodate without distress or damage. It can be expressed as the angle of the settlement profile, as illustrated in Figure 5.2:

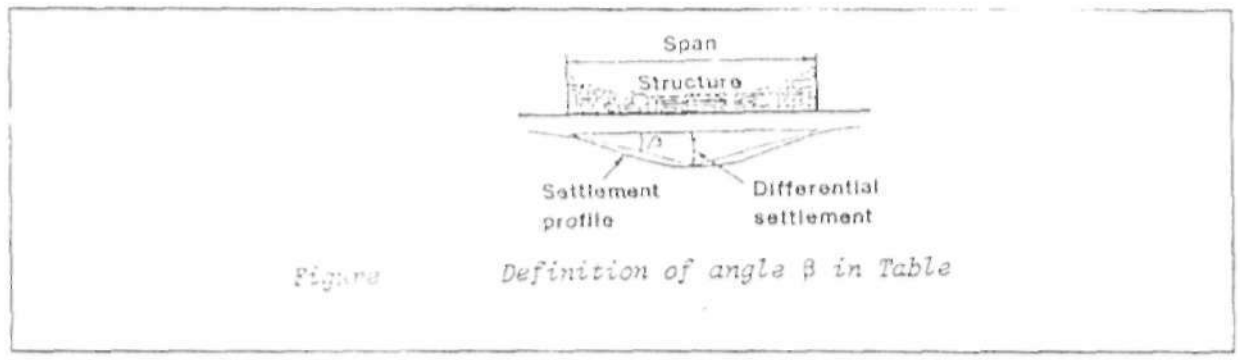


Figure 5.2: Definition of Angle β for Calculation of Differential Settlement (Carter and Symons, 1989).

Differential settlement is thus given by:

$$\Delta y = ? \, span \tan \beta = ? \, span \, \beta \qquad 5.14$$

(since for small angles tanb = b where b is in radians)

Typical values of tolerable differential settlement for different structures and foundation soils have been published by many investigators (Tomlison, 1986; Carter and Symons, 1989; Al-Kafaji and Andersland, 1989 etc.), for one or two-storey houses or similar lightly loaded buildings with brick or block load bearing walls the values of β varies between 0.002-0.003. Larger values are tolerable if most settlement has taken place before finishes are completed. For this research work, the value of 0.0025 was assumed for b, based on the values presented by Carter and Symons (1990). This value of b was used throughout to estimate the range of values of the tolerable differential settlement of the distressed buildings.

5.4 Rate Of Consolidation Settlement

The rate of settlement of a saturated soil is expressed by the coefficient of consolidation, C_v. Theoretically, consolidation takes a long time to be completed and it is usual to calculate the time taken for a given degree of consolidation U to occur, where U is defined by:

$$U = \frac{\text{Consolidation settlement after a given time, t}}{\text{Final consolidation settlement}}$$

The rate at which settlement occur is usually defined by the formula:

$$t = \frac{T_v d^2}{C_v} \qquad 5.15$$

where:

t = the time for a required degree of settlement

T_v = basic time factor

d = maximum length of the drainage path

C_v = coefficient of consolidation

The values of T_v depend on the degree of consolidation, the drainage conditions and the pressure distribution. Settlement is usually considered to be substantially complete when the

degree of consolidation is 90% (U=0.9). To achieve 100% of final settlement would theoretically take an infinite time.

In order to calculate the rate at which the settlement of the distressed building may occur, the calculation was based on the Equation 5.15 above. The average values of C_v used, were the values at the pressure corresponding to foundation pressure plus the overburden pressure at the depth considered. The calculations made to estimate the rate of consolidation of the distressed buildings are presented in **Appendix G.**

However, for complicated soil conditions, such as the subsoils of the sandfilled areas, where there are sand or silty layers, lumps of decaying vegetation which may or may not form continuous drainage paths and where lateral drainage may be significant, the values of d assumed may not be very reliable. Since the time t is proportional to d_2, there is little hope of predicting the rate of consolidation settlement of these reclaimed areas with any reasonable accuracy. Nevertheless, this method of analysis has been used successfully to estimate the rate of consolidation settlement in the majority of the sandfilled areas investigated.

Chapter 6

DISCUSSION OF RESULTS

6.1 Engineering Properties of the Soils

The soil investigation in the vicinity of the distressed buildings of the sandfilled areas revealed that, within the depths explored for this research work, the project sites are characterised by sandfill overlying organic clays and sands. Classification, strength and consolidation characteristics of the soils were obtained from laboratory and insitu tests and their results are presented in the relevant chapters. For clarity, the geotechnical properties of the subsoils are summarised below, and the influence of these properties on the failure characteristics of the lightly loaded buildings of the reclaimed areas of the Niger delta are also discussed in the relevant sections.

(i) Sandfill

In the two sites investigated, the sandfill consists of grey, brown, medium to coarse-grained and loose to medium dense sands. As presented in the classification test results, the sandfill material can also be described as slightly gravelly SAND (< 5% gravel) to gravelly SAND (5-20% gravel). The average thickness of the sandfill in Borikiri is about 1.7 m while in Ogolo-Ama the average sandfill thickness is about 1.65 m. There are lumps of organic clays found within the sandfill in all the sites investigated. From the analysis of the classification test results, the range of variation in the effective and mean particle sizes, coefficient of uniformity and curvature of the sandfill particles and SPT penetration resistance N-values for the sandfill are summarised below:

BORIKIRI SANDFILL	*Min*	*Max*	*Average*
Effective particle size d_{10} (mm)	0.23	0.28	0.27
Mean particle size d_{50} (mm)	0.43	0.76	0.59
Coefficient of uniformity $Cu = d_{60}/d_{10}$	2.0	3.4	2.54
Coefficient of curvature $Cc = d^2_{30}/d_{10}*d_{60}$	0.9	1.1	1.0
SPT penetration resistance N-values (blows/0.3m)	14	23	19

Average angle of shearing resistance of 31E and saturated unit weight of 20 kN/m^3 are obtained for this sandfill layer.

OGOLO-AMA SANDFILL	*Min*	*Max.*	*Average*
Effective particle size d_{10} (mm)	0.21	0.32	0.27
Mean particle size d_{50} (mm)	0.53	0.72	0.63
Coefficient of uniformity $Cu = d_{60}/d_{10}$	2.5	3.0	2.75
Coefficient of curvature $Cc = d^2_{30}/d_{10}*d_{60}$	1.0	1.3	1.2
SPT penetration resistance N-values (blows/0.3m)	12	18	14

Average angle of shearing resistance of 32° and saturated unit weight of 22kN/m^3 are obtained for this sandfill layer.

(ii) <u>Soft organic clays</u>

These are the natural material over which the sandfill is placed in the two investigated sites. The organic clay (about 2.0m below the fill) is dark grey in colour, extremely weak, very soft to soft in consistency (Cu < 40kN/m^2) and has high to very compressibility (m_v > 1.5m^2/MN), and contains abundant swamp decaying vegetation. The lower organic clay is bluish grey in colour, very soft, saturated and highly compressible, very silty and becoming sandy with depth. The light bluish grey, silty clay contain little or no decomposed mangrove vegetation. In Ogolo-Ama, the lower silty clay is very sandy.

The range of variations in the values of the index and engineering properties of the weak and compressible clays are summarized below:

BORIKIRI- Dark grey, very soft organic peaty clay:

	Min	Max	Average
Natural moisture content (%)	166	267	146
Liquid limit (%)	170	199	166
Plastic limit (%)	46	68	60
Plasticity index (%)	109	222	140
Saturated unit weight (kN/m^3)	12.2	13.0	12.80
Dry unit weight (kN/m^3)	5.1	9.27	7.30
Specific gravity	2.33	2.77	2.49
Initial void ratio	1.528	3.695	3.088
Undrained cohesion (kPa)	5.0	23.6	13.00

BORIKIRI- Light bluish grey, very soft,very soft, organic peaty clay:

	Min.	*Max.*	*Average*
Natural moisture content (%)	30	70	34.7
Liquid limit(%)	24	70	35
Plastic limit (%)	9	16	11
Plasticity index (%)	19	54	26.6
Saturated unit weight (kN/m^3)	17.20	21.0	18.2
Dry unit weight (kN/m^3)	16.3	17.6	16.95
Specific gravity	2.58	2.74	2.66
Initial void ratio	0.439	0.64	0.542
Undrained cohesion (kpa)	13	24	18

OGOLO-AMA- Dark grey, very soft, organic silty clay:

	Min.	*Max.*	*Average*
Natural moisture content (%)	46	170	97.3
Liquid limit (%)	44	173	110
Plastic limit (%)	17	61	35
Plasticity index (%)	27	123	75.3
Saturated unit weight (kN/m^3)	13.02	14.4	13.7
Dry unit weight (kN/m^3)	6.0	9.27	7.3
Specific gravity	2.27	2.99	2.64
Initial void ratio	3.283	3.566	3.280
Undrained cohesion (kPa)	6.0	22	13

OGOLO-AMA- Light bluish grey, very soft, very silty clay:

	Min.	*Max.*	*Average*
Natural moisture content (%)	40	120	64
Liquid limit (%)	24	67	39.2
Plastic limit (%)	9	19	13
Plasticity index (%)	19	51	29.2
Saturated unit weight (kN/m^3)	19.6	20.1	20.28
Dry unit weight (kN/m^3)	16.71	17.1	16.9
Specific gravity	2.48	2.71	2.60
Initial void ratio	0.267	0.645	0.456
Undrained cohesion (kPa)	13	24	19

(iii) Medium dense alluvial sand

In the two sites investigated, this natural sand layer is found below the bluish grey, very soft, silty clay. The sand is grey to whitish grey in colour, medium to coarse-grained and predominantly medium dense in its state of compaction. The alluvial sand layer contain high percentage of fines (Silt and Clay >20%) and are gravelly with depth. The fines content decrease with depth and the soil layers become more sandy and gravelly. This sand layer is often encountered at greater depth, thus only when deep foundation is considered that the sand layer can be used as a foundation soil. As mentioned earlier, deep foundation is outside the scope of this work, and the boring was carried out to this natural sand layer to avoid termination of the borehole in the highly compressible weak clays.

(iv) Ground water conditions

Observations of the ground water levels during the investigation of the two sites revealed the following:

(a) The ground water was, in general, encountered just below the ground surface at depths varying from 0.4 to 0.8 m for Borikiri and from 0.6 to 1.0 m for Ogolo-Ama, below the ground level.

(b) The water levels responded rapidly to tidal changes, thus inferring that the deposits including the organic clays were comparatively permeable.

(c) Geotechnically, the ground water conditions existing at the site during the investigation were substantially unfavourable for foundation works.

(v) Strength and Compressibility Results

The penetration tests and the laboratory results have confirmed that sandfill are placed on very weak organic clay. The undrained shear strength (Table 3.1a and 3.1b) revealed that the Cu values for the organic peaty clay in the two sites investigated are less than 40 kN/m^2.

This classified the organic clay as very soft (< 20 kN/m^2) and soft (20-40 kN/m^2) in consistency (Smith, 1990; Carter and Symens 1990). In addition, the coefficient of volume compressibility (m_v) results (**Appendix D1 and D2**), for pressure ranges of 25-50 kN/m^2 and 50 - 100 kN/m^2; the range of m_v values for the organic peaty clay are between 1.21 - 2.73 m^2/MN. Cater and Symons (1989) described clay of m_v values between 0.3 - 1.5 m^2/MN and >1.5m^2/MN as clay of high compressibility and very high compressibility respectively. The authors added that the range of these C_v values are typical of deltaic alluvial and sensitive clays. The consolidations characteristics of the organic clays are as summarized in Table 6.1.

Based on the findings in this study, the organic clays beneath the foundation of the distressed buildings can be described as follows:-

(a) Weak organic peaty clay, very soft to soft inconsistency.

(b) Organic peaty clays of high to very high compressibility.

(c) The organic clays are normally consolidated clays typical of estuarine and delta deposits, and sensitive clays. They are highly organic alluvial and peaty clays.

Table 6.1: Consolidation characteristic of the weak clays underlying the distressed buildings

Depth (m)	Bulk unit weight kN/m^2	C_v m^2/yr		M_v m^2/MN		k(m/s)	
		Pressure ranges kN/m^2		Pressure ranges kN/m^2		Pressure ranges kN/m^2.	
		25 - 50	50 - 100	25 - 50	50 - 100	25 -50	50 - 100
BORIKIRI							
1.50 - 5.70	13.6	0.315	0.310	1.315	1.315	$1.03E^{-10}$.	$1.063E^{-10}$.
5.70-1050	20.7	2.14	1.665	0.273	0.273	$1.35E^{-10}$.	$3.605E^{-10}$.
Ogolo-Ama							
1.50-3.75	15.0	6.385	3.543	3.006	1.569	$2.30E^{-10}$.	$3.45E^{-10}$.
3.75-6.0	21.5	7.05	9.295	0.225	0.196	$4.96E^{-9}$.	$1.95E^{-9}$

(vi) **Bearing capacity results**

In Borikiri sandfill, the average values of the allowable bearing pressure is about 200 kN/m^2, while that of the fill material in Ogolo-Ama is about 100 kN/m^2. However, the allowable bearing pressure of the sandfill material appear reasonably high considering the high ground water levels and shallow thickness of the fill material in the two sites. This implies that the embedment depth of the two buildings is below the water table and substantial part of the foundation load is carried by the underlying organic peaty clays which are of very low bearing strength.

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The organic peaty clays immediately underlying the sandfill are extremely weak and highly compressible. These clays have poor bearing strength and cannot support significant foundation loads without excessive total and differential settlement including possible shear failure. The average allowable bearing pressure of about 36 kN/m^2 using the factor of safety 3 was obtained for the Borikiri sandfill for this clay layer. Also, for Ogolo-Ama, the allowable bearing pressure is about 35 kN/m^2. From the unconsolidated undrained triaxial compression results, the average value of the undrained cohesion is between 6 kPa and 23.6 kPa for the Borikiri sandfill. In Ogolo-Ama, the undrained shear strength values varies between 5.8 to 8.0 kPa. These values, are rather too small to support the estimated foundation load of 36 kN/m^2 and 40 kN/m^2 of the two distressed buildings.

(vi) **Settlements Results**

Many empirical methods were adopted at the initial stage to evaluate the settlement of the distressed buildings. For design purposes, assessment of the settlement was carried out using the conventional relationship containing the coefficient of volume compressibility m_v:

$$S = m_v \times B \times \Delta p \times I \qquad 6.13$$

where S = Settlement

B = foundation width

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ΔP = foundation load

I = influence factor

For the Borikiri sandfill, settlement of the order of 60 to 200 mm were obtained for a strip footing 0.7m wide, 12m long and estimated foundation load of 36 kN/m^2. In Ogolo-Ama sandfill, settlement of the order of 80 to 300 mm were obtained for a strip footing of 1.0m wide, 6m long and estimated foundation load of 40 kN/m^2. However, due to the variation in the thickness of the sandfill material and the organic peaty clays, significant differential settlements should be expected.

In addition, the rate of settlement of the distressed buildings evaluated from the relationship:

$$t = \frac{T_v d^2}{C_v} \qquad 5.14$$

showed that in the Borikiri sandfill, the rate of settlement of the organic clays is in the order of 7 to 28years for 50% and 90% settlement respectively. The rate of settlement obtained for the distressed building in Ogolo-Ama is in the order of 1 yr to 3 yrs for the 50% and 90% settlement, respectively. The rate of settlement results obtained for the Ogolo-Ama sandfilled area confirmed the existing condition of the distressed classroom block. The settlement of the distressed classroom black appears to have stabilized over three years ago as confirmed by the community surveyors, although the cracked areas of the building are still awaiting repairs from the school authority. However, for clarity, the settlement results obtained from the two distressed building are as shown in Table 6.2.

Table 6.2: Summary of Predicted Settlements for Borikiri and Ogolo-Ama

LOCATION: BORIKIRI						
Thickness of weak clays	Average C_v values m^2/yr	Predicted primary settlement (mm)	Predicted Differential settlement (mm)	Range of Tolerable settlement (mm)	Time for 90% settlement (yr)	Ground water level (m)
7.0 - 9.5	0.32	60 - 200	30 - 100	12 - 18	6*	0.4-0.7
LOCATION: OGOLO-AMA						
3.0 - 4.5	2.023	80 - 300	40 - 150	25 - 35	3*	0.6-1.0
Assumption: Different settlement = (Predicted totaf settlement)						

6.2 Solutions to Remedy the Foundation Problems

In the course of this study , it was observed that the subsoils existing within the vicinity of the distressed buildings in Borikiri sandfill and Ogolo-Ama sandfill in the Niger delta have three distinct layers: the sandfill material, the highly compressible organic clays and the loose to medium dense alluvial sand which is in agreement with (George and Abam, 1990, Trevi, 1993 etc.). Experimental results have proved that the excessive settlement of the lightly loaded buildings of the reclaimed areas is dependent on the consolidation properties of the underlying weak organic clays and the adverse effect of the environmental and developmental factors peculiar to the sandfilled areas that is the major source of problems in these sites.

Besides, the settlement results (Table 6.2), show the ranges of excessive total and differential settlement of the distressed buildings. These findings confirmed the causes of the deep cracks on the structures found on the sandfilled areas (plate i-x). During the reconnaissance survey of the reclaimed areas, the crack width measured were between 0.5 to 2cm wide. Based on the provision of the code of practice (BS8110), the serviceability requirements of the two buildings are not fulfilled. Thus, the distressed buildings have failed under serviceability limit state.

In proposing a solution, a comprehensive study was necessary to determine the possible means of solving the problems before making recommendations as to the solution that is both practicable and cost effective. Based on the findings presented above, the following solutions are considered:

(1) Consideration of environmental and developmental factors

(2) Improvement of the reclaimed land properties

(3) Design of effective foundation types.

6.2.1 Consideration of developmental and environmental factors

In order to provide adequate solutions to the foundation problems in the sandfilled areas of the Niger delta, the environmental and developmental factors should be considered as an integral part of the programme. Environmental and developmental factors, such as the high ground water level and its variations, the high organic content of the weak clays and the consolidation characteristics of weak clays etc. are basically due to the geological location of the reclaimed areas. Hence the adverse effect of these geological factors on subsoil properties of the sandfilled areas cannot be avoided, rather they should be provided for both in the reclamation stage and in the development of the sandfilled areas. With respect to the experimental results, the effect of the environmental and developmental factors highlighted in the subsequent paragraphs are considered significant both in the analysis and provision of solutions to the causes of foundation failures of structures found on the sandfilled areas of the Niger delta.

6.2.1.1 Reclamation techniques

The problems associated with reclamation techniques are presented in section 2.41 of this work. Factors like the insufficient sandfill thickness, high ground water levels in the sandfill, high content of decaying mangrove vegetation in the weak clays e.t.c. Are as a result of lapses in the reclamation process. For future reclamation by hydraulic sandfilling in the mangrove swamp environment of the Niger delta, if the reclamation is properly supervised by the relevant authorities. Some of the foundation problems in the area will be controlled.

For instance, before sandfilling of the areas, the thick mangrove vegetation were to be cut down and the fallen vegetation cleaned off the areas. This procedure was not followed in the sandfilled areas investigated. In Borikiri for example, the decaying mangrove vegetation

formed more than 50% of the organic peaty clay immediately underlying the sandfill. The decaying process of these vegetation contributes to the excessive settlement of the distressed building of the sandfilled areas and also have adverse effect on the engineering properties of the subsoils of the reclaimed areas.

6.2.1.2 Effect of the sandfill thickness

Hydraulic fill is supposed to raise the land surface well above the ground water level, provide a suitable foundation base capable of spreading the foundation load without overstressing the underlying weak soils. Insufficient sandfill thickness not only hinders the achievement of these basic objectives, but gives room for water to flood the sandfill material. The presence of excess water in the sand reduces the stiffness of sand and also reduces the density and hence the bearing capacity of the sand (Ramasamy, 1986; Peck *et al.*, 1985). The end result is excessive foundation settlement of buildings found on the fill material. Besides, due to the geological background of this region, the ground water levels respond rapidly to the tidal variation, this will eventually make the pore pressure in the soil excessively variable and may contribute immensely to stability problems. Preloading of the underlying weak soils is one of the main target of hydraulic fill. When the sandfill thickness is too shallow, consolidation pressure imposed on the weak soils will not be effective enough to expel the pore water. Hence the natural soil still remains waterlogged and highly compressible.

6.2.1.3 Organic content of underlying weak soils

The presence of excessive organic matter in any soil affects settlement and strength characteristics of the soil (Odell *et al*, 1960; Bray, 1979, Al-Khafaji and Andersland, 1989). With respect to findings reviewed above, in most sandfill areas the quantity of decomposing plant debris in the weak clays is revealed to be substantial ($> 30\%$), especially at certain

depth just below the sandfill material, they can form more than 50% of the weak clay soils. Decomposition of excess organic material has adverse effect on the geotechnical properties of the soil. The decaying process may induce excess foundation settlement of the soil. Also, with the substantial amount of organic matter in the weak and saturated soils of the sandfilled areas, the soils will exhibit high compressibility and poor strength due to the continuous decomposition of the organic matter. In most cases, the affected areas may assume the characteristics of a collapsible soil.

6.2.1.4 Consolidation characteristics of the organic clays

Experimental results have proved that the organic peaty clay underlying the sandfilled areas are very soft to soft in consistency, high to very high in compressibility, sensitive and saturated. The transfer of foundation load to such weak clay layers, induces excessive consolidation of the clays which inevitably results in unacceptable total and differential settlement. The amount of settlement caused by the consolidation of the weak organic clays are increased by the decomposition of the organic materials.

Nevertheless, the consolidation settlements can be controlled, if foundations are designed to avoid the transfer of substantial foundation load to the organic peaty clay layers. Besides, where the thickness of the organic clay is less 3.0 m, and the structure can justify the cost of expensive foundation, this clay layer should be excavated out and replace with compacted cohesionless material.

6.2.1.5 Time dependant properties of sandfilled areas

Reclamation by hydraulic sandfill method is a time dependant programme which can be assessed in terms of the age of the sandfill material. If sufficient time is not allowed for an adequate densification of the sandfill material and the consolidation of the organic clays, buildings found on a fresh fill will be subjected to excessive settlement initiated by

expulsion of the void water and primary consolidation of the weak soils below it. This may explained the excessive settlement of the residential building at Borikiri which is believed to be the first structure constructed in the area soon after the reclamation.

6.2.1.6. Inappropriate use of the reclaimed areas

In the reclaimed areas investigated, the sandfill were openly removed from the sites both by the developers and by the riparian owners. The riparian were often found excavating and loading their cances with the fill material for commercial purpose and for private use. Majority of the developers, removed the fill from nearby plots and were using it commonly on the sites for moulding blocks. Also, in some parts of the sandfilled areas where the fill material was above 2.0m thick, the developers were seen excavating very depth >1.5m to place the foundation. According to the site workers, the reason for this great depth is based on their working experience with subsoils; 'the deeper you go, the higher the possibility of encountering better foundation soils'.

Obviously, these unfortunate situations are mainly as a result of poverty, ignorance and lack of security for the reclaimed areas. These problems can be minimized by public enlightenment on the use of the fill material and by provision of adequate security from the relevant authority immediately after the reclamation.

6.2.2 Improvement of the reclaimed land properties

With regards to the subsoil conditions prevalent in the sandfilled areas of the Niger delta, the need for some soil improvement becomes a necessity. The soil improvement method considered feasible for this work is basically to improve the engineering properties of the fill material and to preclude the adverse effect of the developmental and environmental factors. Usually, for any reclaimed land, majority of the soil improvement methods available are highly technical and expensive especially for the lightly loaded buildings under

consideration. However for the purpose of this research work the following soil improvement methods are put forward:

(i) **INCREASE OF SANDFILL THICKNESS:** From the basic objective of hydraulic sandfilling, the sandfill material is designed to raise the land surface well above the ground water level or underlying weak material, to provide a dry foundation base capable of spreading the foundation load without overstressing the underlying weak soils. To achieve this objective in the sandfilled areas under consideration, the sandfill thickness should be raised at least 3.0 m above the natural soil level which is the maximum expected flood level of the reclaimed areas of the Niger delta. The presence of excess water in the sandfill material reduces the stiffness of the sandfill and also reduces the density and hence the bearing capacity of the sandfill material. In addition, due to the permeability of the underlying deltaic clays and the sandfill, the ground water levels respond rapidly to the tidal variation, which eventually makes the pore pressure in the foundation soil excessively variable and may contribute immensely to stability problems of the structure found on the reclaimed land. This stability problem will also be overcome if the sandfill thickness is raised to a reasonable thickness.

(ii) **PRELOADING OR SURCHARGING:** Prior to the development of the reclaimed areas especially where the soil investigation report reveals the sandfill thickness to be insufficient for any foundation load, preloading of the sandfilled area will help to improve the engineering properties of the subsoils. For instance, with shallow thickness of the sandfill confirmed in this study, consolidation pressure imposed on the weak clays may not be effective enough to expel the pore water. Hence the natural soil still remains waterlogged and highly compressible. Consolidation of the highly compressible organic clays can be accelerated by placing surcharge loading in the form of extra sandfill over an area that is to be developed. This can be removed just prior to construction, but the remaining sandfill

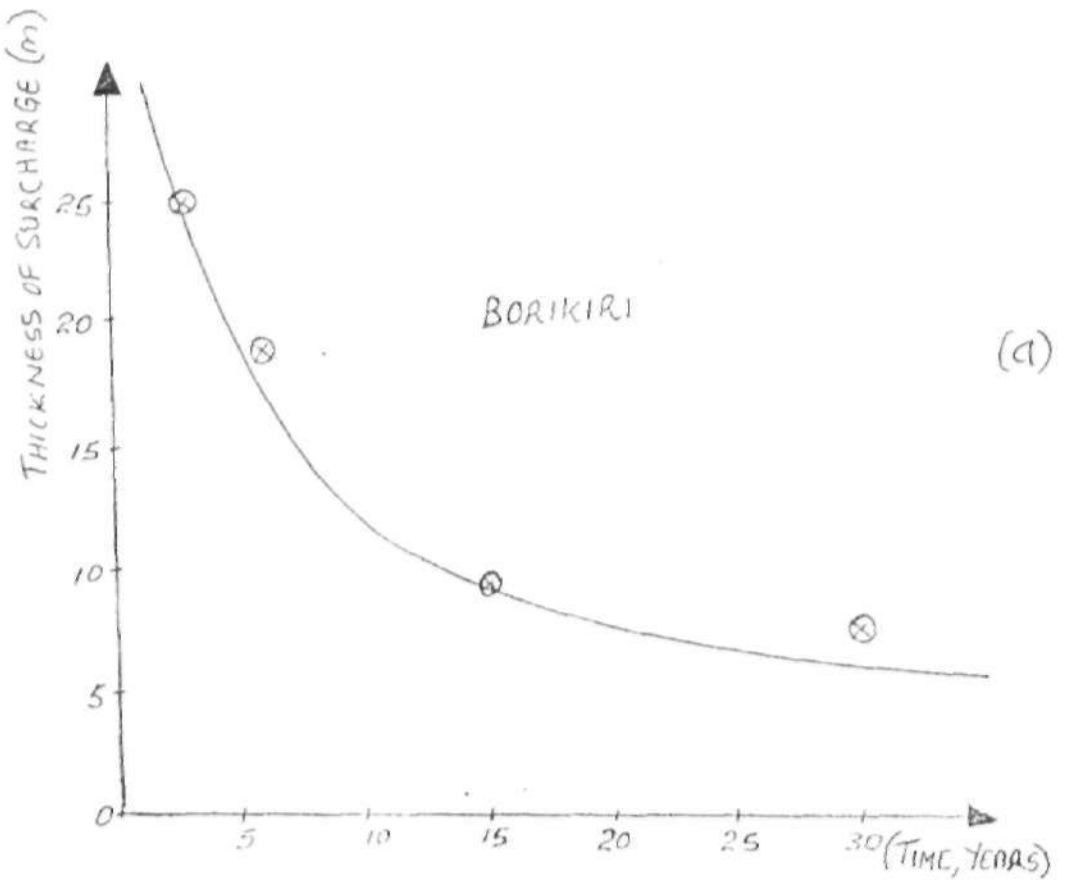
thickness should be at least 5.0m above the compressible material to ensure that the foundation base of the proposed structure is well above the maximum flood level and to avoid overstressing of the compressible material.

The effectiveness of this method, depends on the thickness of material placed, the length of time it is left in place, and the thickness and permeability of the underlying compressible layer. Several metres of fill may need to be left over an area for several months. To enhance adequate application of the preloading method of soil improvement, a detailed design calculations for the computation of the preloading fill thickness is presented in **Appendix H**. Based on the design calculations, the summary of the thickness of the surcharge required for 100% completion of settlement at various times, using the soil results obtained in this study are given in Table 6.3. Graphical presentation are shown in figure 6.1a and figure 6.1b.

Table 6.3: Summary of the thickness of the surcharge required for 100% completion of settlement at various times.

Location: Borikiri		Location: Ogolo - Ama	
Time(years)	Thickness of Surcharge(m)	Time(years)	Thickness of Surcharge(m)
3	25	0.25	16
6	19	0.50	9
10	12	1	7
15	9.5	1.5	5
Note: Consolidation Settlement = 117mm Time for 100% settlement = 66years		Note : Consolidation Settlement = 217mm Time for 100% settlement = 4years	

(iii) SOIL REPLACEMENT: An alternative solution often adopted in the Niger delta for big building projects where the cost of soil improvement can be compensated, e.g. industrial buildings, is the replacement of the weak soil with granular material. In this method, the fill material is completely removed and piled apart, then the organic peaty clay is excavated out and replaced with compacted granular fill the sandfill material is then



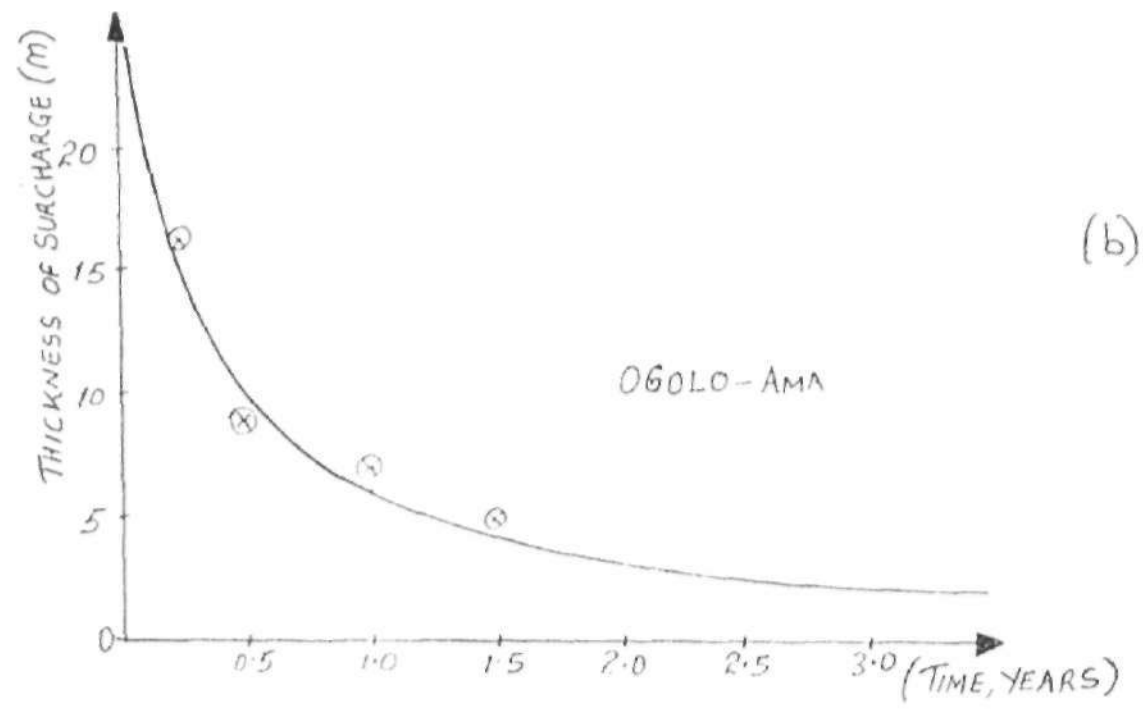


Figure 6.1(a&b): Graphs Showing the Surcharge Thickness Against Time for Borikiri and Ogolo-Ama

placed back and compacted. Although this may prove less satisfactory where the compressible soil extend to some reasonable depth, it is one of the most successful method of soil improvement used in carrying substantial foundation load on a shallow foundation in the reclaimed areas of the Niger delta.

(iv) **COMPACTION:** Although the fill material of this nature compacts naturally, further compaction is often essential to improve the densities of the sandfill and strength of the underlying weak soils before the placement of any structural load. Poorly compacted sandfill may have open structures, thus giving the reclaimed areas properties of collapsible soils.

In addition to the mentioned solution for the industrial buildings where the cost of foundation is balanced with total cost of the building and its services, vibro-compaction (or vibro-flotation) can be adopted. In this method, a large vibrating "poker is inserted into the ground. The vibro-compaction method may also be used in conjunction with the "stone-column" technique, in which stones are poured into the hole created by the vibrating poker. Although this method of soil improvement is highly technical and expensive, it is very useful for soil improvement where heavy load is to be carried on shallow foundations and the option of pile foundation is impossible may be due to granular nature of the underlying soils.

(v) **DRAINAGE:** Vertical drains can be used beneath fill or heavy foundations or in conjunction with surcharging to speed up the rate of consolidation. A special machine pushes the drain a plastic former wrapped in filter fabric into the ground. Alternatively, vertical sand drains may be used.

Frequently from the strength and consolidation test results obtained for the sandfilled areas, the underlying clays are very weak and highly compressible. In addition, due to

environmental and developmental condition prevalent in the reclaimed areas, the clays are saturated and contain substantial amount of organic decomposing plant vegetation. Considering these poor geotechnical properties, especially the low strength, the rational solution has always been to avoid the use of this soil layer as a foundation soil or to preclude the transfer of a substantial amount of the foundation load to the weak clay layers. The problem is particularly intractable where this soft deposits are of great thickness like in some sandfilled areas investigated. High settlement have to be expected and designed for in order to ensure a safe service life of the structure.

However, other measures of dealing with the poor subsoil condition of these reclaimed areas are based on the modification of the foundation design mainly to reduce the structural load; the various approach are as discussed in the next section.

6.2.3 Design of an effective foundation type

An effective design approach which considers the limitation of a reclaimed land, the high compressibility of the underlying organic clay, foundation cost, developmental and environmental factors such as the high ground water level and high organic content of the weak clays can be arrived at by utilizing an adequate design procedure. Majority of the conventional foundation types, (strip, raft and piles) are in common use throughout the sandfilled areas of the Niger delta. Due to variations both in the thickness of the fill and the underlying organic peaty clays, differential settlements are expected (Lafe, 1986), hence the use of pad foundations is not rational and should be avoided.

The choice of a particular foundation type depends on the subsurface conditions encountered on a site and the cost effectiveness of such building project. Generally, the shallow thickness of the sandfill and the weak strength of the underlying highly compressible clays pose the most critical foundation problem in those areas. Hence, most efforts at foun lation

improvement are concentrated on increasing the thickness of the fill material so that the imposed structural loads can be carried by the sandfill layer without overstressing the underlying weak clays. Most of the solutions adopted are in the form of special modification to the conventional foundation types and designed to suit the poor site conditions. Some of these modification are discussed below:

(a) **RAFT FOUNDATIONS:** The conventional raft foundations are usually excessively large and require a large volume of concrete and reinforcing steel. This can give rise to stability problems in the form of excessive total and differential settlements particularly in an area like this. The large volume of materials also leads to very high costs which may adversely affect the economic feasibility of the building project. To reduce the weight of the raft foundation to cope with poor soil condition and minimize settlements and saving on the concrete and steel, the solution is to replace the slab with shell, mainly cylindrical shells. These are referred to as shell raft foundations, an example of which is shown in Figure 6.2a: Other shapes of the shells can be used, such as hollow box (Fig. 6.2b & Fig. 6.2c).

(b) USE OF BUOYANCY RAFT OR BASEMENT: If the weight of soil removed is about the same as the weight of the structure and its foundation, there will be a little or no change in overall pressures within the soil so that settlement will be much reduced or, in theory, eliminated altogether. However, the cost of providing such foundation is high but against this must be balanced by the extra amenity value of the space created, the rigidity which the foundation provides, and the fact that this may be the only method of reducing settlement to tolerable value where deep deposits of soft clay are encountered.

(C) SPREAD FOOTINGS: the loose consistency of the sandfill and the poor bearing capacity of the underlying highly compressible clays call for exceptionally large

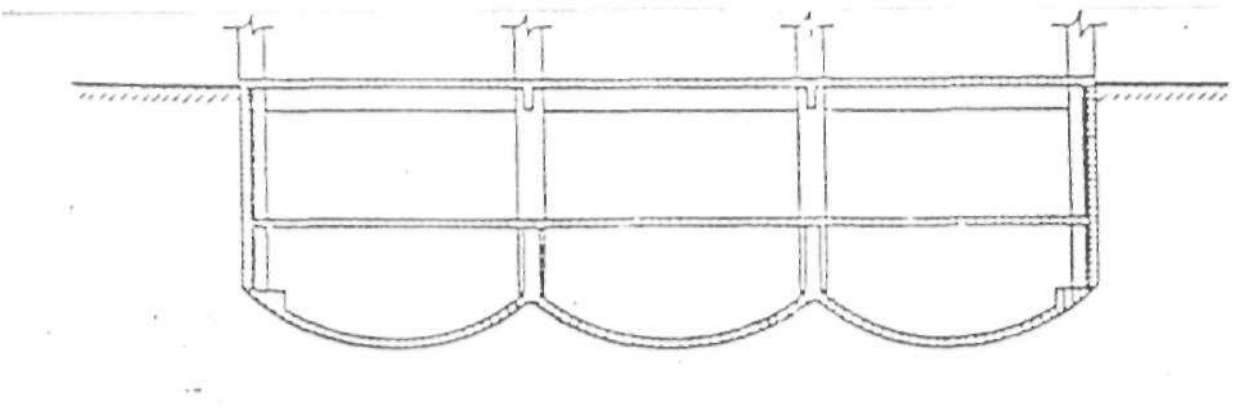


Figure (6.2a): Compensated foundation having cylindrical shells

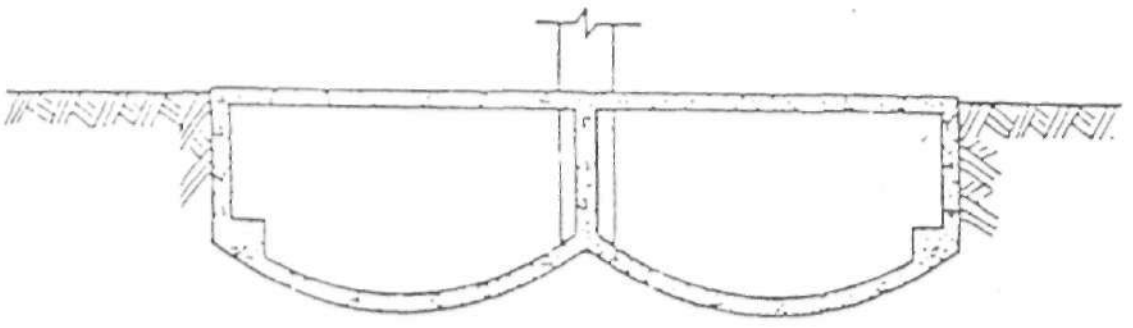


Figure (6.2b): Hollow footing with cylindrical-shells bottom

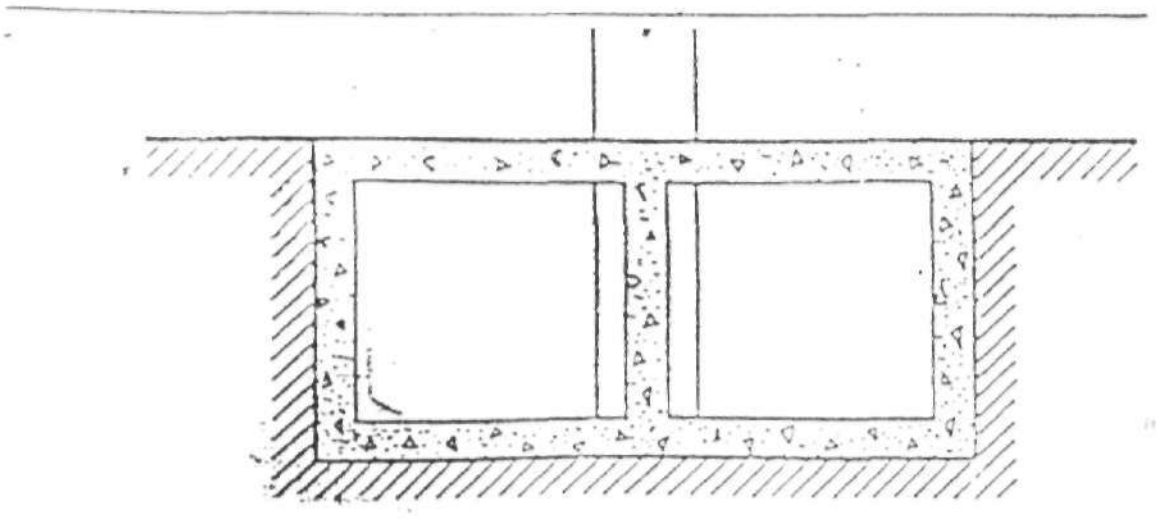


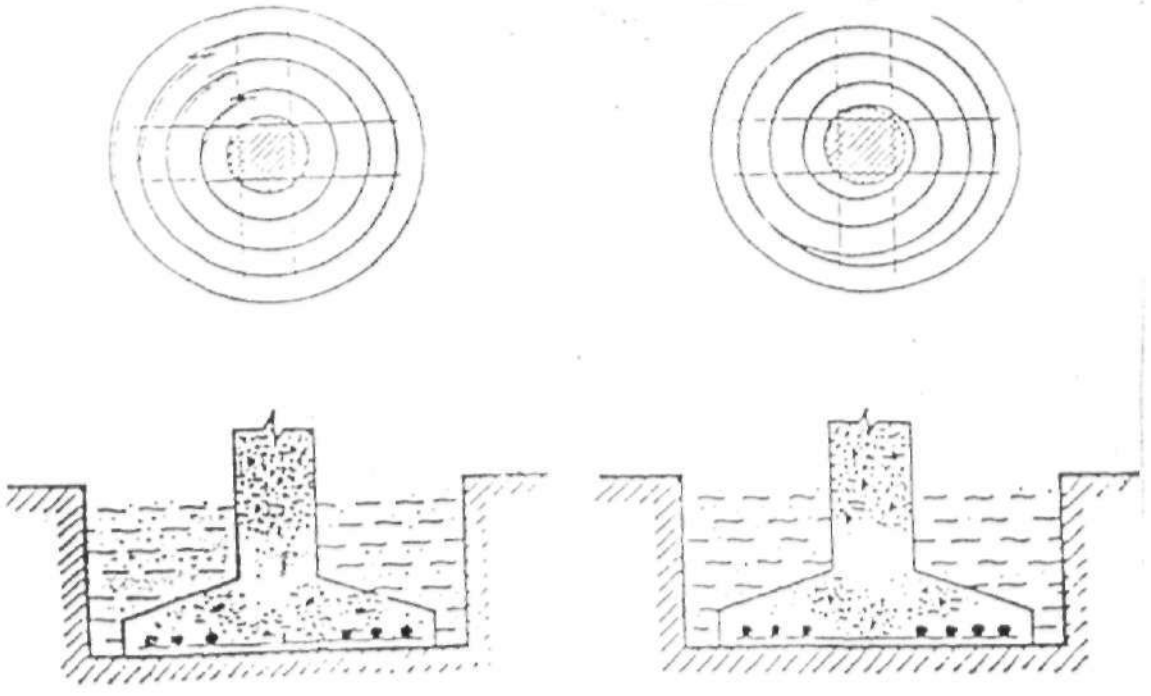
Figure (6.2c): Hollow footing with flat bottom

Figure 6.2(a,b&c): Different Shapes of the Shell Raft Foundations (Ejeze and Gorge, 1987).

spread footings in order to ensure that the bearing pressure are within tolerable limits. However, this will have the effect of substantially increasing the cost, thereby rendering the project economically non-feasible. The cost can be reduced by using circular or spiral reinforcement. Experience has proved that this can yield savings of up to 40% of the steel (Ejezie and George,1987). An example of this application is as shown in Figure 6.3a - 6.3b. The other accompanying problem of excessive volume and weight of concrete can be effectively controlled by using truncated cone shells shown in Figure 6.3c :

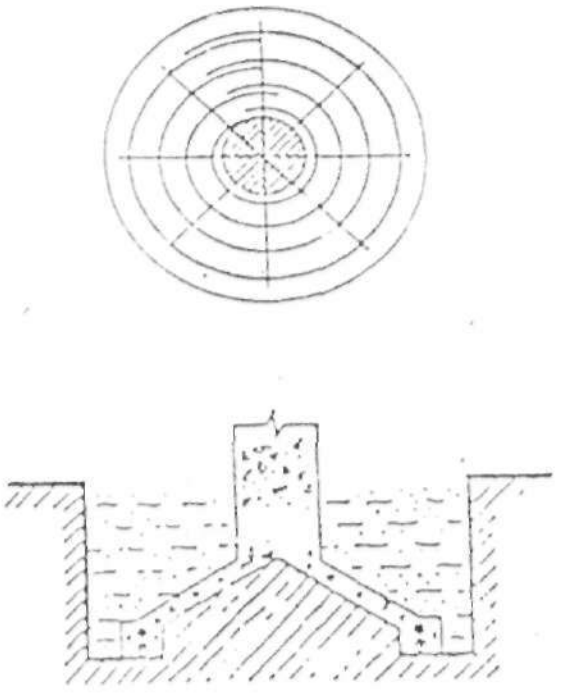
(d) USE OF LIGHTWEIGHT CONSTRUCTION: This is a common practice on industrial estates which are often built over poor ground, such as reclaimed lands and old refuse tips. Steel construction with lightweight flexible cladding reduces settlements and is more tolerant of differential settlements than more traditional block structures. For those who find it acceptable, timber frame construction may be used for housing, to keep down weight, although the use of traditional plaster finishes and brick to render outer walls reduces some of the advantages.

(e) DELAYED APPLICATION OF FINISHES: In many cases, the distortion caused by differential settlement are not a threat to the stability of the structure: the main problem is the damage to surface finishes such as plastering and rendering. If it is estimated that settlement will be substantially completed after several months rather than years, then a delay of a few months before completing surface finishes can reduce subsequent damage. The advantage gained is likely to be better than anticipated because most settlements takes place quicker than the estimated time.



(6·3a): Footing with circular reinforcement

6·3b Footing with spiral reinforcement



6·3c : Truncated-cone footing

Figure 6.3(a,b&c): **Use of Circular and Spiral Reinforcement for Spread Footings (Ejeze and Gorge, 1987).**

This option is quite suitable for the soil condition in Ogolo-Ama, where the rate of settlement is high and often completed under a short period of time due to the sandy nature of the underlying weak clays. In the case of the distressed classroom block, if the existing cracks are repaired, the effect of the settlement will be covered since the settlement process has been confirmed negligible.

Chapter 7

CONCLUSION AND RECOMMENDATION

7.1 Conclusion

In the course of this investigation, a study of the causes of foundation failures of lightly loaded buildings in the sandfill areas of the Niger delta is carried. The result of the preliminary survey of these reclaimed areas, necessitated the detailed geotechnical investigation of the subsoil within the vicinity of the two distressed buildings. The results of both field and laboratory investigation on soil samples led to the recommendations of design criteria that will enable adequate and cost effective foundations to be designed and constructed in the area. Based the results presented so far , the following conclusions can be drawn:

(i) The investigation disclosed that the existing foundation problems in the hydraulic sandfill areas of the Niger delta are mainly due to the effect of the environmental and developmental factors prevalent in the area. These factors include, the high ground water level, the decaying process of the mangrove plant debris, the shallow sandfill thickness, weak consistency of the underlying organic clay, inappropriate use of the reclaimed areas and the consolidation characteristics of the organic peaty clays.

(ii) The ground water levels at the time of the investigation were very close to the ground level between 0.4 – 0.6m in Borikiri and 0.6-1.0m in Ogolo-Ama. In general, the ground water conditions at the sites at the time of investigation were highly unfavorable for shallow foundations.

(iii) The investigation revealed that the thickness of the sandfill in the area investigated, is generally too shallow and variable to support foundation loads from the distressed buildings without excessive settlements.

iv. The underlying weak organic clays are very soft to soft in consistency, the compressibility is high to very high, the thickness of the peaty clays are variable within the vicinity of the two buildings. Thus, it can hardly support any structural load transferred from the distressed structures without differential settlement.

v. The excessive differential settlement of the distressed buildings are due to the consolidation of the underlying weak organic peaty clay, the amount of consolidation settlement induced by the organic peaty clay is increased by the decomposition of the decaying mangrove vegetation.

vi. From the settlement results (Table 6.2), and the deep cracks (<0.3mm) on the two distressed buildings investigated. The deep cracks are due to excessive differential settlements of the foundations. Based on the provision of BS 8110, the residential building at Borikiri and the classroom block at Ogolo-Ama have failed under serviceability limit state of failure.

vii. Findings in this study has proved that some parts of the sandfilled areas can be utilized safely for lightly loaded buildings without any soil improvement. Thus, to enhance optimum use of these areas, a geotechnical investigation is necessary prior to any construction.

7.2 Recommendations

A study of the solutions proposed above, shows that although the presentation on improvement of the sandfilled areas is quite interesting, the technology and cost involved make it not attractive especially considering the lightly loaded residential buildings which are the common structures on the reclaimed areas of the Niger delta. Considering the foundation types, various types of foundation suitable for the subsoil condition of these reclaimed areas were reviewed and the several modifications to reduce the foundation load are also considered. To remedy the existing foundation problems in the sandfilled areas of the Niger delta, the following are recommended:

(i) The existing ground levels where sites have not been developed should be raised by additional sandfilling to heights varying between 1.0 and 3.0 m.. Where preloading method is adopted to improve the area before development, the surcharge thickness should be determined depending on the urgency of the construction as shown in Figure 6.1a & 6.1b.

(ii) To produce a competent load bearing layer for shallow foundations suitable for lightly loaded structures in the sandfilled areas and also keep the founding level reasonably above the ground water table, a minimum sandfill thickness of 3.0m should be placed in unclaimed areas. Where economic considerations limit the filling of the entire reclaimed areas at the same time, planned area by area sandfilling is suggested and the site utilized in stages.

(iii) In the sandfilled areas where the thickness of the underlying weak clay is less than 3.0m e.g. Ogolo-Ama, for big projects the organic clay should be completely removed and replaced with well compacted sand. The sand backfill should be raised to the desired elevations for the foundations. However, it should be borne in mind that the high water table in the area will demand dewatering of the excavation as to allow the construction of the foundations.

(iv) Government or the Oil Companies should enlighten the public especially the riparian owners on the use of the hydraulic sandfill, provide security for the reclaimed areas immediately after reclamation, in order to avoid inappropriate use of the reclaimed areas.

(v) Based on varying thickness of both the sandfill and the underlying organic clays, differential settlement is expected in the sandfilled areas, the use of pad footing for foundation should be avoided. Where pad foundation is adopted, ground beams should be used to tie the foundation to ensure certain degree of distortion as expected from the poor subsoil condition.

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(vi) Only light structures such as bungalow, one or two storey buildings can perform satisfactorily with shallow foundations in such reclaimed areas, heavy structures should not be erected. However, where heavier structures are to be erected at the site, piled foundations should be considered.

(vii) Hydraulically sandfilled area is a reclaimed land, like any other man made soil, poor engineering properties are to be expected and designed for. This can only be done properly if a detailed geotechnical investigation is embarked on before the utilization of these areas. In other word, a geotechnical investigation is a necessity before the development of the hydraulic sandfill areas.

(viii) Where economic constrain limits a geotechnical consultant, the relevant government authority should endeavor to make the geotechnical reports covering the sandfilled areas accessible to the landowners. This will facilitate adequate use of these areas, since the huge financial commitments necessary for the soil investigation on these areas may limit individual approach to the proper development of the sandfilled areas.

(ix) Where development of the reclaimed areas are handled by the Oil companies, PTF, OMPADEC and other big establishment. The community leaders concern should insist on a thorough geotechnical appraisal of the site proir to development.

(x) For future reclamation of more hydraulic sandfilled areas in the Niger delta, it is recommended that cutting and cleaning of the mangrove vegetation before the placement of the sandfill be given top priority . This will reduce the organic content of the underlying soil and preclude the adverse effect of the decomposing process on the geotechnical properties of the subsoils.

(xi) Foundations of buildings on these reclaimed areas should be placed between 0.8 - 1.2m, to avoid transfer of substantial foundation load to the weak organic peaty clays.

It can therefore be seen that the recommendation so far presented is quite effective for shallow foundation design in the reclaimed areas of the Niger delta. In considering foundations suitable for this types of reclaimed areas, it was observed that most shallow foundation, strip, raft etc. can be utilized successfully in the areas provided the environmental and developmental factors in these reclaimed areas are incorporated in the design. Besides, the underlying organic peaty clays are not used either directly or indirectly as foundation soil. Lastly, it is recommended that for effective performance of the foundation, all proposals put forward regarding consideration of environmental and developmental factors be adopted. It is hoped that by these recommendations, design criteria capable of adequately catering for deficiencies in previous designs on the sandfilled areas of the Niger delta may have been considered.

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APPENDICE A_1 & A_2

ATTERBERG LIMITS TEST RESULTS

A.B.U. ZARIA

Project: M.Sc./ENG/0472/93

Operator: Ekpo, A.U Job: M.Sc./ENG/04742/93 Site: Bonkoi Landfill

Date: August, 1996 Borehole No: 1

Depth of sample: 3.00-3.45m

Sample No: 4

LIQUID and PLASTIC LIMITS

Liquid limits using the Casagrande apparatus

Test details: Proportion of sample retained on 425µm B.S. test sieve: %

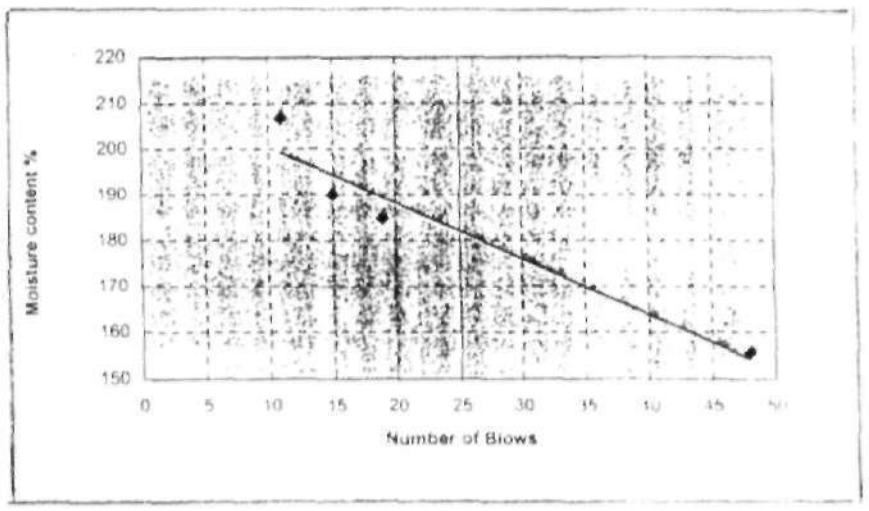
Soil condition: natural moisture content, air dried, oven dried, unknown (delete as appropriate)

Liquid limit machine No.....

Soil equilibrated with water for .....h

Test No.	1	2	3	4	5	6
Type of test	LL	LL	LL	LL		PL
No. of blows (liquid limit test)	48	19	15	11		
Container No.	31	7	27	164		164
Mass of wet soil + container	15.4	18.1	18.1	17.9		15.6
Mass of dry soil + container	11.2	12.0	11.8	11.7		12.8
Mass of container	8.5	8.7	8.5	8.7		8.7
Mass of moisture	4.2	6.1	6.3	6.2		2.8
Mass of dry soil	2.7	3.3	3.3	3.0		4.1
Moisture content	155.6	184.8	190.1	206.7		68.3

Type of test: Natural moisture content (N), Liquid Limit (LL), Plastic limit (PL)



Results:

Liquid Limit (LL)	182%
Plastic Limit (PL)	68.3%
Plasticity Index (PI)	114%

A.B.U. ZARIA

Project: M.Sc./ENG/0472/93

Operator: Ekpo, A.U. Job. M.Sc./ENG/04742/93 Site Borokiri Sandhill

Date: August, 1996 Borehole No 1

Depth of sample: 5.25-5.70m

Sample No 7

LIQUID and PLASTIC LIMITS

Liquid limits using the Casagrande apparatus

Test details: Proportion of sample retained on 425μm B.S. test sieve %

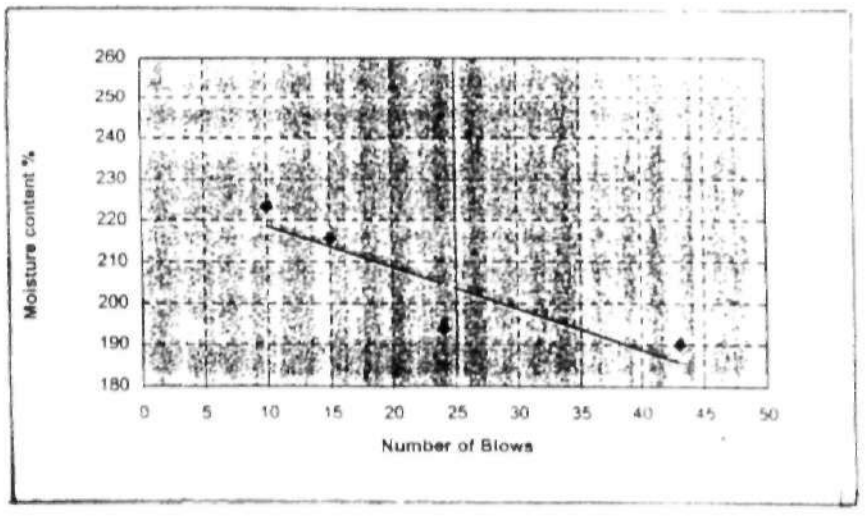
Soil condition : natural moisture content, air dried, oven dried (delete as appropriate)

Liquid limit machine No .....

Soil equilibrated with water for h

Test No.	1	2	3	4	5	P
Type of test	LL	LL	LL	LL		PL
No. of blows (liquid limit test)	43	24	15	10		
Container No.	159	12	B	10		162
Mass of wet soil + container	17.2	17.9	18.8	18.4		17.3
Mass of dry soil + container	11.5	11.7	11.9	11.7		14
Mass of container	8.5	8.5	8.7	8.7		8.7
Mass of moisture	5.7	6.2	6.9	5.7		3.3
Mass of dry soil	3.0	3.2	3.2	3.0		5.3
Moisture content	190.0	193.8	215.3	223.3		62.3

Type of test: Natural moisture content (N); Liquid Limit (LL); Plastic limit (PL)



Results:

Liquid Limit (LL)	203%
Plastic Limit (PL)	62.3%
Plasticity Index (PI)	141%

A.B.U. ZARIA

Project: M.Sc./ENG/0472/93

Operator: Ekpo, A.U.
Date: February, 1997

Job: M.Sc./ENG/04742/93

Site: Oggole Anas
Borehole No: 2
Depth of sample: 2.25-2.70m
Sample No.: 3

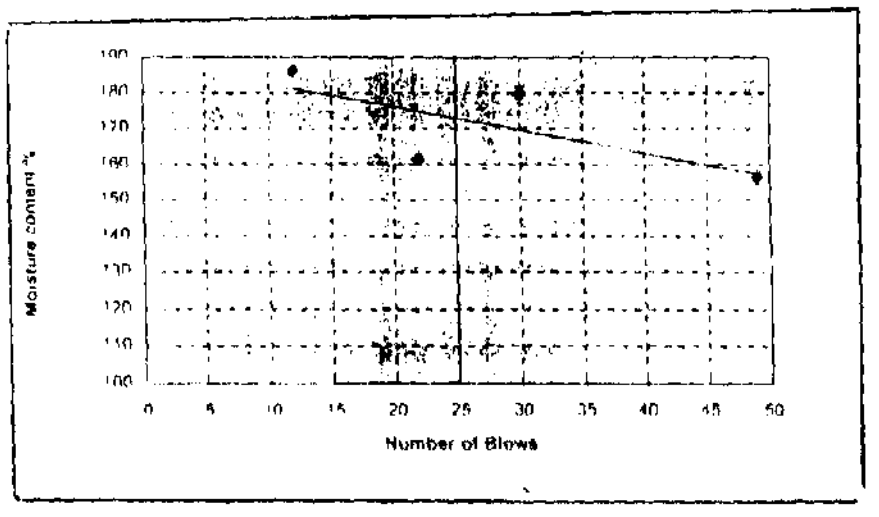
LIQUID and PLASTIC LIMITS

Liquid limits using the Casagrande apparatus

Test details: Proportion of sample retained on 425μm B.S. test sieve ... %
Soil condition: natural moisture content, air dried, unknown (delete as appropriate)
Liquid limit machine No.
Soil equilibrated with water for ... h

Test No.	1	2	3	4	5	6
Type of test	LL	LL	LL	LL		PL
No. of blows (liquid limit test)	49	30	22	12		
Container No.	166	A1	151	160		31
Mass of wet soil + container	16.7	16.8	16.6	17.0		17.2
Mass of dry soil + container	11.7	11.8	11.6	11.6		13.9
Mass of container	8.5	8.7	8.5	8.7		8.5
Mass of moisture	5.0	5.2	5.0	5.4		3.3
Mass of dry soil	3.2	2.9	3.1	2.9		5.4
Moisture content	156.3	179.3	161.3	186.2		61.1

Type of test: Natural moisture content (N), Liquid Limit (LL), Plastic limit (PL)



Results:
Liquid Limit (LL) 179%
Plastic Limit (PL) 61.1%
Plasticity Index (PI) 112%

A.B.U. ZARIA

Project: M.Sc./ENG/0472/93

Operator: Ekpo, A.U. Job: M.Sc./ENG/0472/93 Site: Ogoja Area

Date: February, 1990 Borehole No: 2

Depth of sample: 4.50-4.95m

Sample No: 6

LIQUID and PLASTIC LIMITS

Liquid limits using the Casagrande apparatus

Test details: Proportion of sample retained on 425μm B.S. test sieve: %

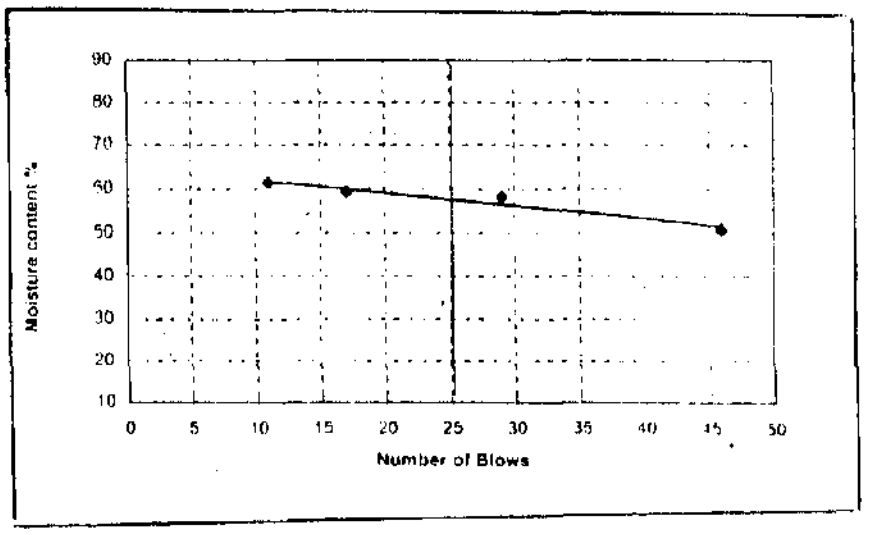
Soil condition: natural moisture content, air dried, oven dried, unknown (delete as appropriate)

Liquid limit machine No.:

Soil equilibrated with water for: h

Test No.	1	2	3	4	5	6
Type of test	LL	LL	LL	LL		PL
No. of blows (liquid limit test)	46	29	17	11		
Container No	162	13	160	24		158
Mass of wet soil + container	20.0	20	20	20		18.3
Mass of dry soil + container	16.3	15.2	15.8	15.7		16.8
Mass of container	8.7	8.3	8.7	8.7		8.7
Mass of moisture	3.7	4.3	4.2	4.3		1.5
Mass of dry soil	7.3	7.4	7.1	7		8.1
Moisture content	50.7	58.1	59.2	61.4		18.5

Type of test: Natural moisture content (N), Liquid Limit (LL), Plastic limit (PL)



Results: Liquid Limit (LL) 58.0%

Plastic Limit (PL) 18.6%

Plasticity Index (PI)

APPENDICE B_1 & B_2

'STRESS VERSUS STRAIN' CURVES

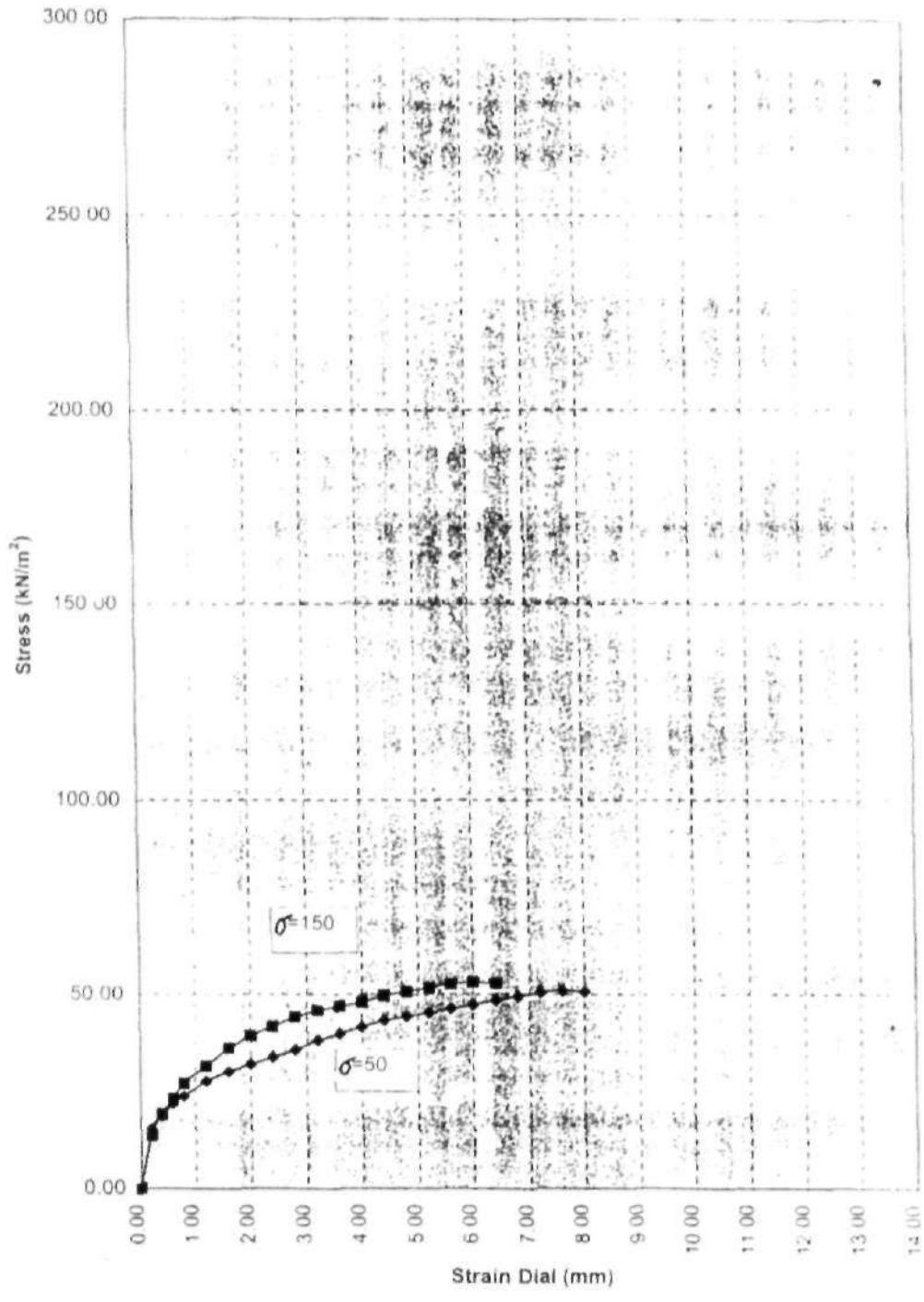
Project: M.Sc/ENG/04742/93
Borehole No.1
Depth: 3.00-3.45m

A.B.U. ZARIA

Quick Undrained Triaxial Test

Site: Borkin Sarafin
Location: Port Harcourt
Date: 25/03/96

Stress versus Strain



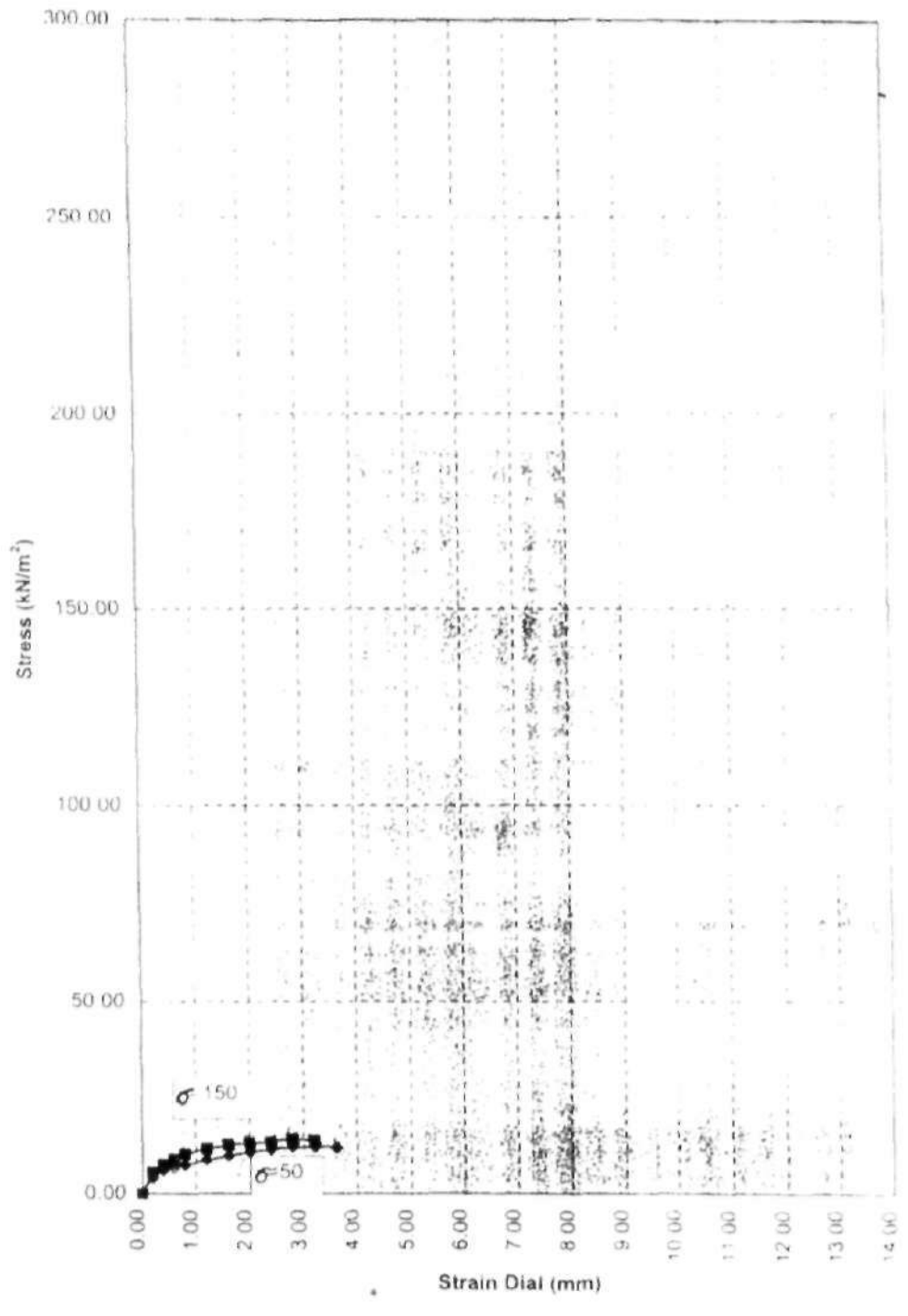
1

Project: M Self/N63/84/142/93
Borehole No 2
Depth: 2.25-2.70m

A.B.U. ZARIA
Quick Undrained Triaxial Test

Site: Barnawa Samaru
Location: Post Graduate
Date: 25/03/96

Stress versus Strain

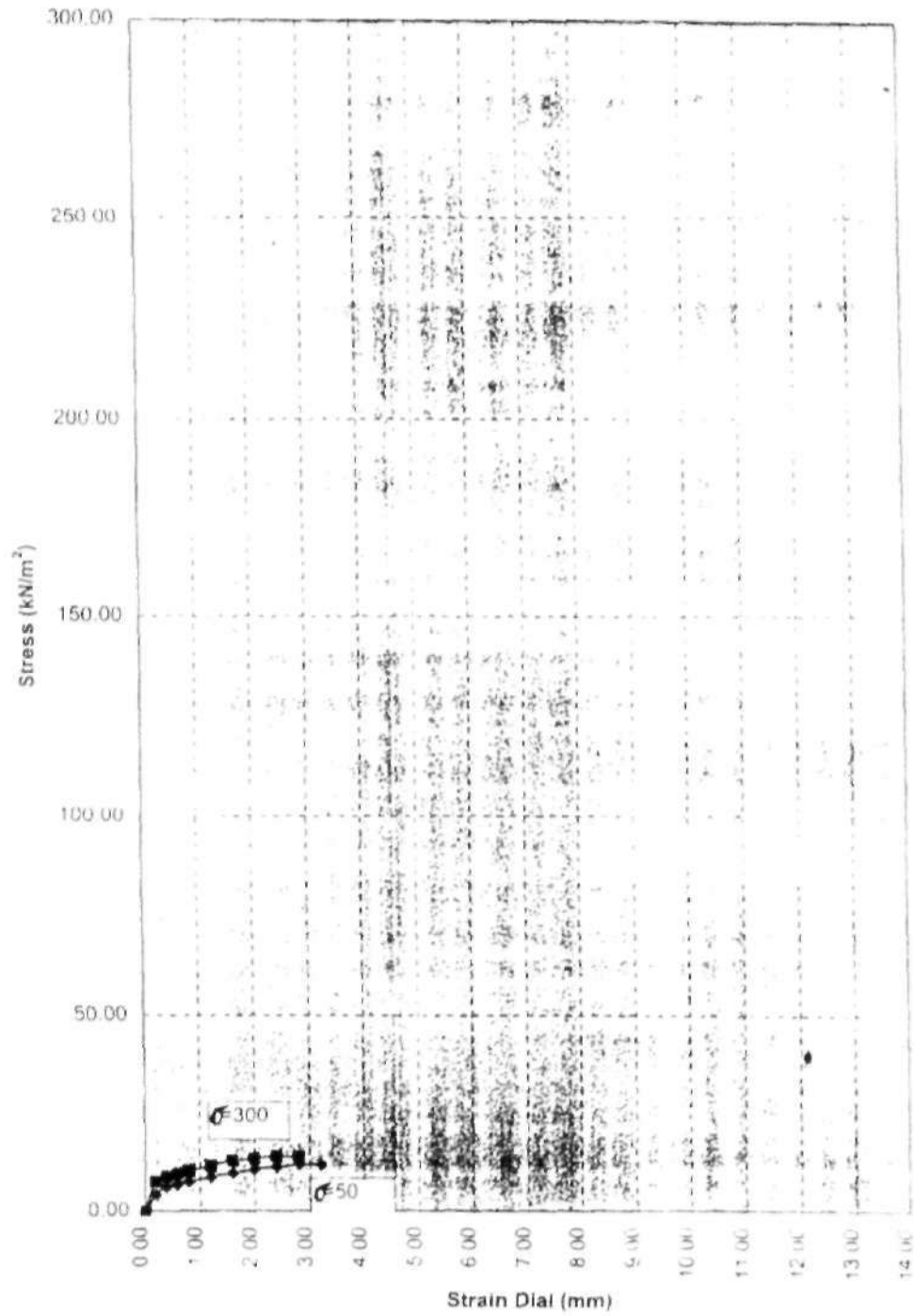


Project: M.Sc/ENG/0474/93
Borehole No.2
Depth: 2.25-2.70m

A.B.U. ZARIA
Quick Undrained Triaxial Test

Site: Ogolo-Amal
Location: Okuku
Date 24/03/97

Stress versus Strain

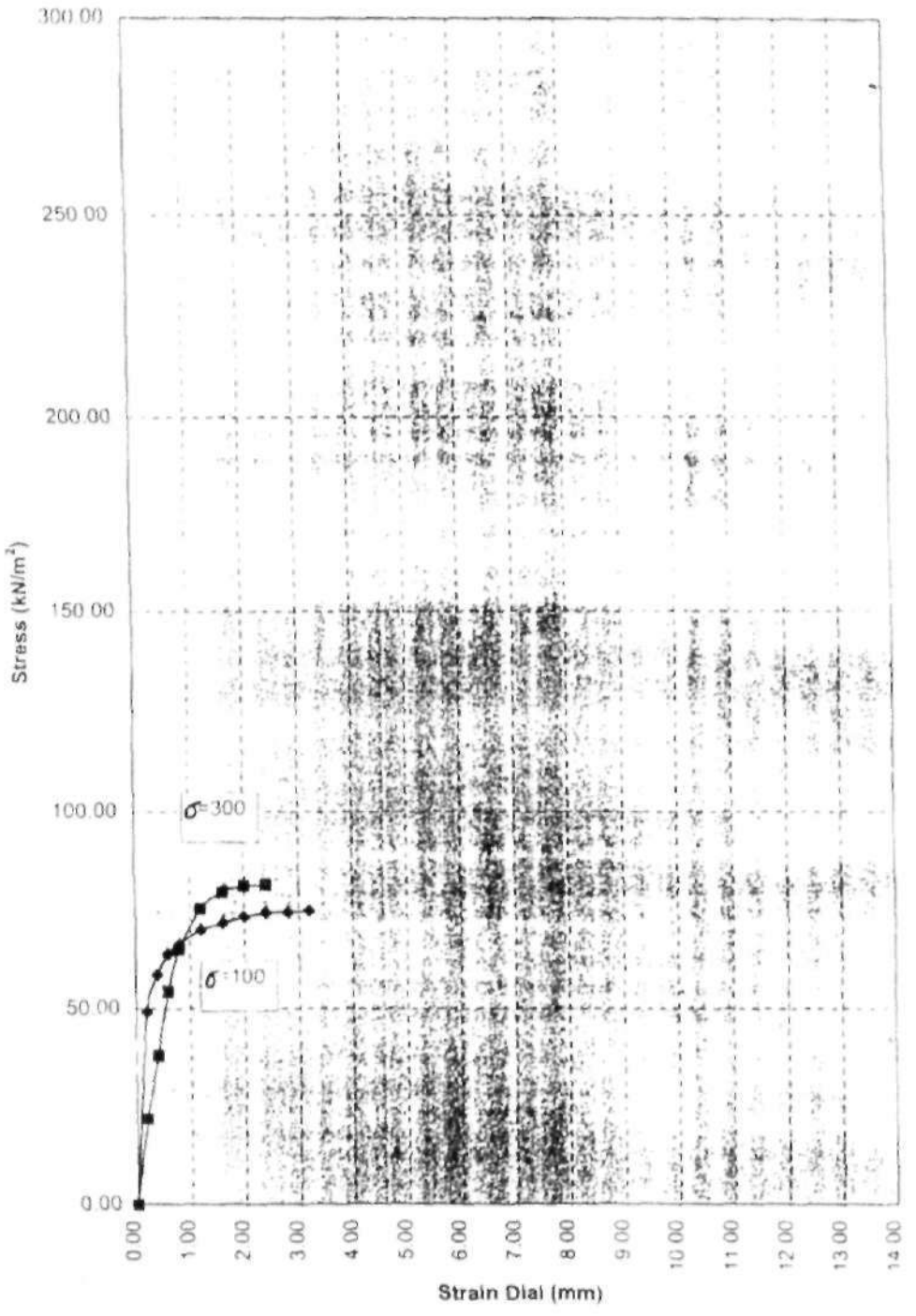


Project: M.Sc/ENG/04742/93
Borehole No.1
Depth: 4.50-4.95m

A.B.U. ZARIA
Quick Undrained Triaxial Test

Site: Ogolo-Ama
Location: Okirika
Date 7/01/97

Stress versus Strain



APPENDIX C

CALCULATION OF COMPRESSIBILITY PARAMETERS

APPEDIX C

Analysis of Compressibility Parameters

In the analysis of m_v and k presented below, the C_v values from the square root-of-time method were used while other parameters are based on the laboratory test results for Borikiri sandfilled area with the mentioned sample descriptions.

Step 1: Presentation of Sample Identification Data:

Borehole No.: 1

Sample No.:4

Depth of Sample: 3.00-3.45m

Soil Description: Black, Soft silty Organic Peaty Clay with Substantial Decomposed Plant Debris.

Sample Detail: Thickness of sample (H) = 2 cm

Weight of dry sample(M_d) = 36.2g

Area of Specimen (A) = 19.63 cm^2

Volume of Specimen (V) = 39.26 cm^3

Step 2: Determination of Natural Moisture Content

Initial Weight of Specimen, (M_0)=59.9g

Final Weight of Specimen, (M_f) =47.7g

Initial Moisture Content, $(m_0) = \frac{M_0 - M_d}{M_d} * 100 = \frac{59.9 - 36.2}{36.2} * 100 = 65.55\%$

Final Moisture Content, $(m_f) = \frac{M_f - M_d}{M_d} * 100 = \frac{47.7 - 36.2}{36.2} * 100 = 31.8\%$

Step 3: Bulk Unit Weight ρ_b

Bulk density, $\rho_b = \frac{\text{Total Mass of Soil}}{\text{Total Volume of Soil}} = \frac{59.9}{39.26} = 1.526 Mg/m^3$

Bulk Unit Weight, $\gamma_b = \rho_b * g = 1.526 * 9.807 kN/m^3 \approx 15 kN/m^3$

Dry Unit Weight, $\gamma_d = \rho_b * g = \frac{36.2}{39.26} Mg/m^3 * 9.807 kN/m^3 = 9.04 kN/m^3$

Step 4: Degree of Saturation S_r

Assumption: The Sample is fully saturated

Volume of water $V_w = 59.9 - 36.2 = 23.7 cm^3$

Volume of solids $V_s = 39.26 - 23.7 = 15.56 cm^3$

Specific Gravity of Solid Particles $Gs = \frac{Ws}{Vs} = \frac{36.2}{15.56} = 2.33$

Step 5: Determination of Void Ratio

a) Height of Solids (H_s), $H_s = \frac{36.2}{19.63 * 2.33 * 1} = 0.791 cm$

b) Initial Height of Voids $H_v = H - Hs$

$= 2.00 - 0.791$

$= 1.209 cm$

c) Initial Void Ratio $e_o = H_v/H_s = 1.209/0.791 = 1.528$

The initial void ratios obtained were used to prepare the e tables as shown in table C below:

Table C .: Typical Analysis of the void Ratio (e)

Pressure (kPa)	Height at end of consolidation H (cm)	Height of solid Hs (cm)	Hv =H Hs (cm)	e = H/H
Loading				
0	2.000	0.791	1.207	1.528
25	1.895	0.791	1.104	1.396
50	1.814	0.791	1.023	1.293
100	1.720	0.791	0.929	1.174
200	1.605	0.791	0.814	1.029
400	1.480	0.791	0.689	0.871
800	1.350	0.791	0.559	0.707
1600	1.281	0.791	0.490	0.619
Unloading				
800	1.289	0.791	0.498	0.630
400	1.306	0.791	0.515	0.651
25	1.325	0.791	0.534	0.675

With the values of void ratios presented in the table above, the M_v and k values were evaluated for each pressure range as follows:

Step 6 Coefficient of Volume Compressibility (M_v)

The coefficient of Volume Compressibility (M_v) is commonly expressed as:

$$M_v = \frac{e_0 - e_1}{(1+e_0)\Delta P} \qquad (3.8)$$

where e_0, e_1 are the void ratios and ΔP is the change in pressure at the end of consolidation.

(1.0) 0-25 kPa $m_v = (1.528-1.396)/(2.528*25)$

$m_v = 20.89*10^{-4} m^2/kN$

m_v = 2.09 m^2/MN

(2.0) 25-50 kPa

m_v = (1.396-1.293)/(2.396*25)
m_v = 17.20*10^{-4} m^2/kN
m_v = 1.72 m^2/MN

(3.0) 50-100 kPa

m_v = (1.293-1.174)/(2.396*50)
m_v = 10.38*10^{-4} m^2/kN
m_v = 1.04 m^2/MN

(4.0) 100-200 kPa

m_v = (1.174-1.029)/(2.174*100)
m_v = 6.67*10^{-4} m^2/kN
m_v = 0.67 m^2/MN

(5.0) 200-400 kPa

m_v = (1.029-0.871)/(2.029*200)
m_v = 3.89*10^{-4} m^2/kN
m_v = 0.39 m^2/MN

(6.0) 400-800 kPa

m_v = (0.871-0.707)/(1.871*400)
m_v = 2.19*10^{-4} m^2/kN
m_v = 0.22 m^2/MN

(7.0) 800-1600 kPa

m_v = (0.707-0.619)/(1.707*800)
m_v = 0.64*10^{-4} m^2/kN
m_v = 0.06 m^2/MN

Step 7 <u>Coefficient of Permeability k</u>

The equation used in the determination of k, was obtained by making k the subject of Equation 3.6, this is expressed as:

$$k = C_v * m_v * \gamma_w$$

(1.0) 0-25 kPa

k =0.642*20.89*10^{-4}*9.807*3.169*10^{-6} cm/sec

		$k = 0.642*20.89*9.807*3.169*10^{-10}$ cm/sec
		$k = 4.17*10^{-8}$ cm/sec
(2.0)	25-50 kPa	$k = 0.30*17.20*10^{-4}*9.807*3.169*10^{-6}$ cm/sec
		$k = 0.30*17.20*9.807*3.169*10^{-10}$ cm/sec
		$k = 1.60*10^{-8}$ cm/sec
(3.0)	50-100 kPa	$k = 0.22*10.38*10^{-4}*9.807*3.169*10^{-6}$ cm/sec
		$k = 0.22*10.38*9.807*3.169*10^{-10}$ cm/sec
		$k = 0.71*10^{-8}$ cm/sec
(4.0)	100-200 kPa	$k = 0.28*6.67*10^{-4}*9.807*3.169*10^{-6}$ cm/sec
		$k = 0.28*6.67*9.807*3.169*10^{-10}$ cm/sec
		$k = 0.58*10^{-8}$ cm/sec
(5.0)	200-400 kPa	$k = 0.29*3.89*10^{-4}*9.807*3.169*10^{-6}$ cm/sec
		$k = 0.29*20.89*9.807*3.169*10^{-10}$ cm/sec
		$k = 0.35*10^{-8}$ cm/sec
(6.0)	400-800 kPa	$k = 0.21*2.19*10^{-4}*9.807*3.169*10^{-6}$ cm/sec
		$k = 0.21*2.19*9.807*3.169*10^{-10}$ cm/sec
		$k = 0.14*10^{-8}$ cm/sec
(7.0)	800-1600 kPa	$k = 0.30*0.64*10^{-4}*9.807*3.169*10^{-6}$ cm/sec
		$k = 0.30*0.64*9.807*3.169*10^{-10}$ cm/sec
		$k = 0.06*10^{-8}$ cm/sec

For detailed analysis of the compressibility parameters the seven steps of consolidation test analysis presented above were carried out on the sample obtained from the two investigation sites. The results of the compressibility analysis obtained using the three methods of Cv analysis are presented in Appendices D_1 and D_2 respectively for the two sites.

APPENDICE D_1 & D_2

SUMMARY OF COMPRESSIBILITY RESULTS

Table 3-8a - Consolidation Test Results

Method of Cv analysis: Square-root-of-time

Location: Borikiri

B/H No.	Depth of Sample (m)	Pressure Range (kN/m²)	Coefficient of Consolidation Cv (M^2/yr)	Coefficient of Volume Compressibility Mv (M^2/MN)	Coefficient of Permeability K (cm/sec)	Description of Sample
1	1.00-3.45	0-25	0.64	2.100	4.10E-8	Black, very soft, organic peaty CLAY with substantial decomposed plant debris (Chikoko mud)
		25-50	0.30	1.710	1.59E-8	
		50-100	0.22	1.036	0.71E-8	
		100-200	0.28	0.669	0.58E-8	
		200-400	0.29	0.389	0.35E-8	
		400-800	0.21	0.220	0.14E-8	
		800-1600	0.29	0.064	0.06E-8	
	5.25-5.70	0-25	0.59	2.10	3.89E-8	Dark grey, very soft, silty organic peaty CLAY with decomposed organic matter
		25-50	0.33	1.837	1.88E-8	
		50-100	0.40	1.594	1.98E-8	
		100-200	0.22	0.980	0.67E-8	
		200-400	0.35	0.557	0.61E-8	
	8.00-8.45	0-25	1.81	0.770	5.31E-8	Light, bluish grey, very soft, very silty CLAY (Marine mud)
		25-50	2.47	0.30.	1.39E-8	
		50-100	1.46	0.27?	1.57E-8	
		100-200	1.83	0.17	1.14E-8	
		200-400	2.08	0.106	1.01E-8	
		400-800	3.17	0.059	0.58E-8	
		800-1600	3.11	0.010	0.41E-8	
2	4.50-4.95	0-12.50	4.50	9.360	131E-8	Black, very soft, organic peaty CLAY with substantial decomposed plant debris (Chikoko mud)
		12.50-25	6.32	4.671	97.4E-8	
		25-50	5.69	3.720	65.8E-8	
		50-100	2.90	2.731	24.6E-8	
		100-200	4.34	1.835	24.8E-8	
		200-400	2.62	0.851	7.28E-8	
		0-6.25	6.52	2.080	42.1E-8	Dark grey, very soft, silty organic peaty CLAY with decomposed organic matter
		6.25-12.5	2.36	2.587	19.2E-8	
		12.50-25	0.12	1.273	0.51E-8	
		25-50	0.14	1.216	0.52E-8	
		50-100	0.21	0.770	0.50E-8	
	7.95-8.45	0-25	7.84	0.940	23.6E-8	Light, bluish grey, very soft, very silty CLAY (Marine mud)
		25-50	18.63	0.225	13.1E-8	
		50-100	10.32	0.175	5.61E-8	
		100-200	20.65	0.103	6.70E-8	
		200-400	23.42	0.066	4.42E-8	
		400-800	21.25	0.032	2.11E-8	
		800-1600	20.70	0.017	1.09E-8	

Table 3.8b – Consolidation Test Results

Method of Cv analysis: Square-root-of time

Location: Ogolo-Ama

B/H N.	Depth of Sample (m)	Pressure Range (kN/m²)	Coefficient of Consolidation Cv (m²/yr)	Coefficient of Volume Compressibility Mv (m²/MN)	Coefficient of Permeability K (cm/sec)	Description of Sample
		0-25	1.14	2.320	8.22E-8	Black, soft organic peaty CLAY with substantial decomposed plant vegetation
		25-50	1.87	5.032	29.2E-8	
	2.25-2.70	50-100	0.13	1.761	0.71E-8	
		100-200	0.14	0.985	0.43E-8	
		200-400	0.29			
		0-25	1.82	0.740	4.19E-8	Dark grey, very soft, very silty, sandy CLAY
		25-50	1.31	0.265	1.08E-8	
		50-100	6.18	0.215	4.34E-8	
1	4.50-4.95	100-200	10.17	0.150	4.75E-8	
		200-400	15.36	0.082	4.52E-8	
		400-800	4.24	0.032	0.60E-8	
		800-1600	7.74	0.020	0.63E-8	
		0-25	3.09	0.950	8.26E-8	Light bluish grey, very soft, very silty, sandy CLAY becoming very sandy with depth
		25-50	4.13	0.225	2.89E-8	
		50-100	8.22	0.216	5.51E-8	
	6.00-6.45	100-200	7.05	0.125	2.73E-8	
		200-400	15.39	0.059	4.28E-8	
		400-800	24.68	0.044	3.39E-8	
		800-1600	23.43	0.027	1.94E-8	
		0-25	6.64	9.700	195.0E-8	Black, soft organic peaty CLAY with substantial
		25-50	5.65	3.673	82.1E-8	decomposed plant vegetation
		50-100	2.66	3.020	33.4E-8	
	[illegible]-7.70	100-200	4.22	2.331	36.7E-8	
		200-400	2.60	1.935	14.8E-8	
2		400-800	2.22	0.855	6.17E-8	
		0-25	7.86	0.460	23.0E-8	Black, soft organic peaty CLAY with substantial
		25-50	18.71	0.215	13.1E-8	decomposed plant vegetation
		50-100	10.37	0.175	5.66E-8	
	[illegible]-8.35	100-200	20.75	0.104	6.71E-8	
		200-400	23.54	0.060	4.62E-8	
		400-800	21.33	0.032	2.31E-8	
		800-1600	29.72	0.022	1.08E-8	

2.

Table 3-8a - Consolidation Test Results

Method of Cv analysis: Log-of-time

Location: Borikiri

B/H No.	Depth of Sample (m)	Pressure Range (kN/m²)	Coefficient of Consolidation Cv (M²/yr)	Coefficient of Volume Compressibility Mv (M²/MN)	Coefficient of Permeability K (cm/sec)	Description of Sample
		0-25	0.359	2.100	4.18E-8	Black, very soft, organic peaty CLAY with substantial decomposed plant debris (Chikoko mud)
		25-50	0.208	1.710	1.59E-8	
		50-100	0.205	1.036	0.71E-8	
	3.00-3.45	100-200	0.284	0.669	2.58E-8	
		200-400	0.221	0.389	0.35E-8	
		400-800	0.526	0.220	0.11E-8	
		800-1600	0.350	0.064	0.06E-8	
		0-25	0.768	2.120	3.89E-8	Dark grey, very soft, silty organic peaty CLAY with decomposed organic matter
		25-50	0.524	1.837	1.88E-8	
1	5.25-5.70	50-100	0.353	1.594	1.59E-8	
		100-200	0.161	0.980	0.67E-8	
		200-400	0.123	0.557	0.61E-8	
		0-25	2.32	0.700	5.37E-8	Light, bluish grey, very soft, very silty CLAY (Marine mud)
		25-50	2.62	0.305	1.39E-8	
		50-100	1.92	0.277	1.07E-8	
	7.70-8.15	100-200	1.58	0.171	1.11E-8	
		200-400	3.52	0.106	1.04E-8	
		400-800	4.13	0.059	0.58E-8	
		800-1600	2.60	0.035	0.43E-8	
		0-25	1.404	5.362	1.41E-8	Black, very soft, organic peaty clay with substantial decomposed plant debris (Chikoko mud)
		25-50	0.796	4.671	0.95E-8	
		50-100	0.436	3.736	0.75E-8	
	3.00-3.45	100-200	0.351	2.731	0.48E-8	
		200-400	0.413	1.137	0.34E-8	
		400-800	0.176	0.894	0.20E-8	
		0-25	0.176	2.050	0.31E-8	Dark grey, very soft, silty organic peaty CLAY with decomposed organic matter
		25-50	0.102	2.067	0.14E-8	
2	5.25-5.70	50-100	0.071	1.774	0.11E-8	
		100-200	0.047	1.104	0.07E-8	
		200-400	0.047	0.551	0.04E-8	
		0-25	4.17*	0.440	0.02E-8	Light, bluish grey, very soft, very silty clay (Marine mud)
		25-50			0.01E-8	
		50-100	4.417	0.175	0.43E-8	
	7.70-8.15	100-200	1.74*	0.101	0.09E-8	
		200-400	5.54*	0.060	0.42E-8	
		400-800		0.041	0.11E-8	
		800-1600	6.65*	0.017	0.09E-8	

Table 3.8b - Consolidation Test Results

Method of Cv analysis: Log-of-time

Location: Ogolo-Ama

B/H No.	Depth of Sample (m)	Pressure Range (kN/m^2)	Coefficient of Consolidation Cv (M^2/yr)	Coefficient of Volume Compressibility Mv (M^2/MN)	Coefficient of Permeability K (cm/sec)	Description of Sample
1	2.25-2.70	0-25	1.036	2.320	0.22E-8	Black, soft organic peaty CLAY with substantial decomposed plant vegetation
		25-50	0.260	5.032	29.2E-8	
		50-100	0.189	1.761	0.71E-8	
		100-200	0.208	0.985	0.43E-9	
		200-400	-	-	-	
	4.50-4.95	0-25	0.733	0.740	4.19E-8	Dark grey, very soft, very silty, sandy CLAY
		25-50	0.654	0.265	1.08E-8	
		50-100	3.286	0.215	4.11E-9	
		100-200	2.604	0.150	4.75E-9	
		200-400	3.277	0.095	4.52E-8	
		400-800	3.852	0.052	0.69E-8	
		800-1600	3.677	0.026	0.46E-8	
	6.90-7.35	0-25	2.318	0.860	8.26E-8	Light bluish grey, very soft, very silty, sandy CLAY becoming very sandy with depth
		25-50	2.068	0.225	2.89E-8	
		50-100	5.194	0.216	5.51E-8	
		100-200	3.675	0.125	2.73E-9	
		200-400	2.920	0.089	1.29E-8	
		400-800	3.280	0.044	3.10E-8	
		800-1600	4.220	0.027	1.91E-8	
2	2.25-2.70	0-25	2.320	9.360	145.0 E-8	Black, soft organic peaty CLAY with substantial decomposed plant vegetation
		12.50-25	0.581	4.671	62.1E-8	
		25-50	1.038	3.720	33.4E-8	
		50-100	0.702	2.731	36.7E-8	
		100-200	0.826	1.835	14.9E-8	
		200-400	0.352	0.895	6.17E-8	
	6.90-7.35	0-25	4.256	0.940	23.0E-8	Black, soft organic peaty CLAY with substantial decomposed plant vegetation
		25-50	4.256	0.225	13.1E-8	
		50-100	4.634	0.175	5.61E-8	
		100-200	4.256	0.104	6.71E-8	
		200-400	5.574	0.060	4.42E-8	
		400-800	5.840	0.032	2.11E-8	
		800-1600	1.164	0.017	1.08E-8	

Table 3-8a - Consolidation Test Results

Method of Cv analysis: Hyperbola

Location: Borikiri

B/H No.	Depth of Sample (m)	Pressure Range (kN/m²)	Coefficient of Consolidation Cv (m²/yr)	Coefficient of Volume Compressibility Mv (m²/kN)	Coefficient of Permeability K (cm/sec)	Description of Sample
1	3.00-3.45	0-25	0.316	2.100	4.18E-8	Black, very soft, organic peaty CLAY with substantial decomposed plant debris (Chiloko mud)
		25-50	0.104	1.710	1.59E-8	
		50-100	0.174	1.036	0.71E-8	
		100-200	0.168	0.669	0.58E-8	
		200-400	0.248	0.389	0.35E-8	
		400-800	0.200	0.220	0.14E-8	
		800-1600	0.261	0.061	0.06E-8	
	5.25-5.70	0-25	0.922	2.120	3.89E-8	Dark grey, very soft, silty organic peaty CLAY with decomposed organic matter
		25-50	0.288	1.837	1.88E-8	
		50-100	0.212	1.501	1.08E-8	
		100-200	0.081	0.960	0.47E-8	
		200-400	0.149	0.557	0.21E-8	
	9.00-9.45	0-25	3.372	0.100	1.23E-8	Light, bluish grey, very soft, very silty CLAY (Marine mud)
		25-50	1.628	0.365	1.35E-8	
		50-100	5.001	0.227	1.53E-8	
		100-200	2.535	0.117	1.14E-8	
		200-400	2.935	0.104	1.04E-8	
		400-800	4.169	0.059	0.58E-8	
		800-1600	2.675	0.030	0.43E-8	
2	3.00-3.45	0-12.50	0.744	9.140	131E-8	Black, very soft, organic peaty CLAY with substantial decomposed plant debris (Chiloko mud)
		12.50-25	0.331	4.633	61.4E-8	
		25-50	0.243	3.720	65.8E-8	
		50-100	0.124	2.731	24.4E-8	
		100-200	0.209	1.435	24.9E-8	
		200-400		0.894	7.28E-8	
	5.25-5.70	0-25	0.167	2.090	62.1E-8	Dark grey, very soft, silty organic peaty CLAY with decomposed organic matter
		6.25-12.5	0.138	2.981	15.2E-8	
		12.5-25	0.317	1.771	0.52E-8	
		25-50	0.220	1.114	0.53E-8	
		50-100	-	0.715	0.50E-8	
	9.00-9.45	0-25	1.248	0.340	23.0E-8	Light, bluish grey, very soft, very silty CLAY (Marine mud)
		25-50	1.790	0.225	17.1E-8	
		50-100	1.716	0.175	5.64E-8	
		100-200	1.045	0.104	6.70E-8	
		200-400	1.780	0.060	4.42E-8	
		400-800	3.221	0.032	2.11E-8	
		800-1600	1.484	0.017	1.09E-8	

Table 3.8b - Consolidation Test Results

Method of Cv analysis: Hyperbola

Location: Ogolo-Awa

B/H No.	Depth of Sample (m)	Pressure Range (kN/m²)	Coefficient of Consolidation Cv (m²/yr)	Coefficient of Volume Compressibility Mv (m²/MN)	Coefficient of Permeability K (cm/sec)	Description of Sample
1	2.25-2.70	0-25	1.420	7.320	8.22E-8	Black, soft organic peaty CLAY with substantial decomposed plant vegetation
		25-50	0.400	5.032	29.2E-8	
		50-100	0.237	1.761	0.71E-8	
		100-200	0.193	0.985	0.63E-8	
		200-400	-	-	-	
	4.50-4.95	0-25	1.118	0.740	5.15E-8	Dark grey, very soft, very silty, sandy CLAY
		25-50	0.669	0.265	1.05E-8	
		50-100	1.001	0.215	4.19E-8	
		100-200	1.782	0.150	4.75E-8	
		200-400	2.973	0.085	4.52E-8	
		400-800	1.218	0.052	0.63E-8	
		800-1600	1.899	0.026	0.63E-8	
	6.75-7.20	0-25	2.713	1.860	8.26E-8	Light bluish grey, very soft, very silty, sandy CLAY becoming very sandy with depth
		25-50	0.761	0.225	2.85E-8	
		50-100	0.360	0.216	1.53E-8	
		100-200	3.028	0.135	2.73E-8	
		200-400	0.871	0.089	1.28E-8	
		400-800	1.536	0.044	1.19E-8	
		800-1600	1.179	0.027	1.94E-8	
2	2.25-2.70	0-12.50	0.531	9.360	195.0 E-8	Black, soft organic peaty CLAY with substantial decomposed plant vegetation
		12.50-25	0.466	4.673	97.3 E-8	
		25-50	0.327	3.720	33.48E-8	
		50-100	0.174	3.731	36.7E-8	
		100-200	0.207	1.235	14.8E-8	
		200-400	-	0.895	6.1E-8	
	6.75-7.20	0-25	5.367	0.940	20.0E-8	Black, soft organic peaty CLAY with substantial decomposed plant vegetation
		25-50	1.908	0.225	13.3E-8	
		50-100	1.053	0.175	5.64E-8	
		100-200	0.784	0.104	6.71E-8	
		200-400	3.165	0.060	4.42E-8	
		400-800	0.989	0.032	2.11E-8	
		800-1600	0.570	0.017	1.04E-8	

APPENDIX E

e-logP CURVES FOR BORIKIRI AND OGOLO-AMA

CONSOLIDATION TEST

PROJECT A B U

LOCATION BORIKIRI

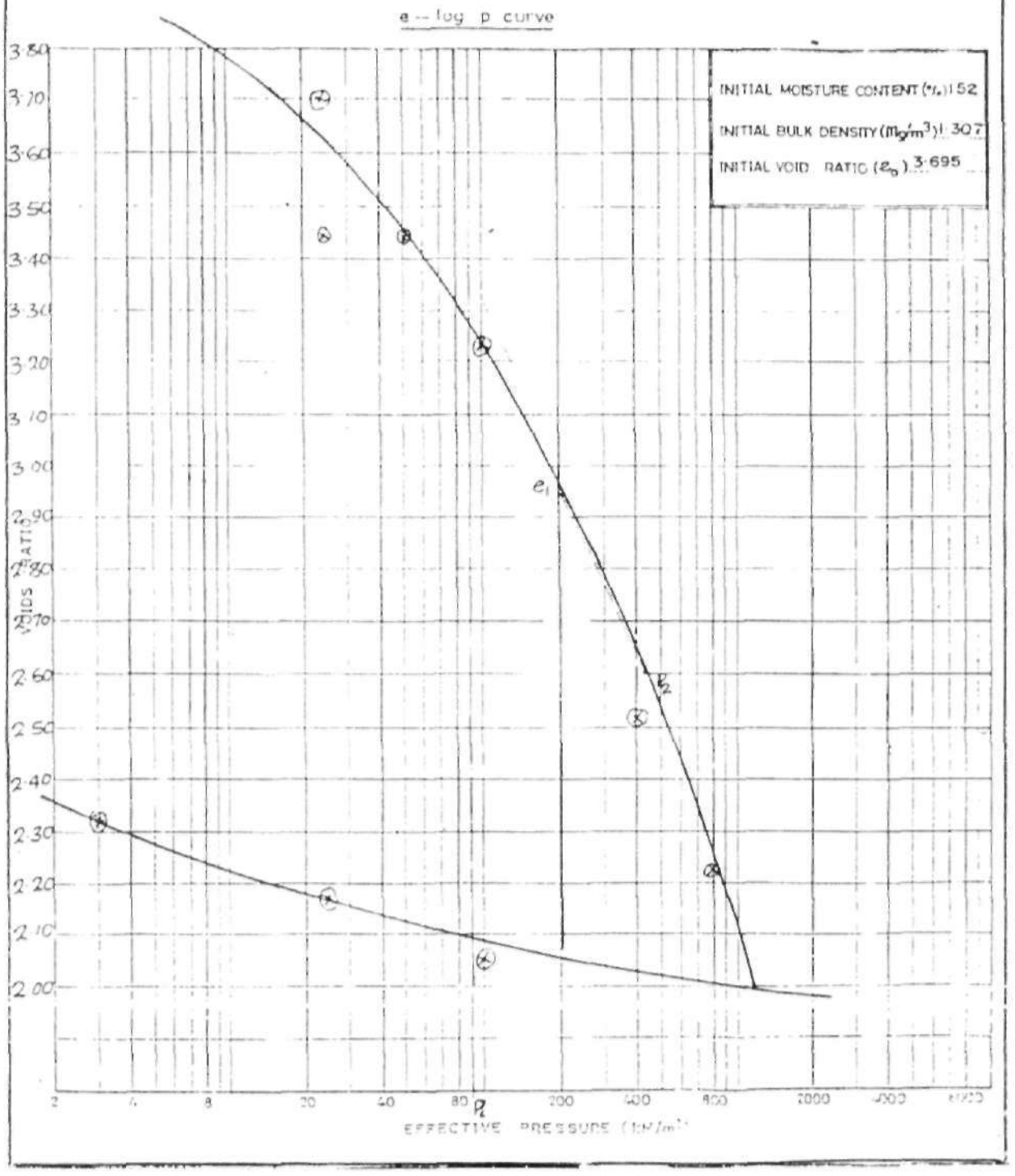
BOREHOLE No 1

SAMPLE No 7

DATE 8/09/96

DEPTH (m) 3.00 - 3.15

e -- log p curve



ENOCH GEORGE ASSOCIATES

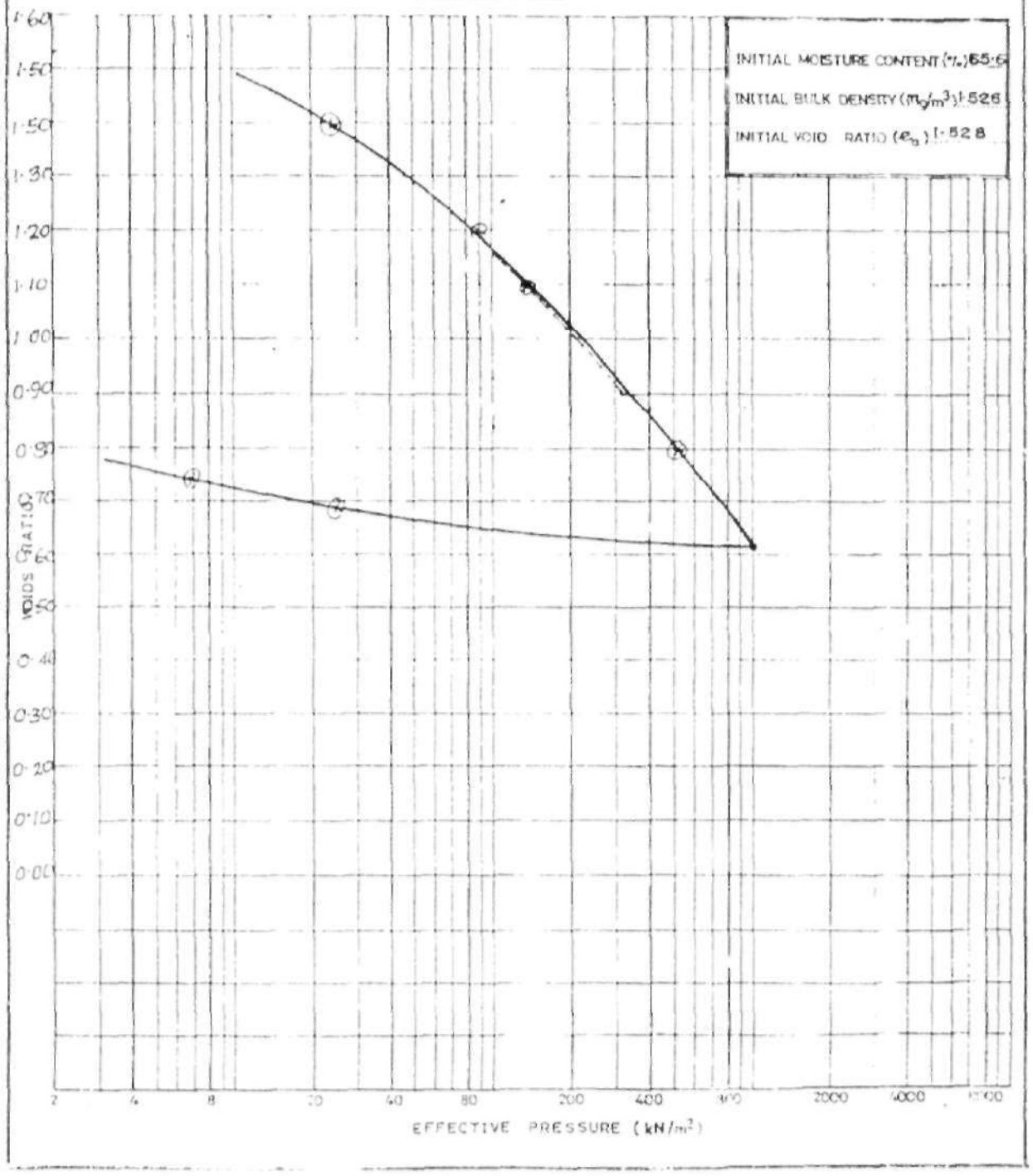
CONSOLIDATION TEST

PROJECT M.Sc/ENG/0474 2/93 DATE 8/04/96

LOCATION BORIKIRI SAND FILL

BOREHOLE No. 1 SAMPLE No. 5 DEPTH (m) 5.25 - 5.70

e — log p curve



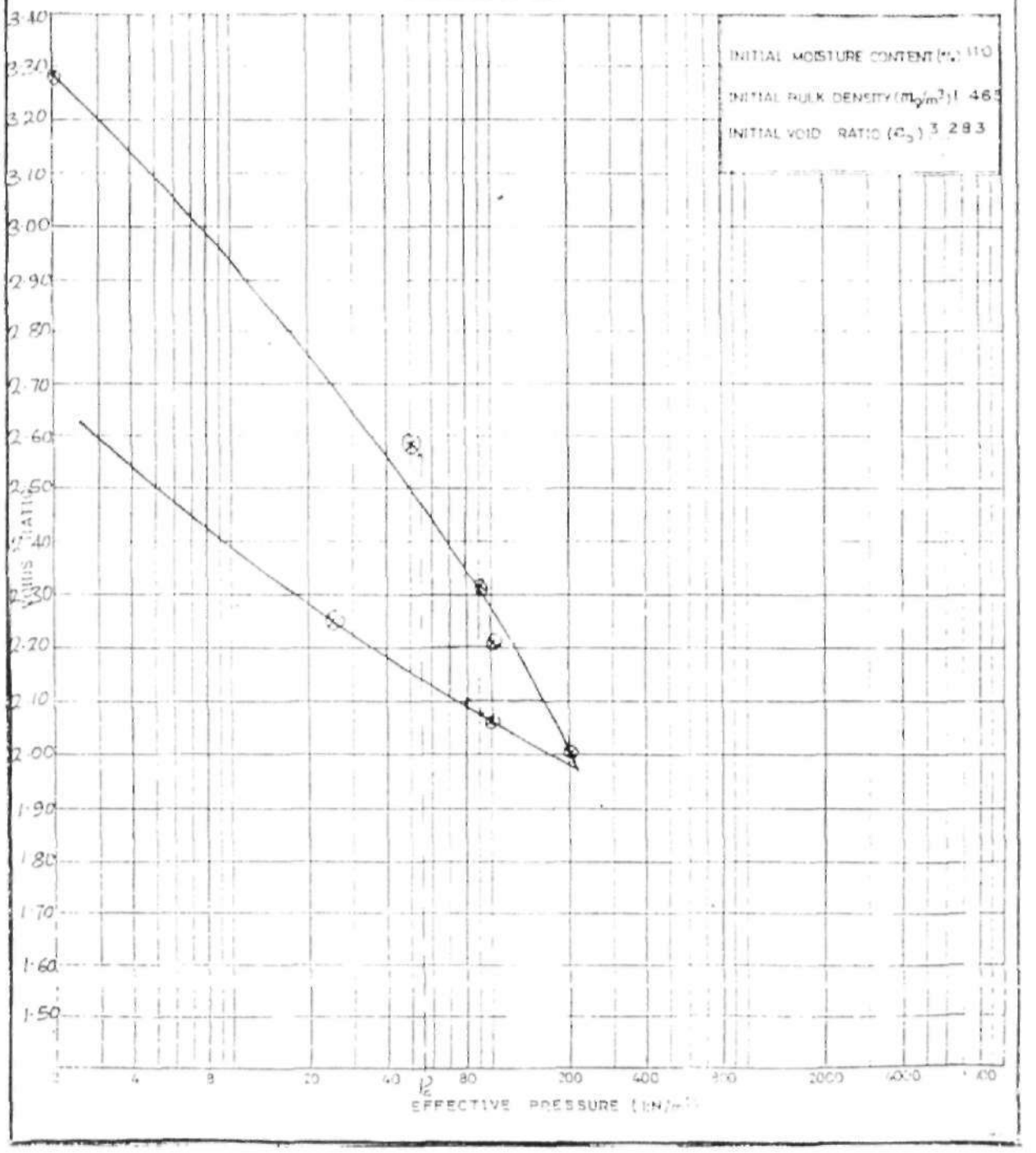
CONSOLIDATION TEST

PROJECT M Sc/ENG/0474/93 DATE MAR 1997

LOCATION UGCLU-AMA

BOREHOLE No 1 SAMPLE No 3 DEPTH (m) 2.25-2.70m

e — log p curve



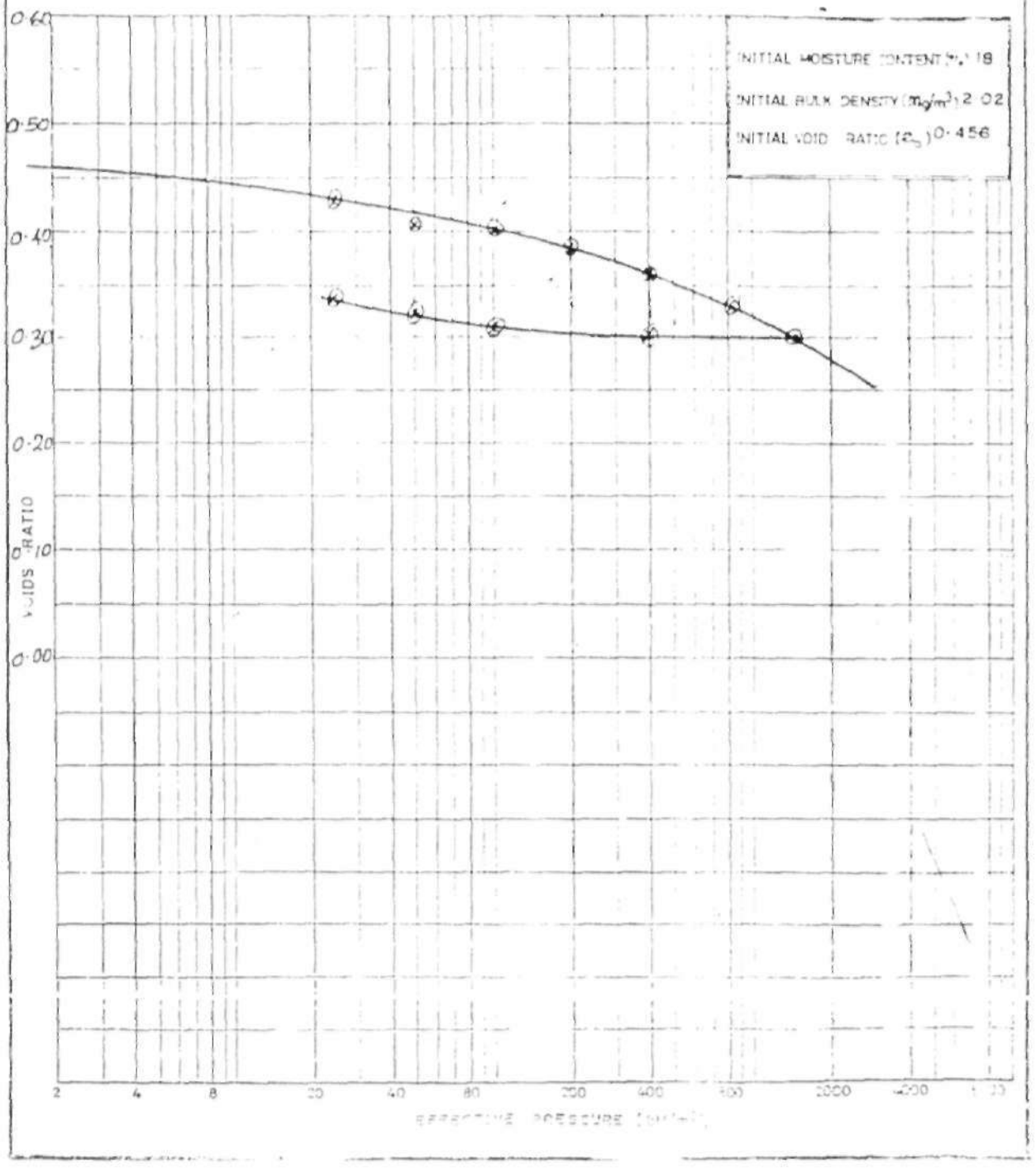
CONSOLIDATION TEST

PROJECT M Sc/ENG/0442/93 DATE MAR 1997

LOCATION OGOLO - AMA

BOREHOLE No. 1 SAMPLE No. 5 DEPTH (m) 4.50 - 4.95 m

e — log p curve



APPENDICE F_1 & F_2

CALCULATION OF FOUNDATION SETTLEMENTS

APPENDIX F1

CALCULATION OF CONSOLIDATION SETTLEMENT

BORIKIRI SANDFILL

The soil profile and the foundation pressure are given in Figure 1 below.

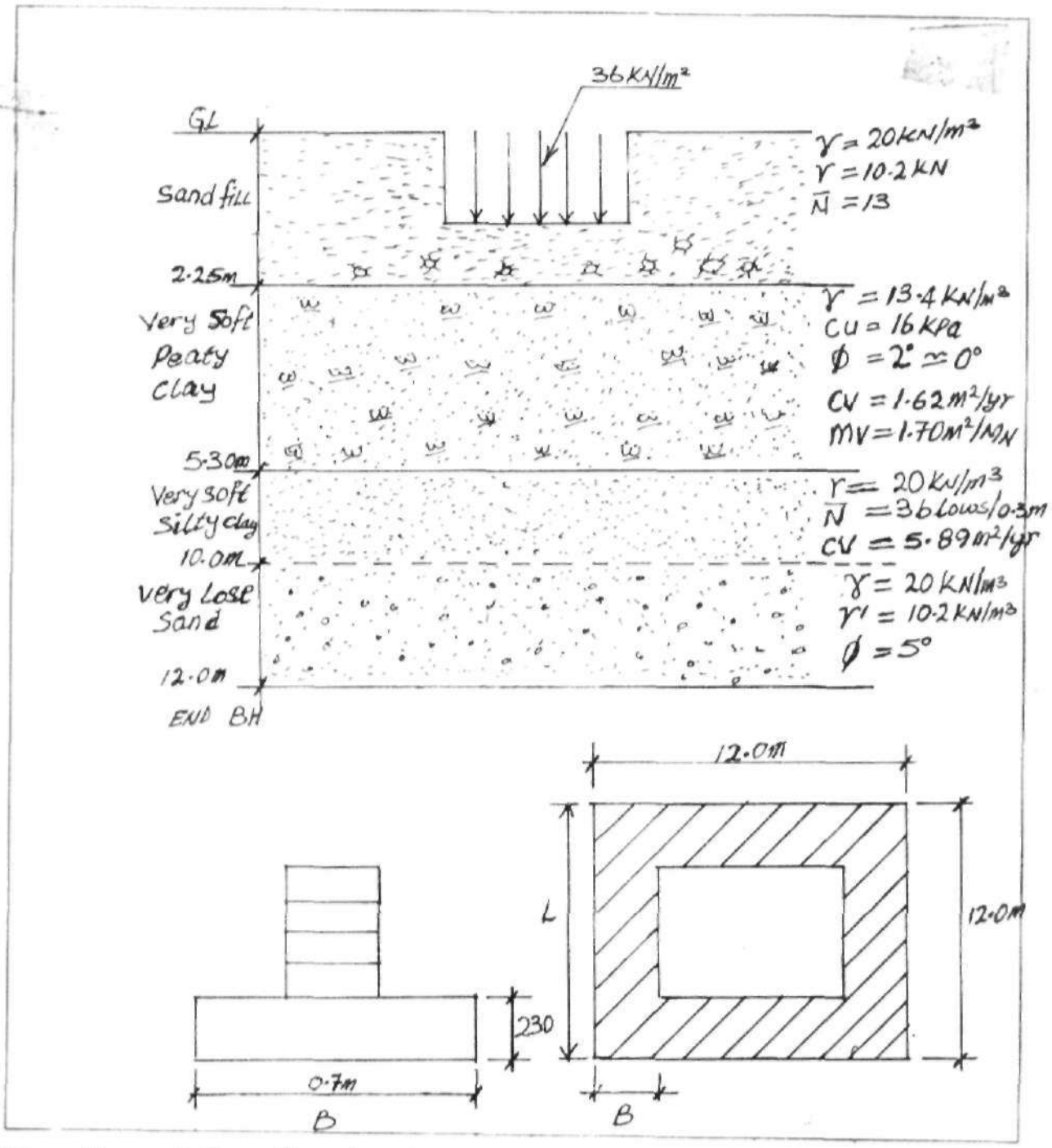


Figure F1: Soil profile taken from the two boreholes for Settlement analysis (Borikiri)

1.0 Determination of Mv:

ZONE 1 GL - 1.50m

$$E = 0.478 \times N + 7.17 \text{ Mpa}$$
$$= 0.478 \times 13 + 7.17 \text{ Mpa}$$
$$= 13.38 \text{ Mpa}$$

2 - Sin Ø

$$V = \frac{1 - \text{Sin } 31}{2 - \text{Sin } 31} = \frac{1 - 0.5150}{2 - 0.5150}$$

$$= 0.485/1.485 = 0.33$$

$$V = 0.33$$

From the equation $Mv = 1/E = (1/E)(1 - 2v^2/2 - v)$

$$= (1/13.38)\,(1 - (2(0.33)^2/2\text{-}033))$$

$$= (1/13.38)(1\text{-}1304)$$

$$Mv = 0.065m2/MN$$

ZONE 2: 2.25 - 5.50m

From the consolidation test result., average at the pressure of 25KN/m2 for the two boreholes.

$$Mv = 1.70\ m^2/MN$$

ZONE 3: 5: 25.- 9.75m

From the consolidation test results, at the pressure of 25 KN/m2 for the two boreholes.

$$Mv = 0.82m^2/MN$$

SETTLEMENT CALCULATION BORIKIRI

SETTLEMENT AT THE CENTRE

ZONE NO	Depth of layer Below Ground Surface z (m)	Foundation Width B (m)	Z/B	L/B	Foundation load P MN/m2	Mv m^2/MN	I_2	I_1	$I_2 - I_1$	$S = Mv.B.P(I_2-I_1)(mm)$
1	0.50	0.7	0.71	17.1	0.036	0.065	0.065	0	0.65	1.07
2	5.750	0.7	8.2	17.1	0.036	3.39	2.01	0.65	1.36	116

ΣS = 117 mm

SETTLEMENT CALCULATION BORIKIRI

SETTLEMENT AT THE CENTRE

ZONE NO	Depth of layer Below Foundation Surface Z (m)	Foundation Width B (m)	Z/B	L/B	Foundation load P MN/m^2	M_v m^2/m_v	I_2	I_1	$I_2 - I_1$	$S = m_v.B.P\ (I_2-I_1)$ (mm)
1	0.5	0.7	0.71	17	0.036	0.065	0.2	0	0.2	0.33
2	5.75	0.7	8.2	17	0.036	3.39	0.83	0.2	0.62	53

ΣS = 53.3 mm

SETTLEMENT CORRECTION

$S_{field} = \mu \times S$

It is obtained from Table 4.2 values of consolidation factor M for various soil types, hence for the soil condition under consideration. $\mu = 1.1$

For centre of foundation, $S_{field} = 1.1 \times 117mm = 129m$

For corner of foundation, $S_{field} = 1.1 \times 53.3mm = 59mm$

APPENDIX F_2

OGOLO - AMA SANFILLED AREA.

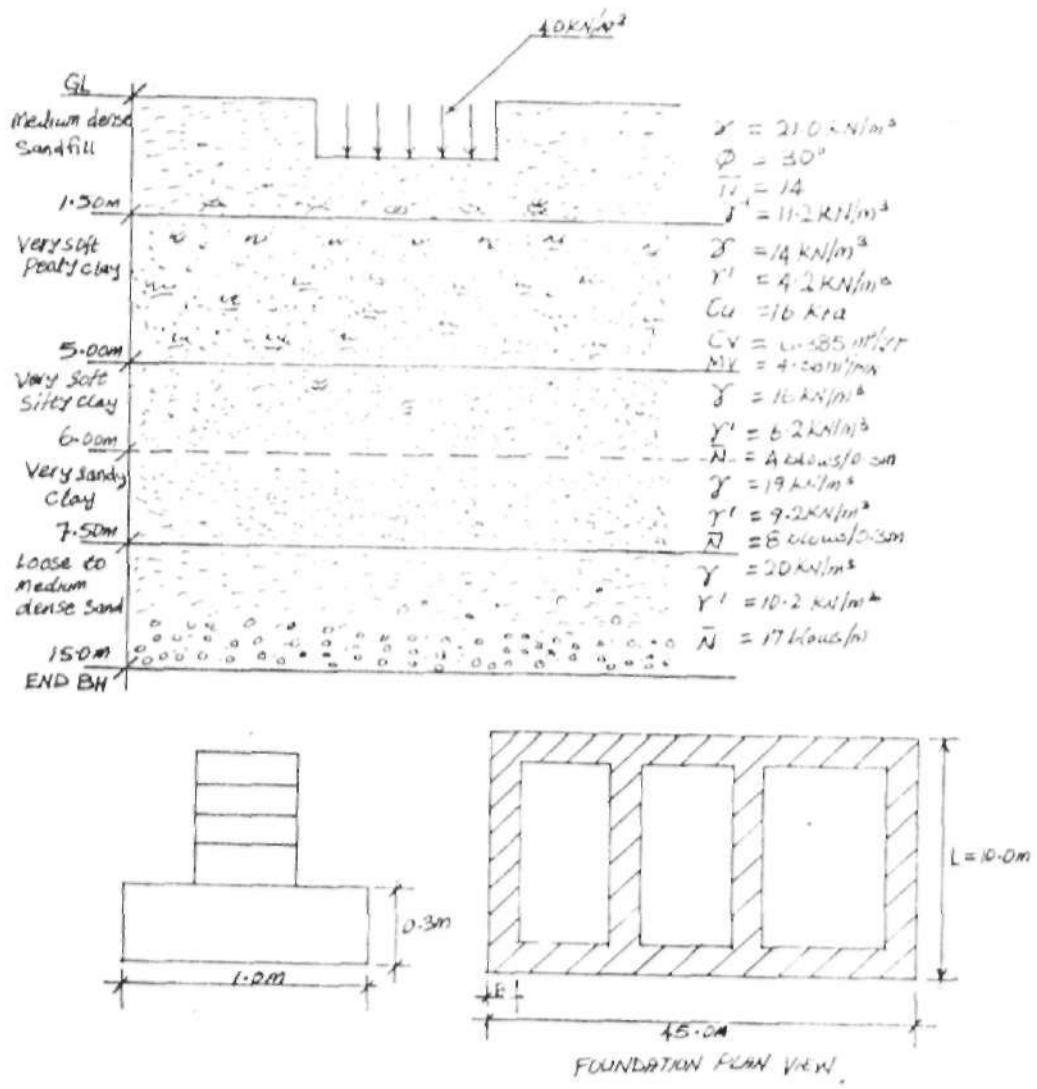


Figure F_2: Soil profile taken from the two boreholes for Settlement analysis (Ogolo-Ama)

1.0 Determination of m_v

ZONE 1: GI - 1.50M

$$E = 0.478 \times N + 7.17 \text{ MPa}$$
$$= 0.478 \times 14 + 7.17 \text{ MPa}$$
$$= 6.692 + 7.17$$
$$= 13.86$$

$$V = \frac{1 - \sin \emptyset}{2 - \sin \emptyset} = \frac{1 - \sin 32}{2 - \sin 32} = \frac{1 - 0.5299}{2 - 0.5299}$$

$$= 0.4701/1.4701$$
$$V = 0.3198$$

From the equation $m_v = 1/E = (1/E)(1 - (2v^2/2 - v))$

$$= (1/13.86)(1 - (2(0.32)^2/2-0.32))$$

$$= (1/13.86)(1 - 0.2048/1.68)$$

$$m_v = 0.0634 m2/MN$$

ZONE 2: 1.50 - 5.00M

From the consolidation test results average at the pressure of 50 Kn/m2 for the two boreholes

$$Mv = 4.376 \text{ m}^2 / \text{MN}$$

ZONE 3: 5.00 - 7.50m

From the consolidation test results, average at the pressure of 100 kN/m^2 for the two boreholes:

$$Mv = 0.196 \text{ m}^2 /MN$$

SETTLEMENT CALCULATION

SETTLEMENT AT THE CENTRE

ZONE NO	Depth of layer Below Ground Surface Z (m)	Foundation Width B (m)	Z/B	L/B	Foundation load P MN/m^2	Mv m^2/MN	I_2	I_1	$I_2 - I_1$	$S = Mv.B.P\ (I_2-I_1)(mm)$
1	0.5	1.0	0.50	6	0.040	0.063	0.5	0	0.50	1.26
2	4.50	1.0	4.50	6	0.040	4.376	1.73	0.50	1.23	215
3	5.50	1	5.50	6	0.040	0.196	1.83	1.73	0.12	0.940
										$\Sigma S = 217mm$

SETTLEMENT CALCULATION

SETTLEMENT AT THE CENTRE

ZONE NO	Depth of layer Below Ground Surface Z (m)	Foundation Width B (m)	Z/B	L/B	Foundation load P MN/m^2	Mv M^2/MN	I_2	I_1	$I_2 - I_1$	$S = m_v.B.P\ (I_2-I_1)(mm)$
1	0.5	1.0	0.50	6	0.040	0.063	0.24	0	0.24	0.605
2	4.50	1.0	4.50	6	0.040	4.376	0.65	0.24	0.41	72
3	5.50	1	5.50	6	0.040	0.196	0.71	0.65	0.06	0.470
										$\Sigma S = 73mm$

SETTLEMENT CORRECTION (OGOLO - AMA)

$S_{field} = \mu \times S$

M is obtained from Table 4.2 values of consolidation factor M for various soil types. Hence for the soil condition under consolidation.

$\mu = 1.1$

For centre of foundation: $S_{field} = 1.1 \times 2.17mm = 239\ mm$

For corner of foundation: $S_{field} = 1.1 \times 73m = 80mm$

APPENDIX G

RATE OF CONSOLIDATION CALCULATIONS

Appendix G

Calculation of the Rate of settlement

In calculating the rate of consolidation settlement, the subsoil profile used for the settlement analysis was utilized. The rate of settlement was estimated for the underlying zones of weak compressible clavey soils.

(1) BORIKIRI SANDFILL

Zone 2: 2.00-5.25 m Very soft,organic peaty clay with substantial decaying vegetation

Using the general formula: $t = \frac{Ty\ d^2}{Cv}$

From the soil profile given in Appendix F above

$H = 3.25$ m
$Cv = 0.32$

the fill material provides drainage at the top of the organic peaty clay, while the organic clays at the bottom of the layer permits little or no drainage, thus for drainage at the top only:

$d = 3.25$ m

At 50% settlement

$Tv = 0.197$

$t_{50} = \frac{0.197*(3.25)^2}{0.32}$ yrs

$= 6.5$ yrs

$t_{50} = 7$ yrs

At 90% settlement

$Tv = 0.848$

$t_{50} = \frac{0.848*(3.25)^2}{0.32}$ yrs

$= 27.99$ yrs

$t_{90} \approx 28$ yrs

Zone 3: 9.75-5.25 m Very soft silty organic clay becoming very sandy with depth

Using the general formula: $t = \frac{Tv\ d^2}{Cv}$

From the soil profile given in Appendix 1.1 above

$$H = 4.5\ m$$

the organic clay on the top of this layer permits little or no drainage, while the alluvial sand at the bottom of the layer provides drainage channel, thus, for drainage at the bottom only:

$$d = 4.5\ m$$
$$Cv = 2.14\ m^2/yr$$

At 50% settlement

$$Tv = 0.197 \text{ (table 4.4)}$$

$$t_{50} = \frac{0.197*(4.5)^2}{2.14}\ yrs$$
$$= 1.86\ yrs$$

t_{50} = 2 yrs (Say)

At 90% settlement

$$Tv = 0.848$$

$$t_{50} = \frac{0.848*(4.5)^2}{2.14}\ yrs$$
$$= 8.02\ yrs$$

$t_{90} \approx$ 8 yrs

APPENDIX G2

(2) OGOLO - AMA SANDFILL

The method of calculating the rate of settlement of the rate of settlement of the distressed building was done in a similar way as presented above for Borikiri sandfill:

ZONE 2: 1.50 - 5.00M - Very soft silty organic peaty clay with substantial decomposed plant debris.

Using the general formulae: $t = \frac{T_v\, d^2}{C_v}$

From the soil profile given in Appendix F2, above.

$H = 3.5m$

$C_v = 6.385\ m^2 / yr$

The file material provides drainage at the top of the organic peaty clays at the bottom of the layer permits negligible drainage channel, thus for drainage at top only:

$d = 3.5\ m$

At 50% settlement

$T_v = 0.197$

$t_{50} = \frac{0.197*(3.5)^2}{6.385}$ yrs

$= 0.38$ yrs

t_{50} = 0.4 yrs or 5 months

At 90% settlement

$T_v = 0.848$

$t_{50} = \frac{0.848*(3.5)^2}{0.215}$ yrs

$= 1,63$ yrs

t_{90} = 2 yrs

Zone 3: 7.50-5.00 m Very soft very silty organic clay becoming sandy with depth

Using the general formula: $t = \frac{Tv\ d^2}{Cv}$

From the soil profile given in Appendix F2 above

$$H = 2.5\ m$$

the alluvial sand at the bottom of the layer provides drainage channels, thus for drainage at the bottom only:

$$d = 2.5\ m$$
$$Cv = 7.05\ m^2/yr$$

At 50% settlement

$$Tv = 0.197 \text{ (table 4.4, Case 1)}$$

$$t_{50} = \frac{0.197*(2.5)^2}{7.05}\ yrs$$
$$= 0.175\ yr$$

$$\mathbf{t_{50} = 0.2\ yr\ or\ 2.4\ months}$$

At 90% settlement

$$Tv = 0.848$$

$$t_{50} = \frac{0.848*(2.5)^2}{7.05}\ yrs$$
$$= 0.75\ yr$$

$$\mathbf{t_{90} = 0.8\ yrs\ or\ 1\ yr}$$

APPENDIX H

PRELOADING DESIGN

APPENDIX H

DESIGN CALCULATIONS FOR RELOADING FILL THICKNESS

1.0 LOCATION: Reclaimed marginal land at Borikiri, Port Harcourt.

2.0 SOIL PROFILE: Soil profile taken from the two boreholes for the settlement analysis.

GL. - 2.25m	Medium dense sandfill
2.25 - 5.50m	Very soft clay (Chikoko mud)
5.50 - 10.00m	Very soft clay (Marine mud)
10.00 - 12.00m	Very loose sand.

Zone 1 — Surcharge Sandfill:
Assumed bulk unit weight $\gamma = 19kN/m^3$

Zone 2 — Existing Sand fill:
Bulk unit weight $\gamma = 20kN/m^3$
Effective unit weight $\gamma' = 10.2kN/m^3$

Zone 3 — Organic Clay (Chikoko mud):
Average moisture content m = $20kN/m^3$
Saturated unit weight $\gamma = 13.4kN/m^3$
Effective unit weight $\gamma' = 3.6kN/m^3$
Unarained shear strength Cu = 16kPa
Initial Void ratio $e_o = 3.609$
Coefficient of consolidation $C_v = 0.32m^2/yr$
Coefficient of Volume Compressibility $M_v = 1.70m^2/MN$
Compression Index = 0.57
Preconsolidation Pressure = 90kpa

Zone 4 — Silty Clay (Marine mud)
Saturated unit weight $\gamma = 20\ kN/m^3$
Effective unit weight $\gamma = 10.2kN/m^3$
SPT penetration resistance N = 3 blows/0.3m
Initial Void ratio $e_o = 1.528$
Coefficient of consolidation $C_v = 2.14$
Coefficient of Volume Compressibility $M_v = 1.59m^2/MN$

Zone 5 — Very Loose Sand
Saturated unit weight $\gamma = 20kN/m^3$
Effective unit weight $\gamma = 10.2kN/m^3$
SPT penetration resistance N = 3 blows/0.3m

Ground water level was at a depth of about 0.7m below below the ground surface.

3.0 SETTLEMENT CALCULATIONS

Assumptions

i. Settlements due to the load from the sand placed for reclamation of the land is complete.
ii. Load from the structures erected on the site is 36kPa.

Calculation 90% consolidation settlement induced by the imposed load is 117mm.

This settlement will take place in 27.9 years (say 28years). It is required that surcharges are placed on the project site to reduce the time of 100% settlement to 3 years or 6 years or 10years or 12 years.

4.0 Determination of Thickness of Surcharges

From calculations 90% of the settlement will take place in about 28 years.
100% of this settlement will take place in about $\frac{2 \times 28}{0.848}$ years

$= 66.03$
$= 66$years (say)

4.1 Required Thickness of sand to reduce the time for 100% settlement to 3 year, 6 years, 10 years, 12 years.

$$Tv = \frac{C_v.t}{h^2}$$

Where $C_v = 0.32m^2/yr$
$t = 3$
$h = 3.25$ for single drainage path

$$Tv = \frac{0.32 \times 3}{(3.25)^2} = 0.091$$

For $Tv = 0.091$
Degree of consolidation $U = 0.385$
Using the equation: $S = \frac{H}{1+e_0} C_c \log_{10} \left(\frac{P_0 + \Delta p}{P_c}\right)$

Where S = Settlement due to consolidation
H = Thickness of compressible layer
C_c = Compression Index
e_0 = Initial void ratio
P_0 = Present effective overburden pressure at midpoint of the consolidating layer.

Δp = Vertical increase at the centre of consolidating layer due to the surcharge

P_c = Preconsolidation pressure

For Tv = 0.091, U = 0.385.
Settlement 117mm correspond to 0.385 of the settlement that would be obtained under a heavier surcharge that would produce a 100% settlement of $= \frac{117mm}{0.385}$

$= 303mm$

$$303 = \frac{3.25 \times 1000}{1+3.609} \times 0.57 \log_{10} \left(\frac{P_0' + \Delta p'}{P_c'}\right)$$

$$= \frac{3250}{4609} \times 0.57 \log_{10} \left(\frac{P_0 + \Delta p}{P_c}\right)$$

$$\text{Let } \left(\frac{P_0 + \Delta p'}{P_c'}\right) = X$$

$$\therefore \log_{10} X = \frac{303 \times 4.609}{3250 \times 0.57} = 0.754$$

$X = 5.68$

$$P_0 = 0.7 \times 18 + (9\text{-}9.807)X(2.25\text{-}0.7) + (13.4\text{-}9.807) \times \frac{3.25}{2}$$

$= 12.6 + (8.2 \times 1.55) + (3.6 c 1.625)$
$= 12.6 + 12.7 + 5.9$
$= 31.2 kN/m^2$

From e_0 - logP curves, $P_c = 90 kN/m^2$

$$\therefore \frac{31.2 + \Delta p'}{90} = 4.65$$

$\Delta p = 5.68 \times 90\text{-}31.2$
$= 511.2 - 31.2$
$= 480 kN/m^2$

Thickness of surcharge: $= 480/19 kN/m^3$
$= 25.3m$
$= 25m(Say)$

Thickness of sand surcharge to reduce the completion o 100% settlement to 3years = 25m.

4.2 Time of surcharge for 6 years completion

$$Tv = \frac{0.32 \times 6}{(3.35)^2} = 0.181$$

Tv = 0.181, U = 0.459,

$$\therefore 100\% \text{ Settlement} = \frac{117}{0.459}\text{mm} = 255\text{mm}$$

$$= 255\text{mm}$$

$$188 = \frac{3.25 \times 1000}{1+3.609} \times 0.57 \log_{10} \left(\frac{p_o' + \Delta p'}{p'_c}\right)$$

$$\therefore \log_{10} \quad X = \frac{255 \times 4.609}{325 \times 0.57}$$

$$= 0.634$$

$$X = 4.31$$

$$\therefore \frac{31.2 + \Delta p'}{90} = 2.94$$

$$\Delta p' = 4.31 \times 90 - 31.2$$

$$= 387.9 - 31.2$$

$$= 356.7 kN/m^2$$

Thickness of surcharge: $= \frac{356.7m}{19}$

$$= 18.8m$$

Thickness of surcharge = 19 (say)

Thickness of surcharge to cause 100% of the settlement to take place in 6 year is 19m

4.3 Time of Surcharge for 10 years completion

$$Tv = \frac{0.32 \times 10}{(3.25)^2}$$

Tv = 0.30, U = 0.627 = 0.30

$$\therefore 100\% \text{ Settlement} = \frac{117\text{ mm}}{0.627}$$

$$= 187$$

$$159 = \frac{3.25 \times 1000}{1+3.609} \times 0.57 \log 10 \left(\frac{p_o' + \Delta p'}{p'_c}\right)$$

$$\therefore \log_{10} X = \frac{187 \times 4.609}{3250 \times 0.57}$$

$$= 0.465$$

$$X = 2.93$$

$$\therefore \frac{31.2 + \Delta p}{90} = 2.49$$

$$\Delta p = 2.93 \times 90 - 31.2$$

$$= 263.7 - 31.2$$

$$= 232.5 kN/m^2$$

Thickness of surcharge: $= \frac{232.5m}{19}$

$= 12m$

4.4 Time for 100% completion for 15 years

$$Tv = \frac{0.32 \times 15}{(3.25)^2} = 0.454$$

$Tv = 0.454$, $U = 0.788$

$\therefore$ 100% Settlement $= \frac{177}{0.788}$ mm

$= 149mm$

$$133 = \frac{3.25 \times 1000}{1 + 3.609} \times 0.57 \log_{10} \frac{(p_o' + \Delta p')}{(p_o')}$$

$$\therefore \log_{10} X = \frac{149 \times 4.609}{3250 \times 0.57}$$

$$= 0.371$$

$$X = 2.350$$

$$\therefore \frac{31.2 + \Delta p}{90} = 2.35$$

$$\Delta p = 2.35 \times 90 - 31.2$$

$$= 211.5 - 31.2$$

$$= 180.3 kN/m^2$$

Thickness of surcharge:

$$= \frac{180.3m}{19}$$

$= 9.5m$

SUMMARY

Thickness of surcharges required for 100% completion of settlement at various times.

Consolidation Settlement = 117mm
Time for 100% Settlement = 66years

Time (yr)	Thickness of surcharge (mm)
3	25
6	19
10	12
15	9.5