GEOTECHNICAL AND GEOCHEMICAL PROPERTIES OF LATERITIC SOIL ON MIGMATITE ALONG OGBOMOSHO - ILORIN HIGHWAY SOUTH-WESTERN NIGERIA

BY

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ABSTRACT

The geotechnical and geochemical properties of lateritic soil developed on migmatite along Ogbomosho – Ilorin road, south-western Nigeria was studied. Soil samples were collected from seven (7) different laterite profiles along the road cuts and analyzed in the laboratory so as to determine the geotechnical, geochemical and petro-graphical characteristics of each horizon within the profile and their suitability as construction and/or foundation material. The geotechnical tests conducted according to the British standard procedures (1377) include: index test and performance test. Field geological mapping shows that the parent rock from which the in – situ lateritic soil was derived is a migmatite with an average soil profile depth of 4.2m. The principal mineral compositions of the migmatite are quartz (40%), biotite (25%), feldspar (21%), hornblende (6%), and muscovite (5%). The moisture content obtained range between 3.1 and 14.2%,Specific gravity value range between 2.50 and 2.74, bulk density and dry density range between 1.44 and 1.77g/cm^3 and 1.31 and1.62g/cm^3 with an average value of 1.56 and 1.44g/cm^3 respectively which indicate that the soil was produced through in - situ weathering of granitic rocks rich in felsic mineral. The results of the Atterberg consistency limit range between 3.3 and 14% (PL), 40 and 58% (LL), 17.7 and 35% (PI) with an average value of 47.2%, 49% and 8.1% respectively. Similarly plots on the plasticity chart shows that the soil falls above A- line corresponding to CL zone which is classified as inorganic clay of moderate plasticity based on Unified Soil Classification System (USCS). The grain size above 0.075 mm revealed fine to coarse grained sand.The OMC and MDD have an average value of 13.08% and 1.76kg/m^3 and the CBR value for soaked and unsoaked ranging between 17 and 66% and 42 – 74% with an average of 57.58% and 34.8% which indicate that the soil can be used for sub - base and subgrade in pavement construction of roads. The value obtained from angle friction (ϕ) and cohesion (C) range between 22 and 32° and 10 and 30KN/m^2 with an average value of 27.04 and 21.5KN/m^2 which indicate that the soils can be used as material in the construction of fill road embankment. The x-ray fluorescence analysis shows that the major oxides include SiO_2, Fe_2O_3, Al_2O_3, K_2O, Na_2O, TiO_2, and MgO while SiO_2, Fe_2O_3 and Al_2O_3 constitute 70–80% of the oxides. The research reveals that with the exception of Horizon A and C of the profile the geotechnical and geochemical characteristics of the lateritic soil derived from migmatite conform to Nigerian Federal Ministry of Works guidelines on road construction materials and compare favourably with the results of other authors. Sectional failures might have resulted from inappropriate material handling and/or equipment inadequacies.

TABLE OF CONTENTS

Title page		i
Declaration		ii
Certification		iii
Acknowledgements		iv
Abstract		v
Table of contents		vi
List of Tables		x
List of Figures		xi
List of Plates		xii
List of Appendices		xiii
	CHAPTER ONE	
1.0	**INTRODUCTION**	1
1.1	Background to the Study	1
1.2	Statement of the Problem	2
1.3	Justification	3
1.4	Scope of work and Limitations	3
1.5	Description of the Study Area	3
1.5.1	Location, Extent and Accessibility	4
1.5.2	Topography and Drainage	4
1.5.3	Climate and Vegetation	6
1.5.4	Geological Setting of Study Area	6
1.6	Aims and Objectives	8

CHAPTER TWO

2.0 **LITERATURE REVIEW** 9

2.1 Review of the Geology of Nigeria 9

2.2 Regional Geology 10

2.2.1 Basement Complex 10

2.2.1.1 Migmatite Gneiss Complex 12

2.2.1.2 Schist Belt (Metasedimentary and Metavolcanic rock) 13

2.2.1.3 Older Granite (Pan African Granitoids) 14

2.2.1.4 Undeformed Acid and Basic Dyke 17

2.3 Previous work 18

CHAPTER THREE

3.0 **RESEARCH METHODOLOGY** 30

3.1 Fieldwork 30

3.1.1 Sampling 30

3.1.2 Analysis 33

3.2 Laboratory Procedure 33

3.2.1 Index Test 33

3.2.1.1 Natural Moisture Content 33

3.2.1.2 Bulk Density 34

3.2.1.3 Specific Gravity 35

3.2.1.4 Grain Size Analysis 35

3.2.1.5 Atterberg Consistency Limit 37

3.2.2 Performance Test 40

3.2.2.1 Compaction Test 40

3.2.2.2 California Bearing Ratio 42

3.2.2.3	Direct Shear Strength	43
3.2.2.4	Consolidation Test	45
3.3	Petrographic Analysis	46
3.4	X-ray florescence Analysis	47
	CHAPTER FOUR	
4.0	**RESULTS AND DISCUSSION**	48
4.1	Geology	48
4.1.1	Field mapping	48
4.1.1.1	Migmatite	50
4.1.1.2	Pegmatite	52
4.1.2	Petrographic studies	54
4.2	Soil Profile	56
4.3	Geochemical Studies	62
4.4	Geotechnical Studies	64
4.4.1	Index test	64
4.4.1.1	Specific Gravity	64
4.4.1.2	Grain Size Analysis	66
4.4.1.3	Atterberg Consistency Limit	66
4.4.2	Performance Test	68
4.4.2.1	Compaction Test	68
4.4.2.2	California Bearing Ratio	70
4.4.2.3	Shear Test	71
4.4.2.4	Consolidation Test	72
4.5	Influence of Parent rock on soil samples characteristics	72

CHAPTER FIVE

5.0 **CONCLUSION AND RECOMMENDATIONS** 74

5.1 Conclusion 74

5.2 Recommendations 75

REFERENCES 76

LIST OF TABLES

Table		Page
3.1	Sample localities and horizons soil profile	31
3.2	Differences in the Proctor Methods Compaction	41
4.1	Percentage Oxide Analysis	63
4.2	Summaries of Laboratory tests	65
4.3	Particle size distribution	67
4.4	General rating for CBR Asphalt Institute	71
4.5	Representative value of ϕ for sand and silt after USCS	72
4.6	Influence of parent rock on position of the engineering characteristics	73

LIST OF FIGURES

Figure		Page
1.1	Location map of study Area	5
1.2	Drainage map of Ogbomosho - Ilorin highway samples	7
2.1	Simplified Geological Map of Nigeria	11
4.1	Geological Map of Study Area	49
4.2	Typical Soil Profile along the Road	57
4.3	Plots of Plasticity Chart	69

LIST OF PLATES

Plate		Page
I	Migmatite outcrops in the study area	49
II	Pegmatite intruding the migmatite	51
III	Mineral assemblage in thin section of the Migmatite	53
IV	Cut section on the highway	55

LIST OF APPENDICES

Appendix		Page
A	Particle size distribution curve grain size analysis	84
B	Compaction curves for standard proctor	89
C	California Bearing Ratio (CBR) Curves	93
D	Plot of Shear Stress versus Normal Stress	101
E	Consolidation curve	105
F	Natural Moisture Content	111
G	Bulk Density	113
H	Specific Gravity	117
I	Atterberg Consistency Curve	119

CHAPTER ONE

1.0 INTRODUCTION

1.1 Background to the Study

Lateritic soils are soils that are composed almost entirely of iron and aluminium oxides; they are usually reddish in colour and are the least soluble product of rock weathering in tropical climates (Plummer, McGeary and Carlson, 2001). They are formed in regions of high temperature and abundant rainfall, where the soils are highly leached. Under such conditions, weathering is deep and intense. Tropical weathering (laterization) is a prolonged process which produces a wide variety in the thickness, grade, chemistry and mineralogy of soils. The leaching of rocks by percolating rain water during wet season results to solution containing the leached ions which form soluble salt compounds by capillary action, during the dry season these salts are brought to the surface and wash away during the following wet season.

Gidigasu (1972) divided the stages of laterite formation into three, namely:

(a) Stage of decomposition:- This involves the physical and chemical breakdown of rock forming mineral to yield simple ionic compounds.

(b) Stage of laterization:- This involves the leaching away of the soluble bases such as the oxides of sodium and calcium, leaving an enrichment of insoluble bases such as the oxides and hydroxides of magnesium, titanium, iron and aluminium. The process of leaching involves acid dissolving the host mineral lattice, hydrolysis and precipitation of insoluble oxides and sulphates of aluminium, iron, and silica under intense temperature condition of humid sub-tropical climate.

(c) Stage of desiccation:- This involves dehydration of insoluble bases.

Lateritic soils are very important in the construction industries and activities as construction material and foundation support for engineering structures. Although studies of the engineering properties of lateritic profile began some decades ago, but there is still paucity of geotechnical and geochemical data. This research tends to focus on the influence of position of horizon within the soil profile and the resultant geotechnical and geochemical properties of soils developed over migmatite and also examine their suitability as construction and /or foundation material.

1.2 Statement of the Problem

Ogbomosho – Ilorin New road was commissioned in 2009 but already showing some signs of failure in some sections. The non durability and cracks on Ogbomosho – Ilorin road are due to the misuse of construction material among other factors. Engineering structures are found in all construction works involving the use of lateritic soil however, not all lateritic soil can be assumed to be suitable for use as construction materials. Their classification test must be performed to ascertain their nature before usage. In Nigeria soils excavated almost everywhere is often used directly for various construction purposes without thorough investigation of their geotechnical properties to ascertain their suitability which give rise to failure in roads. It is therefore reasonable to ensure that the structures are in top conditions at all times. One of the most important ways of attaining this is by ensuring that the soils used in the construction meet design standards and specifications. The geotechnical properties of soil that do not meet design standard and specification can be improved. The commonly used methods are index test and performance test. For this research, index and performance tests are adopted.

1.3 Justification

Road failure is prevalent in Nigeria and the trend is a cause of concern to both the users and the road maintenance authorities as they have economic disadvantages. In this regard, it is necessary that research be initiated to determine the possible causes of such road failures; and a case study was carried out on Ogbomosho – Ilorin Highway, South-western Nigeria.

1.4 Scope of Work and Limitations

The scope of the research constitutes desk work on the available secondary data collection. Geological mapping of highway cut slope at Eyenkorin, Lasoju, Ote, Gambari, Ladoke Akintola University of Technology (LAUTECH) Junction, field studies and sampling of soil and rock. Geotechnical and geochemical laboratory test/analysis was carried out on disturbed soil samples. Thin sectioning of rock samples was done. Inability to sample several of the exposed soil profiles along the study area because of cost implication was a major limitation to this research.

1.5 Description of the Study Area

The study area is located along Ogbomosho-Ilorin road with Ogbomosho in Oyo state to the south and Ilorin in Kwara State to the north with emphasis on places like Lasoju, Ote, Gbede, Abduka, Gambari, Egbeda, Akanbi and LAUTECH junction (Figure 1.1). The lateritic soils found in this area develop on many rock types in different sub-climate and drainage environment.

1.5.1 Location, Extent and Accessibility

The area is situated in the transitional zone between the forest and savannah region of south-western Nigeria. The Ogbomosho – Ilorin new road is situated on latitude 8˚ 26′ N 4˚ 24′ E and 8˚ 9′ N and 4˚ 37′ E. The road trends on NNE – SSW with Ilorin in the north and Ogbomosho to the south.

1.5.2 Topography and Drainage

The topography of the study area is slightly undulating with rounded low hills, occasional often elongated ridges indicating the characteristics residue setting of a typical basement terrain with an average height ranging between 180 and 360m above sea level. The area is drained mainly by river Asa, with Elekunkun, Awe, Idandan, Oshin, Moshi, and Ero being some of the tributaries forming a dendritic drainage pattern and generally flowing northward to the River Niger.

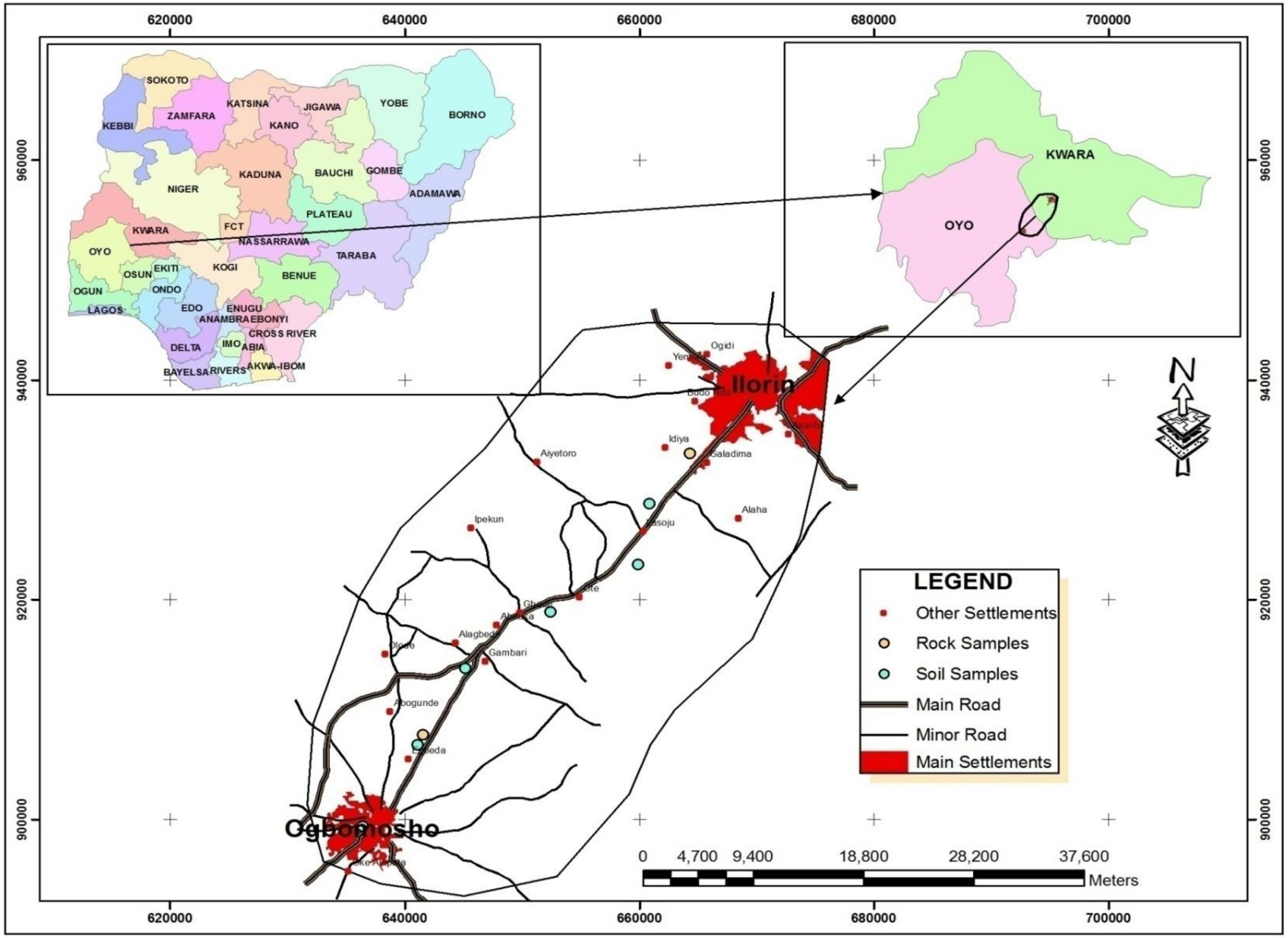


Figure: 1.1 Location map of study area (Modified by Dindey, 2014)

The dendritic drainage type is due to the loose nature of the top soil whereby water flows in all directions (Fig 1.2).The channels of these smaller streams are dry for many months especially from November to May.

1.5.3 Climate and Vegetation

The climatic condition is humid tropic, characterized by alternating wet and dry season with total annual rainfall of 1000 to1200mm which is good for the formation of laterite Alao (1983). The wet season occurs between April and October and dry season from November to March. The vegetation is guinea savannah and characterized by scanty trees and green grasses. However, due to the persistent human activities, the natural vegetation has been destroyed which give rise to the savannah type of vegetation (Ige, Ogunsanwo, and Inyang, 2011).

1.5.4 Geological Setting of Study Area

The study area is a typical Precambrian Basement Complex terrain with an elevation of about 394m above sea level. The rocks of this Basement include granite gneiss, migmatite, biotite gneiss, porphyroblastic gneiss, pegmatite and quartzite of which migmatite is the dominant rock type. The superfacial deposit within the Basement Complex terrain varies in thickness from 4m to 8m and are mostly clayey loamy topsoil and dark sandy soil, usually less than 2m thick followed by reddish brown laterite soil in most cases.

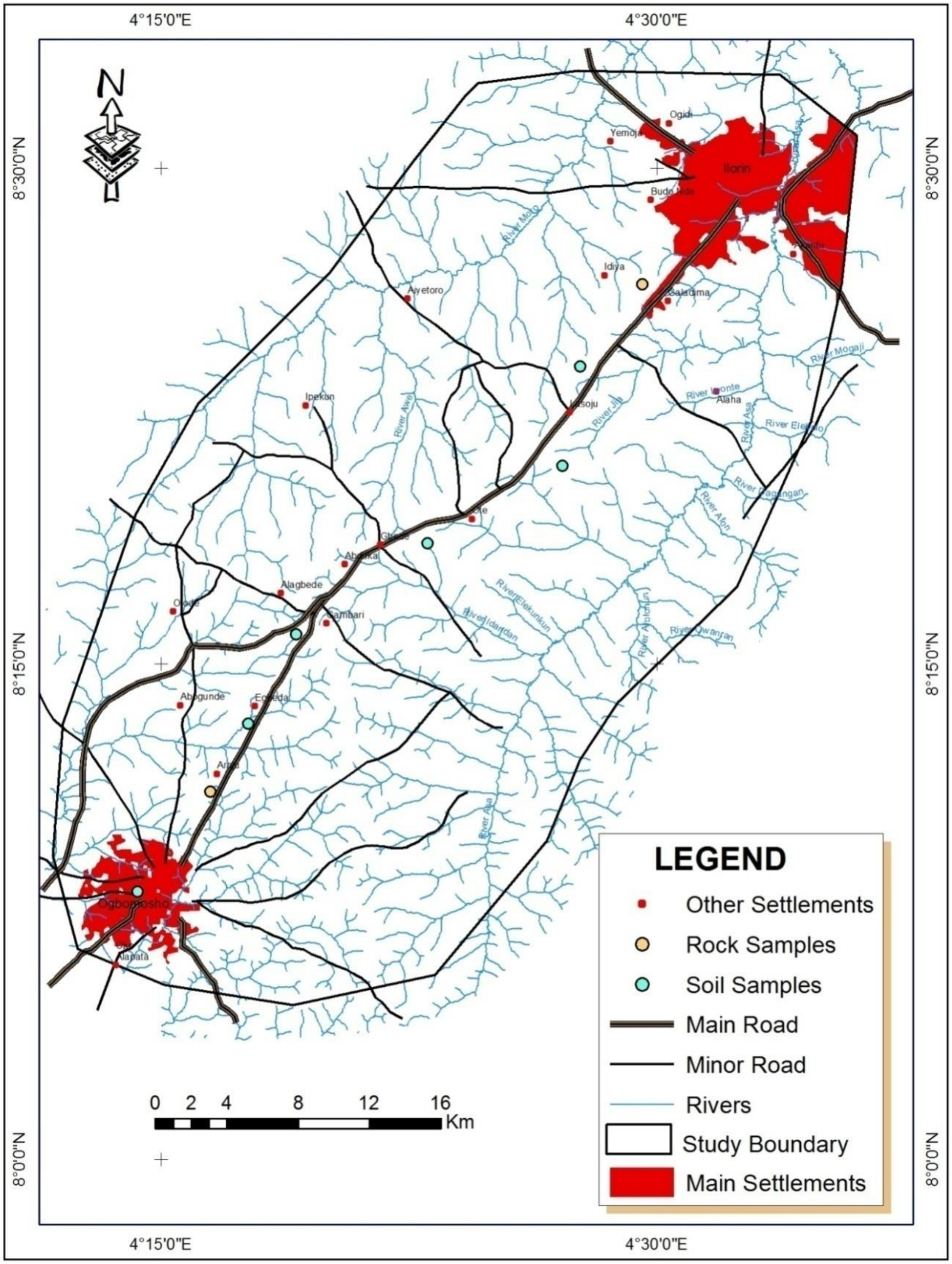


Figure: 1.2 Drainage map of Ogbomosho - Ilorin highway showing location of samples (Modified by Dindey, 2014)

1.6 Aim and Objectives

This research project is aimed at studying the influence of position of the lateritic horizons on the geotechnical and geochemical properties within the profiles developed over migmatite.

Objectives:-

The major objectives of the study were as follows

I. To study the geology of the study area.
II. To determine the geotechnical, geochemical and petrographical characteristics of each soil horizon within the profile.
III. To determine the suitability of each horizon as construction and/or foundation material.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 Review of the Geology of Nigeria

Nigeria lies between latitudes 4ºN and 15ºN and longitudes 3°E and 14°E within the Pan African mobile belt and Congo craton. These two cratons indicate the borders of mobile belts where Nigeria is bordered by the Gulf of Guinea to the south, Republic of Benin to the west, Niger Republic to the north, Chad and Cameroon to the east (Figure 2.1). The Nigerian Basement was assumed to be Precambrian in age (600Ma) which thus occupies the reactivated region that resulted into plate collision between the continental margins of the West African craton. (Burke and Dewey, 1972; Dada, 2006). The geology of Nigeria is dominated by crystalline and sedimentary rocks both occurring approximately in equal proportions. The crystalline rocks are made up of the Precambrian Basement and Phanerozoic rocks which occur in the eastern and north central part of Nigeria.

The Geology of Nigeria can be divided into three categories (Black, 1980), which are;

I. Sedimentary Basin

II. Younger Granite

III. Basement Complex

This classification is primarily based on the ages of the rocks and their modes of formation (McCurry, 1989). Some of the rock types are as a result of the earth's internal processes, for instance igneous rock are formed from the cooling and consolidation of magma or pre-existing rock undergoing certain processes which thus modify their physical, chemical and mineralogical properties. Sedimentary rocks are formed as a result of weathering, erosion, transportation and deposition of pre-existing rocks. The sedimentary basins covers about 50% of the total surface and are made up of Cretaceous to Recent sedimentary rocks. There are seven of these basins. Obaje (2009) and Malomo, Obadina and Adebo (1983) mentioned them to include: Calabar flank, Benue trough, Chad basin, Sokoto basin, Bida basin, Dahomey basin and Niger Delta basin.

2.2 Regional Geology

2.2.1 Basement Complex

The Basement Complex refers to group of rocks underlain by crystalline rocks. They are of Precambrian to lower Palaeozoic age in Nigeria (Figure 2.1). Isotopic age determination shows that at least four major orogenic cycles of deformation, metamorphism and remobilization occurred during the tectonic event corresponding to the Liberian (2700Ma), Eburnean (2000Ma), Kibarean (1,100Ma), and Pan African cycles (600Ma).The first three cycles were characterized by intense isoclinal folding and deformation accompanied by regional metamorphism with extensive migmatization. The deformations occur by a regional metamorphism, migmatisation, extensive granitization and gneissification which resulted into syntectonic granite and homogenous granodiorite with their associated contact metamorphism accompanying the end stages of this last deformation. The end of the Orogeny cycle was marked by fracturing and faulting (Gandu, Ojo and Ajakaiye, 1986; Olayinka, 1992).

The rocks of the Basement Complex of Nigeria can be conventionally divided into four major petro-logical units, namely:

I. Migmatite – Gneiss Complex (MGC)

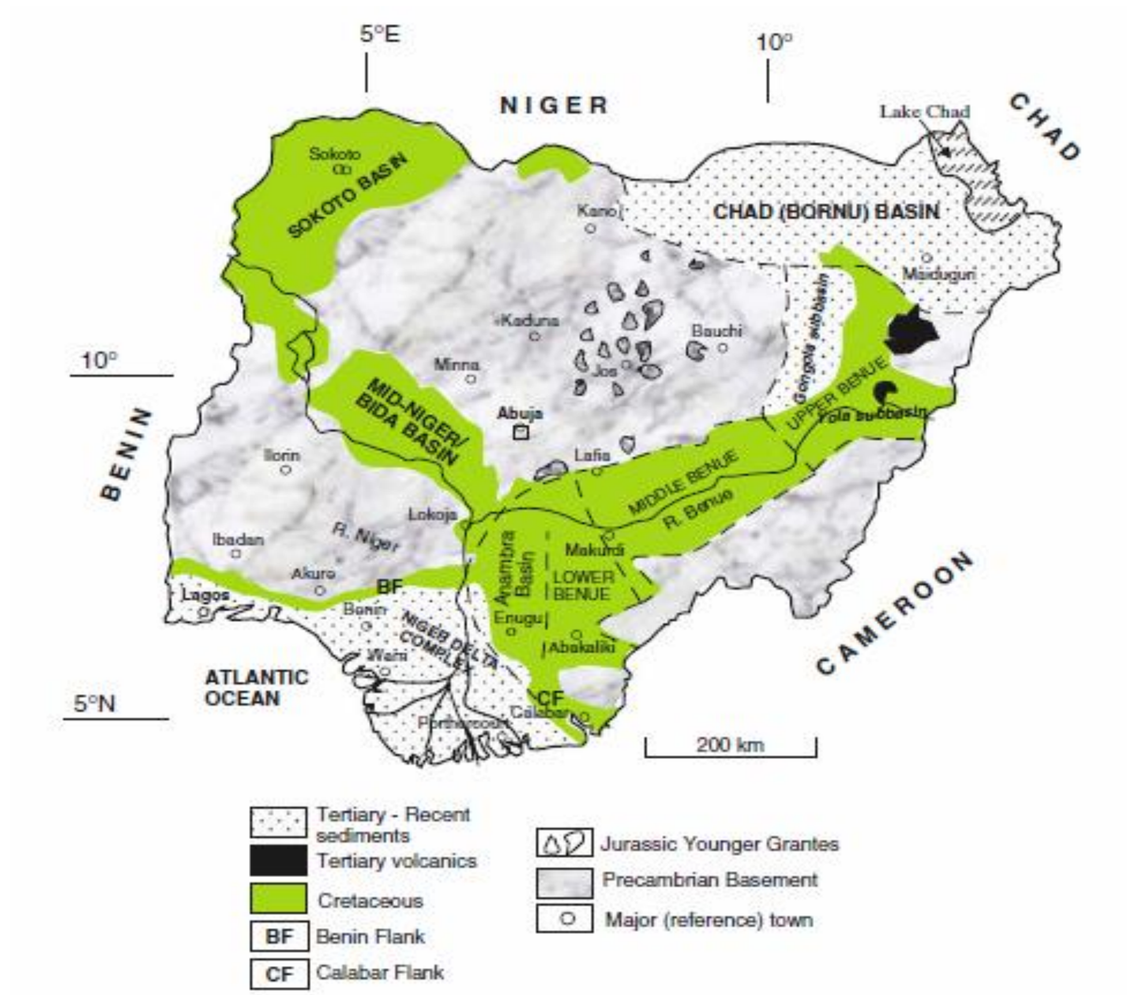


Figure 2.1: Simplified geological map of Nigeria (Obaje, 2009)

II. Schist Belt (Metasedimentary and Metavolcanic rocks)

III. Older Granites (Pan African Granitoids)

IV. Undeformed Acid and Basic Dykes

2.2.1.1 Migmatite – Gneiss Complex (MGC)

This complex occupies a significant area in the Nigeria Basement which resulted from complex association of folding, granitization, deformed shearing and migmatization process which has heterogeneous assemblage comprising migmatite, orthogneisses, paragneisses and series of metamorphosed rocks. Rahaman (1988) characterized three petrological units representing about 50% of the surface area of the Nigerian Basement. The petrological units are: grey foliated biotite-hornblende quartzo-feldspathic gneiss of tonalitic to granodioritic composition Rahaman (1981) alternating the mafic to ultramafic component which defines the bands on the outcrops. Petrographic evidence shows that the Pan African modification led to recrystallization of many constituent minerals of the migmatite - gneiss complex by partial melting with majority of the rock types displaying medium to upper amphibolite facies. The migmatite - gneiss complex has ages ranging from Pan African to Eburnean which is the oldest commonest rock types in the Nigerian basement comprises two main types of gneisses, the biotite gneiss and banded gneiss.

The banded gneiss shows alternating dark and light bands and also exhibits folding of their mineral bands. The biotite gneiss is a fine grained rock with foliation plane caused by parallel arrangement of light and dark minerals. These rocks record three major geological events; Liberian (2,700±200Ma) which involved beginning of crust forming process example the Ibadan banded gneiss. This was followed by Eburnean (2,000±200Ma) marked by the Ibadan granite gneiss with the imprint of the Pan African event which not only structurally overprinted and reset many geochronological clocks in the older rocks, but also gave rise to migmatite -gneiss, granite gneisses and other similar lithological units. Lithologically, similar rocks in other part of Nigeria including the western, northern, and eastern parts are covered by migmatite- gneiss complex which include Ibadan, Ilorin, Ile-Ife, Akure, Okenne, Egbe, Ajaokuta, Ikare, Abuja, Keffi, Akwanga, Kano, Kaduna, Bauchi, Funtua, Oban massif and Obudu areas in the eastern Nigeria.

2.2.1.2 Schist Belt (Metasedimentary and Metavolcanic rocks)

The N-S trending schist belt comprises low grade metasediment which are considered to be upper Proterozoic age. These belts are known to be early Proterozoic super crustal rocks that have been folded into gneissic complex (phyllite, schists, pelites, semi pelites, quartzite, marble, banded iron formation, ultramafite, minor felsic to intermediate metavolcanics, greywacke and amphibolites). The lithological variation of the schist belts compose of coarse to fine grained clastic, pelitic schists, phyllite, banded iron formation, carbonate rocks and mafic metavolcanic (amphibolites) with some fragment material from small back arc basin. The schist belts are mostly developed in the western part of Nigeria including the following groups, the Iseyin Oyan river Schist Belt, Ilesha Schist Belt Kusheriki Schist Group, Karaukarau Schist Belt, the Kazaure Schist Belt, Maru Schist Belt, Anka Schist Belt, Zuru Schist Belt, Zungeru Schist Belt, Kushaka Schist Belt, Iwo Schist Belt, and Igara Schist Belt though smaller occurrences are found mostly in the eastern part which has extensively been studied in the Rahman (2006). Grant (1974) for example recommend that there were strict basin of deposition with different ages of sediments base on structural and lithological association whereas Oyawoye (1972) and McCurry (1976) consider the Schist Belts as relict of a single supercrustal cover. Olade and Elueze (1979) consider the schist belts to be fault guarded rift like structures. However, Ajibade (1980) differ from this conclusion and show that both series contained identical deformational histories. The geochronology of the schist belts remain challenging although the ages of the intrusive cross cutting Older Granite provides a lower limit of ca 750 Ma, Rb/Sr age of 1,040± Ma for Maru belt phyllite which has been accepted by Ogezi (1977) metamorphic age. The schist belts are best developed in the western part of Nigeria and belts are confined to a NNE trending zone of about 400km wide. The area to the west of this zone is made up of gneisses that constitute the Dahomey Basin. Similarly on the east of the basin, no schist belts are recognized for a distance of 600km except Cameroun where a number of Schist belts are considered to be upper Proterozoic. Detailed mapping and study of the Schist belts were carried out in the following localities; Iseyin, Iwo Oyan, Maru, Zungeru, Anka, Kushaka, Zuru, Kazaure, Kusheriki and Ilesha where they are known to be associated with gold mineralization as shown by Ajibade (1980) who first mapped a structural discontinuity.

2.2.1.3 Older Granite (Pan African Granitoids)

The term "Older Granite" as introduced by Falconer (1911) is a distinguishable deep seated, often disconcordant or semi concordant granite of the Basement Complex. They range widely in ages and composition and represent a long lasting magmatic cycle associated with the Pan African Orogeny. The early appearance of these rocks is basic - intermediate intrusions which are represented by small irregular bodies of pyroxene, quartz diorite, and gabbro in the northwestern part of Nigeria which now form part of the basic – acid complex. Granitization on an extensive scale strongly identifies the earlier rocks and the extensive migmatite gneiss which culminated in the intrusion of various Syn – to late tectonic granite, granitoids and syenite. The rocks of this group range in composition from tonalite, diorite, granodiorite, granite, syenite and charnockite.

They are generally high level intrusions which belong to two generations, the first preceeding the emplacement of older granite and second emplaced in the last stage of Older Granite Orogeny (Rahaman, 1981). The Older Granite group is known for its lack of associated mineralization. The contact features between the members indicate the coexistence of magma. Compositionally, the rocks contain significant amount of alkalis and slight normative corundum (Dada, 2006) suggests that the term “Pan African Granitoids” be used for the Older Granite not only on the merit of age that was not available at the time but based on several important petrologic groups formed at the same time. The Granitoids which outcrop with the schist belt in south and northwestern Nigeria include biotite – muscovite - granite, biotite granite, charnockite, syenite, serpentinites and anorthosites. Rahaman (1988) disagree with the earlier classification of Older Granite group on the basis of their texture, mineralogical composition and the relative timing of their emplacement. He classified them based on textural characteristics as follows; magmatic granite, granite gneiss, early pegmatite, fine grained granite, porphyritic granite, slightly deformed pegmatite, quartz vein and aplite.

In northern Nigeria, the large quantity of Pan African granite appears to augment eastwards. They occur west of Zaria as isolated intrusion. McCurry (1976) divided the granite into two main groups according to their field relationships. The first “syntectonic” group include elongated batholiths which are partly concordant and foliated. The second discordant bodies are rich in mafic xenoliths a lower proportion of potassium feldspar. The later is considered to be the products of the extensive mobilisation and reactivation of the older basement rock during the Pan African Orogeny. The Older Granite occurs intricately associated with the migmatite- gneiss complex and schist belts into which they intruded. However, Older Granite is particularly found around Ado- Ekiti, Abuja, Wusasa (Zaria), Akwanga, Bauchi and Obudu areas. In Bauchi area, these rocks occur as dark, greenish grey granite with considerable amount of olivine (fayalite) and pyroxene occurring with quartz, feldspars and micas. Based on their composition, the Older Granite in the area is termed Bauchite and Oyawoyite (after Professor Oyawoye who first mapped them) in southwestern Nigeria. For uniformity of terminology, both the Bauchite and Oyawoyites constitute the charnockite of the Basement Complex. According to (Dada, 1998), charnockite was first described within the Nigeria basement at Toro by Falconer (1911) as a "quartz diorite porphyrite". It was assume to present a certain resemblance with the basic member of the charnockitic series of the Ivory-Coast.

Wright (1985) described it as an annular complex of hypersthenes diorite at the centre of three circular, concentric granites and considered older than the granite from contact relations. Rahaman (1981) consider granite and charnockite as either simultaneous or the latter emplaced shortly after the former. The basement in Toro area consists of gneiss into which the charnockite complex intruded (Dada, Lancelot and Briqueu 1989). This charnockite could be described as a fine to medium grained, greenish black, equigranular and massive, sometimes porphyritic. The granite consists from the periphery (in contact with the migmatite gneiss) towards the centre (in contact with the hypersthene diorite) of a fine to medium grained biotite muscovite granite, equigranular biotite- hornblende granite and porphyritic biotite hornblende granite. Other localities where charnockite occurs include Idanre, Akure, Ado-Ekiti, Ikare (Ekiti), Bauchi and Obudu plateau.

2.2.1.4Undeformed Acid and Basic Dykes

The undeformed acid and basic dykes cross cutting migmatite gneiss complex, schist belt and Older Granite are late to post- tectonic Pan African. The dykes include:

(a) Felsic dykes such as muscovite, tourmaline and beryl bearing pegmatite, micro-granite, aplite and syenite dyke which are associated with Pan African granitoids.

(b) Basic dyke such as dolerite and the less common basaltic, lamprophyric dykes which are regarded as the youngest (Ca 500Ma) unit in Nigeria basement (Dada, 2006).

The Palaeozoic and Precambrian rocks in Nigeria can be divided into four major groups. The Basement Complex (sensu stricto) comprises rocks older than the late Proterozoic metasediment which include metasediments of high grade metamorphism such as parageneiss, basic and calcareous schist, marble and quartzite as well as orthogenesis. The whole Basement Complex has been through at least two tectonic cycles and consequent metamorphism, migmatization and granitization has extensively modified the original rock so they generally occur as relict rafts and xenoliths in migmatite and granite. The metasediments recognised within the Basement Complex are believed to be relicts to an old supracrustal cover of Birrimian age which are termed older metasediments in order to distinguish them from late Proterozoic supracrustal sediment known as younger metasediments.

This younger metasediment trending in the N- S belts are extensively developed in the northwest and they are not recorded east of longitude 8° but believed they represent remnant of a once more wide supracrustal cover. They were steeply folded during the Pan African Orogeny along the Basement Complex so as to occur in synchnorial keels in a sea of granitic material, resembling the greenstone migmatite association of cratonic regions. They were steeply folded during the Pan African Orogeny along the Basement Complex so as to occur in synchnorial keels in a sea of granitic material resembling the greenstone migmatite association of cratonic regions (McCurry and Wright 1976). The volcanic rocks are the youngest rocks recognized in both north – west and north - east Nigeria. They belong to post Older Granite episodes of high level magmatic activity, preliminary age determinations suggest that these rocks were intruded during uplift and fracturing in the final stages of the Pan African Orogeny. For this reason, they are included in a description of the basement rocks and not with the other volcanic rocks of Nigeria.

2.3 Previous work

Lateritic soil is widely depended upon by the construction industry where it is used as either foundation or construction materials of roads, buildings, dams, bridges and embankment as a result of its availability and favourable geotechnical properties. Quite a number of people have worked on the properties and possible uses of laterite and it has gained wider application since 1807 when Buchanan first identified and named the tropical soil laterite. Alexander and Cady (1962) define laterite as extremely weathered material rich in secondary oxides of aluminium, iron, or both. It is nearly devoid of base element with primary silicates but may include huge amount of quartz and kaolinite. It is either hard or capable of hardening on exposure to wetting and drying. Maignien (1966) and Gidigasu

(1976), define the term “lateritic soil” as reddish brown residual and non-residual weathered tropical soils which are capable of hardening when it is wetted or dried and classified them into three stages of formation; the first stage is characterized by physico-chemical breakdown of primary mineral and discharge of constituent elements.

The second stage is laterization which involves leaching under suitable drainage conditions of combined silica and bases with a relative accumulation or enrichment from outside sources of oxide and hydroxide of sesquioxide (mainly Al_2O_3 and Fe_2O_3, the most resistant compounds to leaching). The soil condition under which the various elements are considered soluble and removed through leaching depends mainly on pH, chemical weathering of the primary minerals and drainage conditions. Based on the conditions of intense and prolonged chemical weathering, even clay minerals are damaged and silica is leached.

The third stage is desiccation (or dehydration) which involves biased or complete dehydration (sometimes involves hardening) of secondary minerals and sesquioxide rich materials. Under tropical conditions of rain fall and high temperature, clay minerals tend to decompose into various forms of aluminous and iron oxides in relation to the nature of weathering conditions. Laterite formation factors include climate (precipitation and temperature), topography, drainage, vegetation, parent rock (iron rich rocks) and time.

Climate is considered the most important of these primary factors. Maignien (1966) and Loughnan (1969) discussed on the effect of high temperature and rainfall on tropical weathering processes and are both direct causes of predominantly chemical weathering of rocks and leaching, and indirect, through the influence of the vegetation and high level of bacterial activity in the soil. Although laterization in a distinct wet and dry climate leads to the development of a titaniferous crust.

Topography of laterite is also important since the profile is related to relief, with a regular sequence of different soils encountered from hill top to valley bottom due to differences in parent materials. Differences in site include difference in slope, drainage and position in relation to other soils. Adeyemi (2003) worked on the influence of topography on some engineering and geological characteristic of sandstone derived lateritic soil from south-western Nigeria and his studies show that soils along the gentle slope that has undergone better drainage are more laterised than those from flat terrain. This phenomenon is responsible for higher amount of gravel size particles, lower amount of sand size particles, lower plasticity and higher strength characteristic exhibited by soil samples along the gentle slope than those on the flat terrain.

Alao (1983), studied the geotechnical property of lateritic soil in different parts of Ilorin. His studies show that the terrain is made up of three lateritic layers which are;

I. Laterite crust: This layer is the topmost layer, it has a cellular texture and usually hard to break.
II. Laterite gravel: This layer is usually pisolithic, found below the lateritic crust; it is a mixture of small stone and coarse sands with variable thickness.
III. Laterite clay: This is a fine grained layer overlying the weathered basement. It is reddish brown in colour with patches of pinkish white material and it is mostly employed for engineering construction purposes. It is found to be

rich in SiO_2 (>45%), Fe_2O_3 (>16%) and Al_2O_3 (>10%) which are not expected to perform well as concrete.

Maignien (1966) revealed that there is a close relationship between the occurrence of an indurated horizon and the succession of soil along slopes. The movement of sesquioxide across a terrain is due to oblique leaching of the soil solution, therefore the concept of soil chain is of great interest in connection with the mechanism of laterite rock formation.

D'hoore (1954) revealed that the sesquioxide bearing water moves laterally downward in relation to topography and contributes to the enrichment in the low-lying position. It appears that slopes at the base of higher lying land, low lying plains which receives water from other nearby or distant areas and similar topographic position are frequent sites of enrichment outside source.

Malomo, Obadina and Adebo (1983) define laterite simply as soil that does not have reproducible results with standard laboratory testing procedures while Shellman (1983, 1986) defined laterite as products of intense sub-aerial rock weathering which consist predominantly of mineral assemblage of goethite, aluminium hydroxides, kaolinite and quartz.

The mineralogical and engineering properties of some Nigerian lateritic soils have been studied considerably by some researchers (Madu, 1976; Ola, 1978; Meshida, 1987; Ogunsanwo, 1986, 1988; Adeyemi, 1997, 2001, 2002&2003). Their studies revealed that most of the soils are composed predominantly of quartz and kaolinite or predominantly quartz with some kaolinite. The most important factor with respect to engineering characteristic is the absence of any swelling mineral type, like montmorillonite. They went further to postulate that engineering property of laterite soils are partly related to the distribution and types of clay mineral present.

Joachim and Kandiah (1941) also defined laterite to include those molecular ratios of silica to sesquioxide (SiO_2/ Fe_2O_3 + Al_2 O_3) that are less than 1.33 while lateritic soils are those with ratio greater than 2.0 are non-lateritic types. Therefore, in order to address this difficulty, lateritic soils are said to include reddish brown colour, with or without noodles, concretions and all products of tropical weathering that are not found below hardened ferruginous crusts or hard pan.

Oladele, Olusola and Emmanuel (2012) studied the engineering properties of lateritic soil developed over migmatite around Dall quarry Ilorin, Their studies revealed that the soil has LL range between 40 and 46%, PL 18.2 and 23.5 %, PI 21.8 and 22.5%, LS 7.4 and 8.2%, activity clay 0.63 to 0.95 (normal clay) with an average CBR value of 4% which are said to be good for subgrade construction material. The cohesion ranges between 60 and 100kpa and angle of internal friction ranges between 31 and 35°. This result shows that the soils have strong bearing capacity with little or no volume change that can support dam construction, shallow foundation, homogeneous embankment, slope stability and subgrade material in road construction.

Bello & Adegoke (2010) evaluated the geotechnical property of laterite soil around Ilesha east, southwestern Nigeria and their studies show that LL range between 15.5 and 48.6%, PL 4.66 and 25.6%, PI 7.17 and 23%, and CBR(un-soaked) 37 and 85%, SG 2.61 and 2.80, and MDD range between 2.3 and 2.62mg/m^3, OMC 14.5 and 28.0%. The result of the grain size analysis shows that percentage passing No 200 BS sieve are 69%, 51%, 33%, 34%, 56%, 32%, 80% and 64% respectively. Based on USCS classification, samples with grain size below 35% is classified as well graded soil. Thus this soil is considered good subgrade and sub-base materials. However samples with grain size analysis of 80% is considered as very poor soil and should not be used as highway construction material. Geochemical characteristics and field performance of most of the soils are influenced considerably by degree of weathering, morphological characteristics, genesis, mineralogical and chemical composition as well as environmental conditions (Vergas, 1953; Terzaghi, 1958; Little, 1969; Gidigasu, 1972&1974). (Sherwood, 1957 and Terzaghi 1958) noted that samples pre-test preparations and testing procedures also affect geotechnical characteristics and reproducibility of test results for lateritic soils.

Elueze, Ekengele, & Bolarinwa (2005) also worked on the geochemical trends of a weathered profile above granite gneiss and schist of Abeokuta area. Their study revealed that Fe_2O_3 (7.5%) and Al_2O_3 (28.9%) are enriched in the laterite and clayey horizon respectively while the other oxides notably Na_2O (0.01%), K_2O (0.98%), CaO (0.01%), and MgO are depleted due to leaching. Ti_2O (0.98%) on the other hand is enhanced due to the presence of leucoxene and anatase which are weathering products of illmenite. Other studies revealed that environmental conditions (Ola, 1978; 1980& Adeyemi, 1999), affect the behaviour of laterite profile.

Adeyemi and Akinseli (1995) worked on the influence of texture of granite on some index properties of residual lateritic soil and their result revealed that the differences caused by the parent rock texture in the values of water absorption limit, linear shrinkage, specific gravity of grains and grain size distribution were significant while that of plasticity was not.

Therefore importance of parent rock texture should be given proper consideration in order to understand the engineering characteristics of lateritic profile.

Adeyemi and Ogundero (2001) examined some geotechnical properties of soils developed over migmatite gneiss and their results show that the position of the soil had significant influence on the engineering properties such as plasticity index (12.72%), consolidation (56.55), unconfined compressive strength (49.38kpa) and coefficient of permeability (68.47mm/sec).

Adeyemi (2002), worked on the geotechnical properties of lateritic soil developed over quartz schist in view of assessing the suitability of the residual lateritic soil for highway subgrade and sub-base material and his study shows that between 80% and 95% kaolinite, 2 to 4% illite, and amount of iron in terms of ferric oxide range from 6.79 and 10.5% while silica sesquioxide of iron and aluminium molar ratio were between 1.80 and 1.87. Comparison of the value with those of moulded or for linear shrinkage and asphalt institute for CBR shows that the soil meets the international standard for flexible highway subgrade and sub-base material. The value for linear shrinkage varies between 4.3 and 6.4% while CBR value range between 5.7 and 83.3%.

Omotoso, Mamodu and Ojo (2011) evaluated geotechnical property of lateritic soil around Asa Dam area, Ilorin, southwestern Nigeria based on their geotechnical properties, permeability and suitability as construction materials. The grain size analysis shows that one of the samples is silty clayey sand with 21% silt, 24% clay, 45% sand and 10% gravel while the other sample is a clayey sand with 13% silt, 35% clay, 47% sand, and 5% gravel. Samples are above the A- line in the zone of intermediate plasticity clay (CI). Sample one contains normal clay (activity 0.6) which suggests that there is negligible or no swelling of the soils as construction material since values obtained are 27.6 and 22.6% for plasticity index, 8.6 and 9.2% for linear shrinkage, 44 and 46% for LL, 16.4 and 23.4% for PL, 2.61mm/sec and 2.72mm/sec for permeability respectively. The CBR value for unsoaked and soaked proctor test range between 2 to 4%, shear test gave angle of internal friction of 31° and 33° with cohesion of 59 and 70Kpa. Based on the values obtained from laboratory tests the soils are said to be good for subgrade in road construction, dam, and embankment can support a moderate steep slope to a greater height.

Osuolale, Oseni and Sanni, (2012) worked on the rate of highway pavement failure along Ibadan – Iseyin road, Oyo state, Nigeria and their studies revealed that the percentage passing sieve No 200 for grain size analysis are 17.30 and 32% for subgrade and 19 and 39.10% for sub-base, LL and PI for subgrade 26 to 35% and 9 to15% while LL and PI for sub – base range between 8 and 15% and 26 and 35% respectively. The value of MDD and OMC ranges from 1.88 to 2.12g/cm^3 and 10.15 to 13.2% for subgrade and 1.90 to 2.24g/cm^3 and 9.2 to 14.45 for sub – base. CBR values for subgrade ranges from 10 to 29% (soaked) while CBR values for sub –base range 9 – 35% (soaked). The results show that subgrade and sub – base material conforms to Federal Ministry of Works specification for road works except sub-base samples which may be responsible for the failure along the road.

Bayewu, Olountola, Mosuro, and Adeniyi, (2012) Carried out a petrographic and geotechnical properties of laterite soils developed over different parent rocks in Ago Iwoye area, southwestern Nigeria and their studies revealed that grain size analysis derived from biotite gneiss contain 15% gravel, 45% sand 40% fines with LL 40.31% and PL 25.08%, These properties placed the soil in the group A – 3(0) of the AASHTO Classification system. Banded gneiss consist 10% gravel, 36% sand, 34% fines with LL 40% and PI 16.11%. These properties placed the soil in group A – 2 – 7 of AASHTO Classification system. Quartzite schist derived soil contain 18% gravel, 36% sand, 46% fines with LL 48.30% and PI 18.7 and these properties placed the soil in A- 7 – 6 of AASHTO Classification system. Porphyroblastic gneiss derived soil contains 3% gravel, 46% sand, 49% fines with LL 45.20% and PI 17.83. These properties belong to group A- 7- 6 of AASHTO Classification system. Granite gneiss derived contain 4% gravel, 37% sand and 48% fines, LL 42.07% and PI 15.44, LS of 12%, 11%, 9%, 8% and 7% respective. OMC and MDD values at modified AASHTO level are 18%, 17.20%, 17.40%, 15.10%, 16.40% and MDD of 16.5KN/m^3, 16.40KN/m^3, 16.80KN/m^3, 16.9KN/m^3, and 16.18KN/m^3. The study has proved that the influence of parent rock factor on engineering index properties of the soil studied were significant. It is thus necessary to take proper cognizance of parent rock features prior to an adequate consideration of their engineering property and behaviour of residual soil.

Owoseni, Adeyemi, Asiwaju-Bello and Anifowose (2012) worked on engineering geological assessment of some lateritic soil in Ibadan using bivariate and regression analysis and their studies revealed that pedogenic factor of parent rocks significantly influence the engineering index properties of lateritic soil. The particle size distribution characteristics show that the soils are sandy, with values ranging between 33 - 70% (gravel + sand), 30 - 67% (silt + clay), SG 2.70 and 2.77, LL range between 25 and55%, PI 14 and 32, MDD 1660- 1800(West African level), 1820 and 1970Kg/m^3 (Modified AASHTO level), OMC value range between 18 -33% unsoaked, 10 and 22% soaked(WA) 26 and

43% unsoaked and 14 and 30% soaked (MA). Modified AASHTO level compacted effort which produces better compacted soil than the West African level is recommended for the soil.

Ige, Ogunsanwo, and Inyang (2011) worked on the characterization of terrain on the biotite gneiss derived lateritic soils of Ilorin for use in landfill barriers and their studies revealed that SG value range between 2.50 and 2.73, percentage of fines range 20 and 67%, LL range from 21.64 and 46.53%, PI value range 13.47 and 25.95%. MDD value obtained using standard proctor energy range between 1.72 – 1.97t/m^3. Hence this study confirms the suitability of the soil for construction of landfill facility.

Ugbe (2011) worked on basic engineering geological properties of lateritic soils from western Niger delta and his results shows that LL range between 22.2 and 48.3%, PI 4.60 and25.8, MDD value range between 1700 and 2140Kg/m^3, OMC between 7.7 and 18.0% and CBR value range between 3.00 and 43.0% respectively. The soils have fine content ranging from 14 and 50% and extremely low gravel percentage of 0 and 6% which render them unsuitable for granular road base and sub – base course in their natural state. The skepton activities classify the soils as inactive to moderately active clay and PI ranging from 8 to 25% indicates that the soils are of low to medium swelling potential. The CBR value shows the soil cannot be utilize to construct durable road except they are stabilized with appropriate material that are compatible with the soil properties.

Ogunsanwo (1989) worked on CBR and shear strength of compacted laterite soil from south-western Nigeria and the study revealed that the compacted laterite soil possess unsoaked and soaked CBR values that make them adequate for use as sub–base material in road construction as their California bearing ratio (CBR) values fall within the limit specified for this purpose (7 to 20%) by Asphalt Institute (1962) soils which plot in the CH zone of Unified Soil Classification chart are generally not recommended for use as sub–base material as they are illustrated by very low (< 7%) CBR values.

Their shear strength are quiet high under total and effective stress condition and thus with their acceptable permeability level the soil are suitable for use in the construction of embankment and dams. Such soil possesses high initial and long term stability when used in dam or embankment for with C = 0 analyses, $Ø^1$ value vary from 26° to 31°.

Olugbenga, Kolapo, Oludare, and Abiodun (2007) worked on a study of compressive strength characteristics of laterite and sand hollow blocks. The results of the two mixed proportion (1.6 and 1.8) were used with laterite content between 0 to 50%, these results shows that the mix proportion, compacted hollow sandcrete blocks from mix ratio 1.6 with 10% laterite content is suitable and hence could be recommended for building construction having attained a 28-day compressive strength of 2.07N/mm^2 as required by Nigerian standard.

Badmus (2010) worked on plasticity and compressibility characteristic of laterite soils from south-western Nigeria with a view of establishing the relationship between their plasticity and compressibility and determines the effect of the parent rock on the plasticity and compressibility. The study revealed that SG value range between 2.48 and 2.72, PI 9.7 and 21.4%, coefficient consolidation range between 29.39 and 32.56mm^{2}mm^{-1} with coefficient of volume compressibility ranging between 1.08 and 1.94 × 10^{-3}KN/m^2. All these parameters are influenced by the Parent rock, soils derived with little compaction are suitable for landfills sites.

Ogunsanwo (1990) also worked on geotechnical properties of undisturbed and compacted amphibolite derived laterite soil and his result shows that properties such as compression index, void ratio, cohesion, and coefficient of permeability improve with increased compaction energy. Critical pressure and angle of internal friction were more or less constant while the maximum dry density increases only at modified proctor. MDD values when compacted at the energies of West African and modified proctor range $1.59 t/m^3$ and $1.70 t/m^3$ with natural wet density of $1.72 t/m^3$, compaction index range between 0.08 – 0.24 and permeability (K) value range between 1.17×10^{-9} and 4.58×10^{-18}m/s. These properties show that the soil will be suitable material for dam core in view of its very low permeability and improved cohesion with better friction are more or less constant while the maximum density increases only at modified proctor.

CHAPTER THREE

3.0 RESEARCH METHODOLOGY

3.1 Fieldwork

Topographical map of Ogbomosho sheet 222 NE was used to locate the rocks and soils within the study area. The mapping exercise was aimed at reviewing the detailed occurrence and distribution of rocks, structures and position of parent rock on the weathered soil. On the arrival at the outcrops, a walk around and over the outcrops was done; observable features were identified and noted. This gave idea of the distribution of the rocks and their structures. Tools such as hammer, chisel, measuring tape, sample bag and shovel were used during the collection of rock samples.

3.1.1 Sampling

Reconnaissance survey of sample localities was done in order to identify possible sample locations. About twenty three (23) disturbed soil samples were collected from seven different locations along the Ogbomosho – Ilorin Highway and labelled appropriately. The sampling horizons, colours, symbols and textures of soils are shown in Table 3.1. The soil samples were collected by digging pits on each horizon of the profiles. A field notebook was used to record all the observations, measurements and interpretation. The thickness of each horizon was carefully measured with measuring tape in order to draw the soil profile for further investigations while the hand-held Global Positioning System (GPS) device was used to obtain the coordinate of the locations. The soil samples obtained were sealed in a polythene bag to preserve the natural moisture between the soils. Other field characteristics such as colour and coordinate were determined.

Table: 3.1 Sample localities and profile horizons

Symbols	Localities	Coordinate		Thickness (m)	Colour	Texture
		N	E			
L1SP1	Eyenkorin Bridge	8.350	4.452	0.54	Bright red	Coarse
L1SP2	Eyenkorin Bridge	8.350	4.452	0.68	Brown	Fine
L2SP1	Araromi	8.416	4.166	0.52	Bright red	Coarse
L2SP2	Araromi	8.416	4.166	1.22	Reddish Brown	Coarse
L2SP3	Araromi	8.416	4.166	1.64	Brown	Fine
L2SP4	Araromi	8.416	4.166	1.72	Brown	Coarse
L3SP1	Aiyede	8.311	4.384	0.65	Bright red	Coarse
L3SP2	Aiyede	8.311	4.384	0.1.36	Brown	Coarse
L3SP3	Aiyede	8.311	4.384	2.56	Reddish Brown	Coarse
L3SP4	Aiyede	8.311	4.384	3.34	Whitish Brown	Coarse

Table 3.1 cont'd

Symbols	Localities	Coordinate		Thickness (m)	Colour	Texture
		N	E			
L3SP5	Aiyede	8.311	4.384	4.2	Weathered parent rock	Coarse
L4SP1	Otte	8.311	4.384	0.64	Bright red	Coarse
L4SP2	Otte	8.311	4.384	1.78	Reddish Brown	Coarse
L4SP3	Otte	8.311	4.384	2.59	Brown	Fine
L4SP4	Otte	8.311	4.384	1.64	Brown	Fine
L6SP1	Abduka	8.400	4.461	0.88	Bright red	Coarse
L6SP2	Abduka	8.400	4.461	0.94	Bright red	Coarse
L6SP3	Abduka	8.400	4.461	2.2	Brown	Fine
L7SP1	Gambari	8.265	4.318	0.72	Bright red	Coarse
L7SP2	Gambari	8.265	4.318	0.86	Reddish Brown	Coarse
L7SP3	Gambari	8.265	4.318	1.68	Brown	Fine
L8SP1	Lautech Roundabout	8.135	4.238	0.65	Bright red	Coarse
L8SP2	Lautech Roundabout	8.135	4.238	0.95	Reddish Brown	Coarse
L8SP3	Lautech Roundabout	8.135	4.238	1.44	Brown	Fine

3.1.2 Analysis

The collected soil samples from each horizon of the profile were analyzed at the mechanics laboratory of Civil Engineering Department, University of Ilorin, Ilorin while the geochemical analysis was carried out at the Nigerian Geological Survey Agency, Kaduna.

Soil samples were air dried in the laboratory to expel the in – situ moisture content for about three weeks before the commencement of the laboratory investigation. The soil samples were tested according to the BS 1377 of 1990 procedures.

The Geochemical analysis of soil samples and thin sectioning of rock samples were done at Nigerian Geological Survey Agency (NGSA) Laboratory, Kaduna.

3.2 Laboratory Procedures

The soil classification tests which include index and performance test were carried out following the guidelines of the British Standards (BS 1377).

3.2.1 Index Tests

3.2.1.1 Natural Moisture Content

Natural moisture content is the ratio of weight of water present in the soil and weight of dry soil expressed in percentage. It is carried out to determine the percentage of water present in the soil. Apparatus used include moisture can, weighing balance and drying oven.

The moisture content (m) or water content (w) is expressed as

$$w = \frac{Mw}{Ms} \times 100 \qquad \text{(i)}$$

Where, $Mw\ = mass\ of\ water, Ms\ = Mass\ of\ dry\ soil$

Procedure

Moisture can was weighed and soil sample filled with about ⅔ of its volume and reweighed(wt of can + wet soil).Oven dried for 24hrs at temperature of about 105 - 110ºC and the samples were removed from the oven and allowed cooling for 10minutes before reweighing (wt of can + dry soil)

3.2.1.2 Bulk Density

Bulk Density$(\rho)$of a soil is the ratio of the total mass to the total volume. It indicates the heaviness of the soil. It is used to determine the bearing capacity of soil indirectly, in conjunction with angle of internal friction using Terzaghi's formula. The fundamental relation of weight and volume of various component of a soil mass can be derived using the simplified sketches and formula. The quantities which must be known to compute these relationships are weight and volume of wet soil and weight of soil after oven dried.

The bulk density (ρ) is expressed as

$$\rho = \frac{M}{V}(Kg/m^3) \quad \text{(ii)}$$

Where,$M = mass\ of\ soil, V = Volume\ of\ mould$

Apparatus: mould, weighing balance, dry pan, spatula and pestle.

Procedure

An empty mould is weighed (wt of mould) and then filled with sample and reweighed (wt of mould + soil). Soil sample was poured into a clean dry pan and reweighed (Wt of dry pan + soil) and then placed in the oven for 24hrs. The pan is removed from the oven and reweighed (wt of mould + dry soil).

3.2.1.3 Specific Gravity

Specific Gravity is the ratio of a unit mass of a soil to unit mass of water. It is performed to determine the weight of soil. Occasionally, specific gravity (Gs) may be useful in mineral classifications e.g. iron (Fe) having large value of specific gravity than silica (SiO_2) Bowles (1979).

Calculations

$$SG = \frac{w_2 - w_1}{(w_4 - w_1) - (w_3 - w_2)} \qquad \text{(iii)}$$

Where, $W_1 =$ Weight of dry empty bottle

$W_2 =$ Weight of bottle + ⅓ of soil

$W_3 =$ Weight of bottle + ⅓ of soil + water

$W_4 =$ Weight of bottle + water

Apparatus: Density bottle, measuring cylinder, and weighing balance.

Procedure

An empty density bottle with its cork was weighed $(w_1)$and then filled with sample to about ⅓ its volume and reweighed(w_2). The density bottle was filled with water so as to expel air from the bottle and reweighed(w_3). Bottle was then emptied and filled with distilled water and reweighed(w_4). Air can also be expelled from the density bottle by leaving it filledwith water for about 24hrs or by a machine to induce vacuum.

3.2.1.4 Grain Size Analysis

Grain Size Analysis is the determination of diameter of the soil particle that makeup the soil mass. It is carried out in an attempt to describe the soil type. There are two grain size analyses; wet sieving and dry sieving but in this research dry sieving by mechanical method was adopted for particles greater than 0.063mm in diameter. **Dry sieving** requires that the sample particle are disaggregated into their component before sieving while wet sieving requires that the sample is first soaked for 24hrs before sieving with mesh size 0.063mm under running water to eliminate the clay from silt. The residue is then dried. Dry sieving analysis was carried out in this project.

Apparatus: stack of sieves, mechanical sieve shaker, weighing balance, drying oven and clean brush.

Procedure

The set of sieve was arranged in descending order from top with retainer beneath it and 100g of sample was weighed and poured into the sieve stack and then placed on mechanical sieve shaker for about 10-15minutes to enable it separate into different sieves. The whole sieves were weighted and soil fractions retained by each sieve was weighted and recorded. A graph of diameter is plotted against percentage passing.

Calculation

$$\%\, retained = \frac{weight\ of\ soil\ retained}{Total\ weight\ of\ sample} \times 100 \qquad \text{(iv)}$$

$$\%\, Passing = 100 - \sum \%\, retained \qquad \text{(v)}$$

Coefficient of Uniformity (Cu)

$$Cu = \frac{D_{60}}{D_{10}} \qquad \text{(vi)}$$

Where,

$D_{60} = Diameter\ of\ 60\%\ finer$

$D_{10} = Diameter\ of\ 10\%\ finer$

$D_{30} = Diameter\ of\ 30\%\ finer$

Coefficient of Curvature (Cc)

$$Cc = \frac{(D_{30})^2}{(D_{60}D_{10})} \qquad \text{(vii)}$$

3.2.1.5 Atterberg Limit

Atterberg Limit is the absolute ease with which a soil can be deformed. It describes the degree and kind of cohesion, and adhesion between the soil particles as related to the resistance of the soil to rupture. The consistency of the soil depends on its mineral and water content. It may appear in four states: liquid, plastic, solid, and semi-solid. The result is used to deduce the permeability of the soil, bearing in mind that the higher the clay content in the soil, the lower the permeability of the soil. It can also be used to determine the relative rate of settlement of the soil. Thus these tests are used in preliminary stages of building any structure to ensure that the soil will have the correct amount of shear strength and not too much change in volume as it expands and shrinks with different moisture contents.

Apparatus:-Weighing balance, flat glass plate, dry oven, pestle, mortar, spatula, cassagrande apparatus, washing bottle, sieve No 40, 425µm and moisture can.

Liquid limit is carried out in order to determine the amount of water that the soil can absorb before it starts behaving like a liquid. It is the lowest water content above which remoulded material behaves as a viscous fluid and below which it acts as a plastic. Part of thoroughly moulded soil is placed in a brass cup and a groove is made down its centre with a standardized tool of 13.5mm width. The cup is repeatedly dropped at a rate of 25 blows from a height of 1cm and each blow closure of the standard groove correspond to about 1g/m of shear strength (Cassagrande, 1959; Bowles, 1979).

Procedure

A dried sample was gently pounded using the pestle and mortar. Pounded sample was sieved with sieve No 40, 425µm, and 300g and sieved sample was weighed and water was added to it until stiff paste was produced. The stiff paste was packed into the cassagrande apparatus, surface levelled and smoothened using the spatula. A groove in the soil was made with the cutting groove and the handle was rotated (blows) until the groove close.

A small paste was scooped into the moisture can with the spatula, weighed and dried in the oven for 24hrs and then reweighed in order to determine its water content. The soil was emptied from the cup unto a glass plate and water was added to the soil and the process is performed all over again. This was done until the soil behaves likes a liquid, number of blows to close the groove is less than 10.

Plastic limit is defined as the moisture content at which the thread breaks apart at 3mm diameter. It determines the point at which the soil can be rolled into a plastic form by removing water from it. It is determined by rolling out a thread of the fine portion of a soil on a flat, non porous surface the thread retain its shape down to a very narrow diameter when the soil is plastic,. The sample can then be remoulded and the test repeated. Bowles (1979), define plastic limit as the water content at which the soil, when rolled into thread of about 3mm in diameter, crumbles. The plastic limits tend to increase in numeric value for decreasing grain size because it is the lower boundary range of plasticity.

Procedure

A small part of the paste in the cassagrande cup was scooped and rolled into ball and placed on a glass plate then rolled into a thread until it breaks. The crumbled threads were poured into three different moisture cans and placed in the oven for 24hrs (15°C).Soil was reweighed and the plastic limit is determined, the process is repeated for subsequent samples.

Linear Shrinkage is performed in order to know the extent to which the soil type can reduce when dehydrated. Thus it is the water content where additional loss of moisture could not result in more volume reduction. This implies that any changes in moisture below the shrinkage limit do not cause soil volume change but above the shrinkage limit volume change will occur in water content.

Procedure

Distilled water was mixed with about 120g of air dried soil sample from thoroughly mixed portion of material passing 425μm sieve to form a uniform paste. The paste was packed into the cassagrande apparatus with it surface levelled and smoothened using spatula. Two sets of well greased, rectangular aluminium containers was placed on the weighing balance one after the other, and soil was properly trimmed to a depth of 1cm at the point of maximum thickness and placed on balance while the excess of soil was returned to the dish. A ruler was used to obtain the length of the rectangular mould corresponding to the initial length (Lo) of wet soils and placed in the oven for 24hrs. The dry soil in the rectangular mould was then placed on weighing balance after which a ruler was used to measure the new length.

Calculations

$$LS = initial\ length - final\ length \quad \text{(viii)}$$

$$Plasticity\ index = (PI) = LL - PL \quad \text{(ix)}$$

$$Flow\ index\ (F_I) = \frac{W_1 - W_2}{Log\frac{N_2}{N_1}}$$

(x)$Where\ w_1 = Lowest\ water\ content$ %, $w_2 = Highest\ water\ content$

$N_1 = Lowest\ No\ of\ blows$, $N_2 = Highest\ No\ of\ blows$

For example L1SP1

$$Flow\ index\ (F_I) = \frac{42.1 - 57.9}{Log\frac{50}{14}} = \frac{-15.8}{0.55} = -28.7$$

Toughness Index

$$Toughness\ index = \frac{Plasticity\ index}{Flow\ index} = \frac{27.9}{28.7} = 0.97\% \cong 1 \qquad \text{(xi)}$$

3.2.2 Performance Tests

3.2.2.1 Compaction

Compaction Test: - Compaction is a soil improvement test developed by R.Proctor in late 1920's as control specification for the cohesive soil. Proctor (1933) published a series of four articles on soil compaction which are; dry unit weight or dry density, water content, compaction energy and soil type in terms of gradation and uniformity. In the laboratory there are three processes of compaction efforts used in compacting soil, they are the impact, kneading and vibratory process (Lambe, 1951). The impact process was adopted in this analysis and it involves two methods which are;

- Standard Proctor Method
- Modified Proctor Method

The two methods differ in terms of compaction energy, number of layers of soil and number of blows required in the process (Table 3.2).

Table:-3.2 Differences in the proctor methods

	Standard Proctor	Modified Proctor
No of Layers	3	5
No of Blows	25	55
Compaction Energy	$600KN/m^3$	$2696KN/m^3$

Compaction test is carried out in order to increase the density of the soil by removing the voids in the soil under the application of mechanical energy. Density of the soil increases in the process, pore water get ejected from the soil which tends to reduce further settlement into the soil and increase its shear strength, reduce permeability, compressibility and shrinkage of the soil. The maximum dry density (MDD) and optimum moisture content (OMC) is obtained from the graph of dry density against moisture content.

Apparatus**:-**Compaction rammer, mould consisting of base plate, cylinder mould and extension collar, mixing pan, moisture content can, weighing balance, spatula, dry oven, measuring cylinder, and hand trowel.

Procedure

About $150cm^3$(5%) of water was added to about 3kg of soil sample and weighed then poured into the mixing pan using hand trowel. The cylinder mould was then placed on a base plate, and compacted with 25 evenly distributed blows of rammer for the first layer of soil after compacting the volume of the soil reduces; more soil was added into the mould and compacted with another 25 evenly distributed blows. Extension collar was then fixed on the mould before the addition of the third layer of soil so as to achieve a smooth levelled surface. The mould was then filled with more soil and compacted with 25 even blows and reweighed and soil sample is taken from the top and bottom of the mould for water content and dry density determination. The mould is emptied into a mixing pan and 150cm³ of water was added to the soil and mixed. This process was repeated for subsequent samples and the compaction curve was obtained by plotting dry density against water content.

3.2.2.2 California Bearing Ratio

California Bearing Ratio (CBR):- CBR is used to determine the supporting capacity of the soil (Chropa, 1989). The CBR is also used to determine the strength loss from field saturation, expansion of soil beneath the pavement when saturated with water and the bearing capacity of the soil by measuring the resistance of the soil to penetration. The values obtained gives information on the kind of load the soil can support under different moisture – density condition. There are two types of the CBR tests namely:

- Soaked CBR test
- Unsoaked CBR test

The unsoaked CBR is carried out on a fresh soil sample taken from the field while the soaked CBR test has to be soaked in water before the commencement of the test.

Apparatus:-CBR mould (Volume $\times$ 10m³), rammer (4.5kg), extension collar, CBR machine, sample can, surcharge weight, weighing balance, mixing plate, cylinder, dry oven and spatula.

Procedure

6kg of air dried sample is measured into a mixing pan. Water equivalent to optimum moisture content (OMC) of the sample earlier calculated are measured, added and well mixed. The mould is filled with the soil and compacted with 25 blows on each of the three layers. This was done for soaked and unsoaked samples. After compacting the third layer a surcharge weight of 2kg and 40mm thick was placed on the extension collar and placed on the CBR machine plate, while the plate is adjusted until the piston of the machine makes a contact with the soil in the mould. The load and penetration gauge are set at zero, just before the test proceeds by motorized movement of the piston and corresponding reading on the load gauge were taken at different penetration level (Top reading).

The mould was removed from the machine and turned upside down with the piston of the base plate and extension collar changed and then placed back on the plate of the CBR machine and repeated the procedure (Bottom reading).

3.2.2.3 Direct Shear Strength Test

Direct Shear Test:- The shear strength of the soil is the maximum internal resistance of the soil to the movement of particles. This test involves putting the soil sample into a box that split across its middle with a confine pressure alongside a shear force applied to the soil so as to cause a relative displacement between the two parts of the box i.e. the failure plane is made to occur at a predetermined location.

A normal stress which is due to vertical load and shearing stress which is due to confining pressure acts on the failure plane. This test is performed in order to determine the shear strength parameters which include angle of internal friction (θ) and the cohesion of the soil (C). The angle of internal friction is largely dependent on the internal density of the soil and it reduces with increase in the confining pressure.

Apparatus:- Direct shear machine, compaction mould, compaction rammer, surcharge weight and mixing pan.

Procedure

A box of an area of 60 × 60mm was filled with ⅔kg of soil sample, weighed and mixed with the corresponding optimum moisture content. The soil samples were compacted as described in the above procedure. Each sample was then placed in a shear box and a load was placed on top, deformation dial gauge was set at zero and attached to horizontal and vertical position. Readings were taken until the load gauge started to move backward.

At this point the load dial reading was taken and soil were removed from each box and filled with another soil while the load was increased from 5kg to 10kg.The above procedure was followed until a set normal load of 5kg, 10kg, 15kg, and 20kg were applied one after the other in a successive order. The normal stress and shear stress reading was calculated and a graph of shear stress (KN/m^2) was plotted against normal stress (KN/m^2).

The angle of internal friction θ in degree and cohesion in KN/m^2 was obtained from the graph.

Calculation

Normal stress

$$Normal\ stress\ (KN/m^2)\ =\ \frac{Added\ load\ \times Ratio\ of\ machine\ + (wt\ of\ hanger\ and\ frame) \times g}{Area\ of\ Box} \qquad \text{(xii)}$$

$$Where\ Ratio\ machine\ = 10$$

$$Wt\ of\ hanger\ and\ frame\ = 19kg$$

$$g = Acceleration\ due\ to\ gravity\ = 9.81m/s$$

$$Area\ of\ box\ = 3.6\,m$$

$$Normal\ stress\ = \frac{5 \times 10\ + 19\ \times 9.81}{3.6} = 188.0KN/m^2$$

$$Normal\ stress\ at\ 10KG\ \ \frac{10 \times 10\ + 19\ \times 9.81}{3.6}\ =\ 324.3KN/m^2$$

$$Normal\ stress\ at\ 15Kg\ \frac{15 \times 10\ + 19\ \times 9.81}{3.6}\ =\ 460.5KN/m^2$$

$$Normal\ stress\ at\ 20Kg\ \frac{20 \times 10\ + 19\ \times 9.81}{3.6}\ = 596.8KN/m^2$$

Shear Stress

$$Shear\ stress\ =\ \frac{Load\ dial\ reading \times Loading\ ring\ constant}{Area\ of\ box} \qquad \text{(xiii)}$$

$$Shear\ stress\ at\ 5kg\ =\ \frac{232\ \times 1.733}{3.6}\ = 111.7KN/m^2$$

$$Shear\ stress\ at\ 10kg\ =\ \frac{330\ \times 1.733}{3.6}\ = 158.9KN/m^2$$

$$Shear\ stress\ at\ 15kg\ =\ \frac{437\ \times 1.733}{3.6}\ = 210.4KN/m^2$$

$$Shear\ stress\ at\ 20kg\ =\ \frac{559\ \times 1.733}{3.6}\ = 269.1KN/m^2$$

$$From\ the\ graph\ C\ =\ 3OKN/m^2$$

$$\theta\ =\ \frac{220-130}{470-250} = \frac{90}{220}\ 0.409$$

$$\theta\ = tan^{-1}0.409\ = 22°$$

3.2.2.4 Consolidation Test

Consolidation (Odometer)Test: -Consolidation is defined as the process whereby reduction in volume occurs by expulsion of water by static loads. It happens when stress is applied to a soil that causes tightening of the soil particles, therefore reducing its bulk volume. When this occurs in soil that is saturated with water, water squeezed out of the soil. According to Terzaghi (1986) consolidation is any process which involves decrease in water content of a saturated soil without replacement of water by air. It is carried out in the laboratory to determine the rate of settlement of soil (mostly clayey soil) under applied load. The consolidation test is performed to determine the consolidation parameter like compressibility (av), Coefficient of compressibility (mv), and consolidation settlement (s) of soil due to loading at final settlement and creep.

Apparatus: Consolidation device (including ring, porous stones, water reservoir, and load plate), dial gauge (0.0001 inch = 1.0 on dial), Sample trimming device, glass plate, metal straight edge, clock, moisture can and filter paper.

Procedure

The test specimen with dimension 63.5mm in diameter and 25.4mm in height was prepared at liquid limit condition. The soil sample is enclosed inside a metal ring, edge was trimmed, and weight of the ring and the soil sample was recorded. Two porous stone was used to distribute load over the ring above and below and filter paper is added between the soil and the porous stones, the sample was then mounted in the consolidation cell and loading unit.

Water is added into the cell around the sample, so the sample remained saturated during test. Load was applied on the specimen and the corresponding settlement increment reading on a dial gauge was taken for 24hrs and change in the thickness of the sample is recorded during each loading increment. The procedure was repeated with the application of double load to the specimen at two different times. The procedure as described above was repeated for other samples at a load of 5kg, 10kg, and 15kg each for 24hrs.

3.3 Petrographic Analysis

The petrographic analysis is to ascertain the mineralogical composition of the rock through thin sectioning and the modal analysis of the mineral. The thin section was obtained by cutting off thin slides from the specimen using a diamond saw. One side of the rock was then polished to a perfectly smooth surface using abrasives. The polished side was mounted on a thin glass slide using super glue as an adhesive, and heated on an electric heating plate to harden. It was then placed on the rock cutting machine where the specimen was further reduced in thickness to a very thin, almost transparent slide by rotary grinding blades. It was removed from the machine and finished to correct thickness of (0.03mm) using carborundum powder. The surface of the specimen was then covered with a thin glass over slip using Canada balsam and was heated to harden. After producing the slide, it was then subjected to microscopic examination using the petrographic polarizing microscope with a camera, to determine the different mineral constituents and their relative composition. The minerals were studied in cross and plane polarized light and their contact and boundary were inferred in thin section.

3.4 X-ray Fluorescence Analysis

Procedure

Sample drying

About 10g of the sample was weighted into the drying dish and dried to a constant weight (± 0.01g) in the oven at 100° to 120°C and cooled in the desiccator. This was repeated with other samples.

Preparation of glass beads

Glass beads were prepared by mixing approximately 7.60g lithium borate flux with 0.40g of the sample in a platinum crucible. The mixture was then fused on air – acetylene flame (800 to 1200°C) for 15minutes so that the flux melts and the sample dissolves (Bruker axs 2004). The melt was allowed to cool into a one – phase glass bead. Finally the glass bead was transferred into the X-ray analyzer sample holder for analysis.

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Geology

4.1.1 Field mapping

Topographical map of Ogbomosho sheet 222 NE was used as a base map in order to examine the parent rock in which the lateritic soil along the road cut has been derived and migmatite is found to be the most widely spread rock in the area (Figure 4.1).The outcrop around Eyenkorin bridge is a large rock body with a medium grained texture, showing dominantly north – south foliation trends. Texturally, the crystalline basement rock was identified with foliation plane along the light and dark mineral with pegmatite and quartz vein cutting through the host rock as shown in Plates I respectively. Based on the texture, composition and mineral assemblage, the rock was identified to be migmatite, which is a rock that has a feature of both metamorphic and igneous rock. The outcrop in the area is a series of variable migmatite with concordant quartzo-feldspathic segregation and mineral bands. The migmatite is generally dark grey in colour. The dark minerals are rich in biotite and hornblende while the light minerals are quartz and feldspar (plagioclase). Minor folds, joints and minor shear are structures recorded on the migmatite.

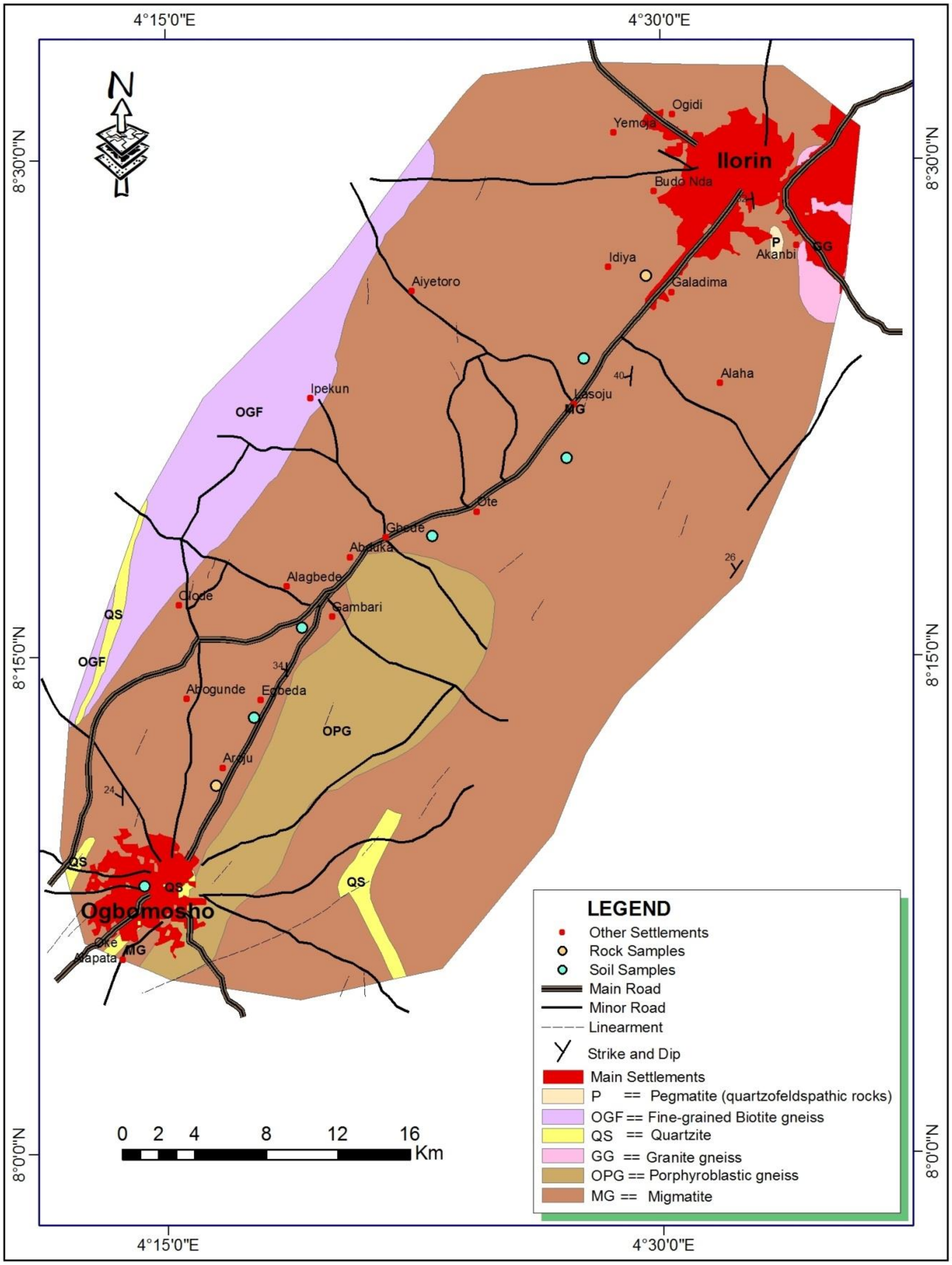


Figure: 4.1 Geological map of study area (Modified by Dindey, 2014)

4.1.1.1 Migmatite

The migmatite occurring as parent rock along Ogbomosho-Ilorin high way is strongly foliated, generally dark grey in colour. The rock is formed by regional metamorphism with disconcordant quartzo-feldspathic vein and streaked mineral. Minor folds, intrusions, joints and minor shear are structures recorded on the outcrop as shown on Plate I. Other rock types in the study area include quartzite, granite gneiss, porphyroblastic gneiss and pegmatite as shown on Figure 4.1. The dominant orientations of Joints are E-W or ENE-WSW suggestive of dominant N-S or NNW-SSE directed stresses, next to these dominant orientations are N-S, NE-SW orientation of lineament.

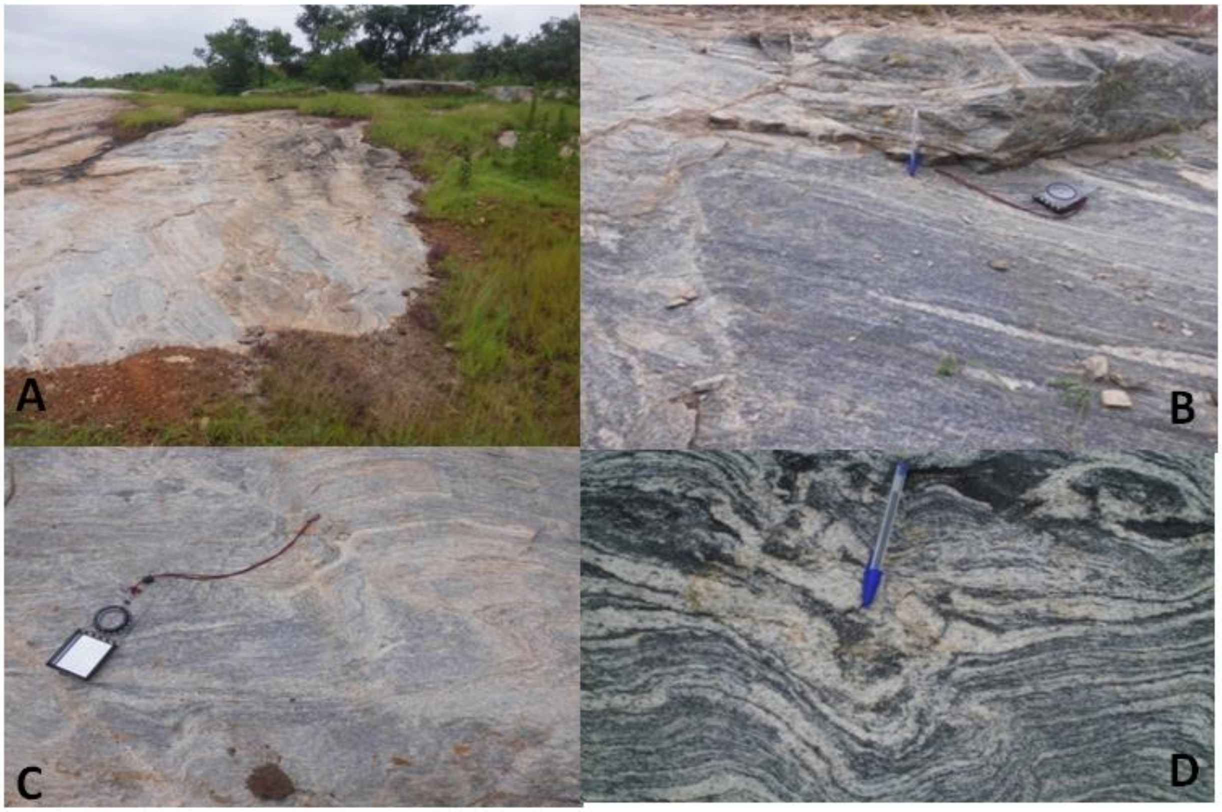


Plate: I Migmatite outcrops in the study area (A – Migmatite at Eyenkorin, B – Migmatite at Lasoju, C – Migmatite at Ote, D - Ptygmatic Folding on migmatite at Gambari, Dindey, 2014)

4.1.1.2 Pegmatite

The pegmatite found is a very coarse grained rock, granitic in composition and occurs as intrusion (vein) in the parent rock (Plate II). The compositions of the pegmatite include feldspar, muscovite, quartz, tourmaline, biotite, and hornblende. They are highly irregular form on the migmatite and discordant.

Plate: II Pegmatite intruding the migmatite (Dindey 2012)

4.1.2 Petrographic Description of the rock

Mineral in thin section include biotite, Plagioclase, hornblende, muscovite, orthoclase, microcline and quartz and the modal and average percentage were estimated. The characteristic of the different minerals observed in thin section (Plate III) are discussed as follows:- The biotite appears grey to brown with subhedral to anhedral habit, perfect cleavage and without twinning. Birefringence is first order and mineral possess no extinction angles. The biotite form interstial lamellae with brown pleichroism and common inner zone of deep green hornblende surrounded an outer biotite and quartz.

The mineral quartz is colourless under plane polarized light with no pleichroism and twinning. The habit is subhedral to anhedral; birefringence is first order appearing with some inclusion and minor fractures. Feldspar appears colourless with subhedral to anhedral birefringence is first order while the muscovite appears colourless with no pleichroism and twinning. The habit is anhedral to subhedral; birefringence is third order with extinction angle at 37°.The hornblende is deep green, pleochroic, prismatic crystal in thin section and twinning is totally absent.

The rock identified from petrographic studies is migmatite which is a medium grained texture, showing dominantly north – south foliation trends, composed of dark coloured mineral such as biotite and hornblende and light coloured mineral such as quartz and feldspar (Plagioclase). The relative mineral compositions are 25% biotite, 6% hornblende, 40% quartz, 12% plagioclase feldspar, 3% albite, 6% microcline, 5% muscovite and 3% accessory minerals. Based on laboratory results, quartz, biotite and feldspar constitute about 77% of the rock.

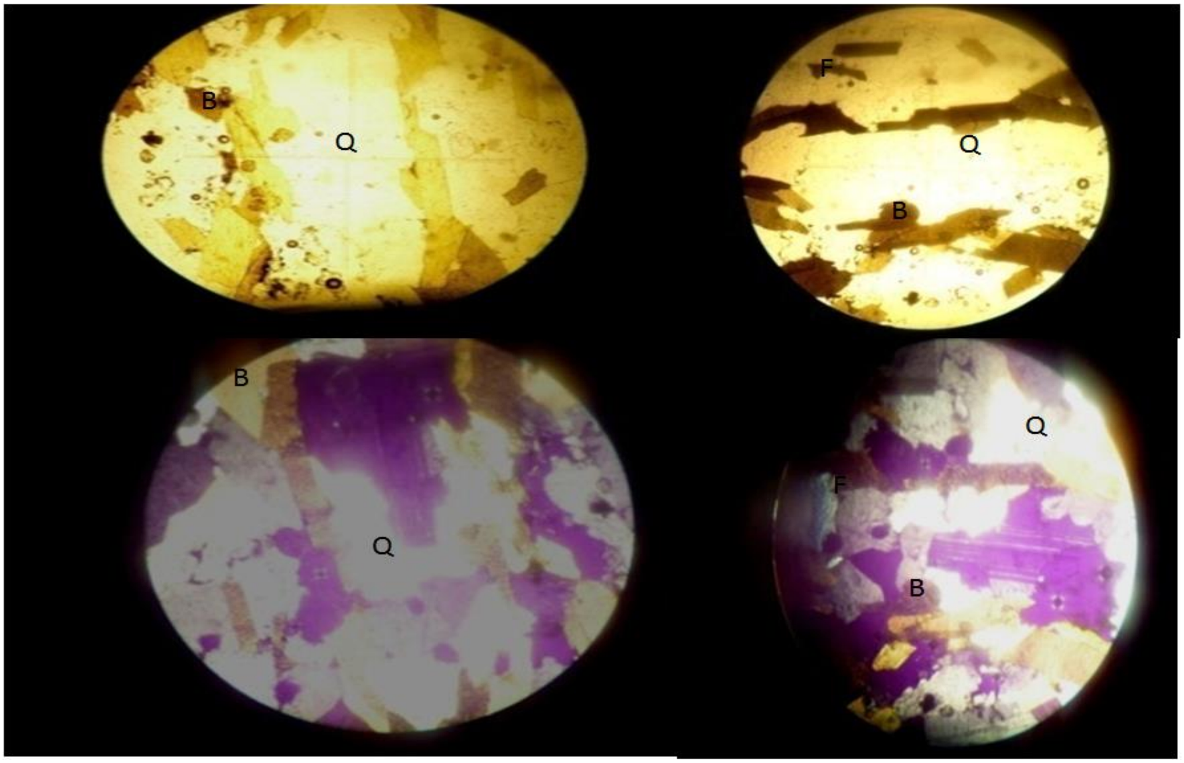


PLATE: III Mineral assemblage in thin section of the Migmatite (B= Biotite, Q= Quartz F= Feldspar; top two, under plane polarized light and last two cross polarized light)

4.2 Soil Profile

The road cuts established some years ago during the highway construction had exposed laterite profiles that reached thickness of about 4m in some locations (Plate IV). About five horizons were identified which consist mainly of thin top soil, reddish brown sandy silty clay, concretionary gravelly reddish brown laterite horizon, mottled horizon, which essentially consist of reddish brown and whitish soil and saprolite horizon which consist predominantly of greyish soil that is weak, soft and easy to excavate. Towards the base lies soil with their relict structure attributed to the retention of the parent rock textures (Figure 4.2).

Plate IV: Cut section on the highway (A - LAUTECH Junction, B -typical road cut with relics of migmatite, C - laterite soil at Lasoju with A horizon absent, D -laterite soil with vein cutting across B and C horizons, Dindey, 2012)

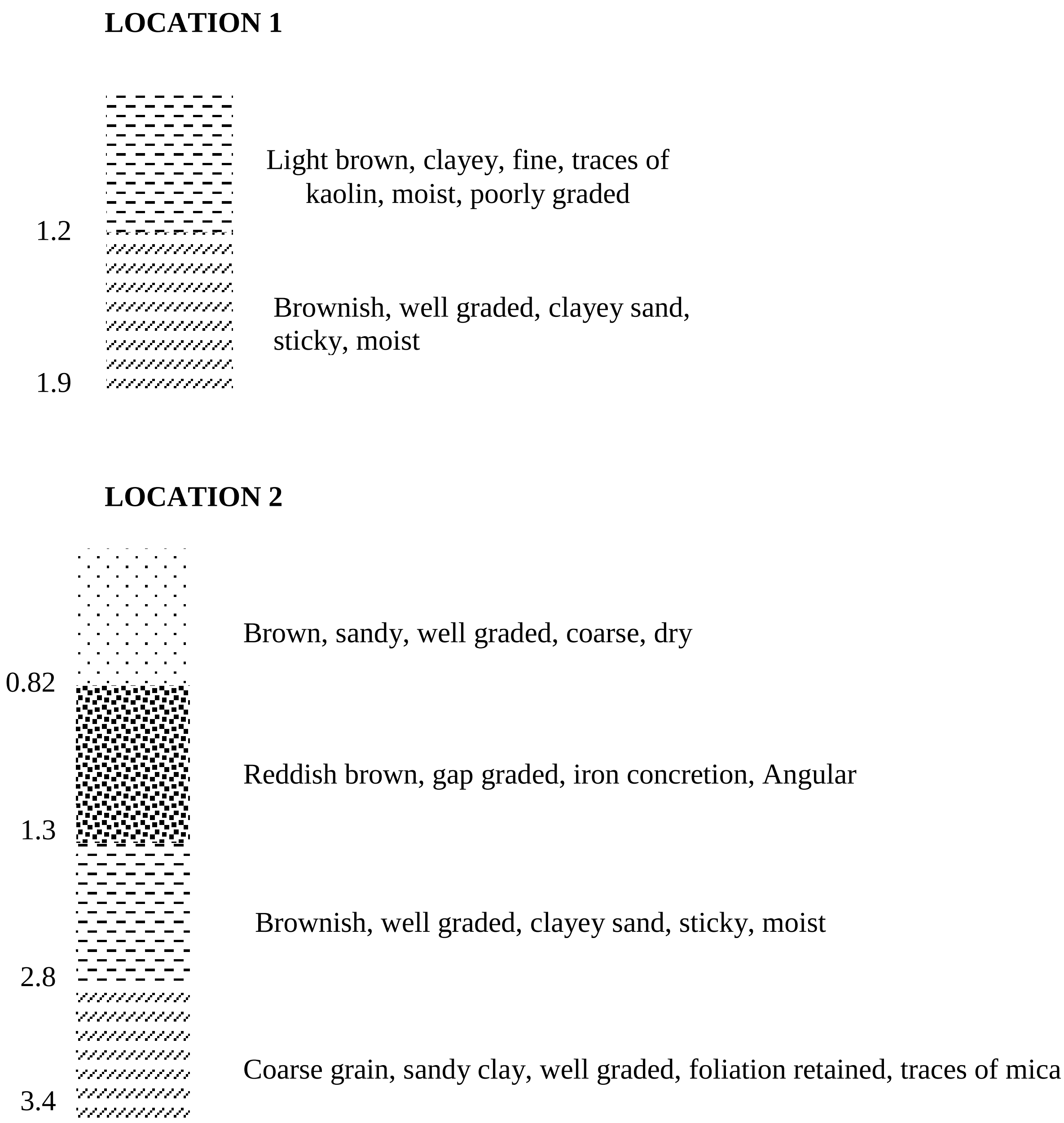
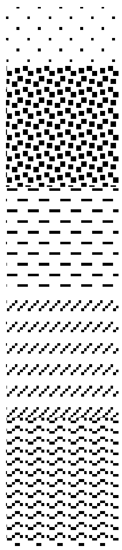


Figure: 4.2 Typical Soil Profile at Eyenkorin Bridge along the road

LOCATION 3



Brown, well graded, coarse grained

0.68

Reddish brown, gap graded, iron concretion, Angular

2.0

Brownish, well graded, clayey sand, sticky, moist

2.7

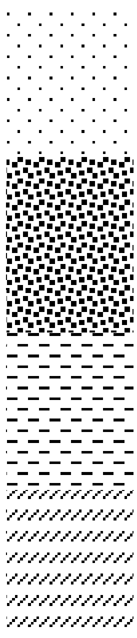
Greyish, sticky, blocky texture, moist, Gap graded, clayey, traces of biotite

3.6

Coarse grain, traces of quartz and biotite, sandy clay, well graded, foliation retained

4.2

LOCATION 4



Brown, sandy, well graded, coarse, dry

0.82

Reddish brown, Gap graded, iron concretion, traces of kaolin, Angular

1.3

Brownish, well graded, clayey sand, sticky, moist

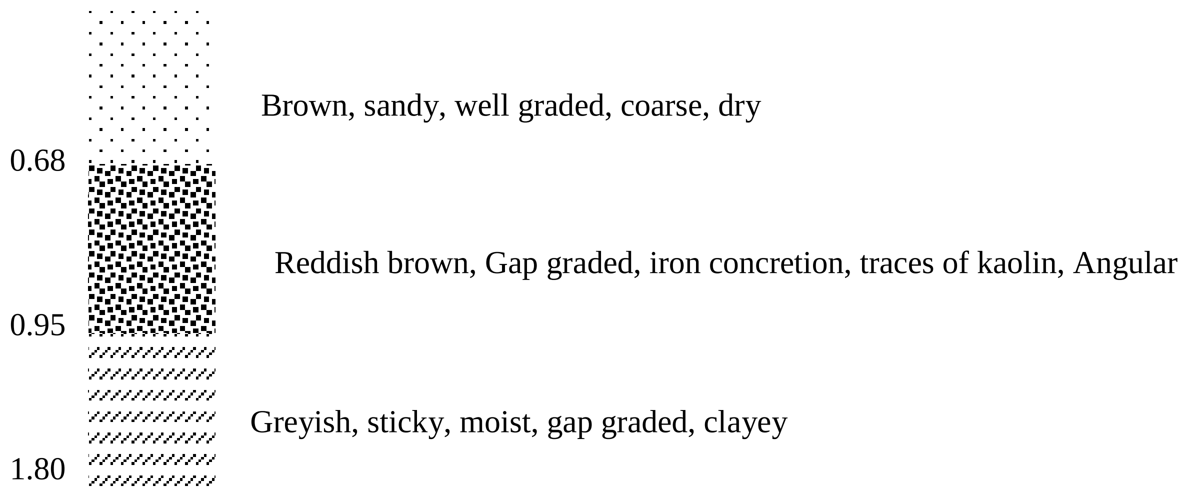
2.8

Greyish, sticky, moist, gap graded, clayey

3.4

Figure 4.2 Typical Soil Profile at Araromi along the Road

LOCATION 6



LOCATION 7

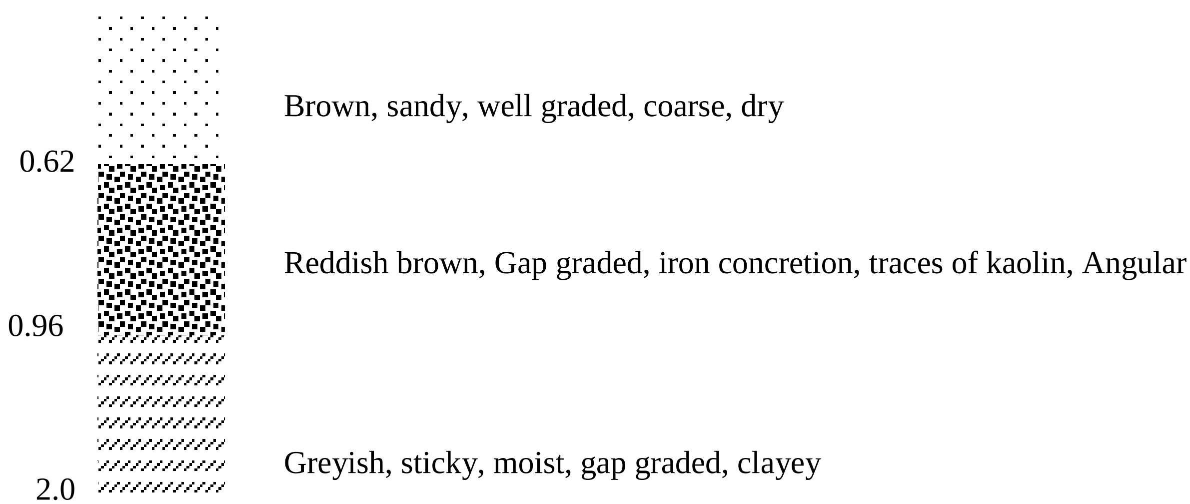


Figure 4.2 Typical Soil Profile at location Aduka and Gambri along the Road

LOCATION 8

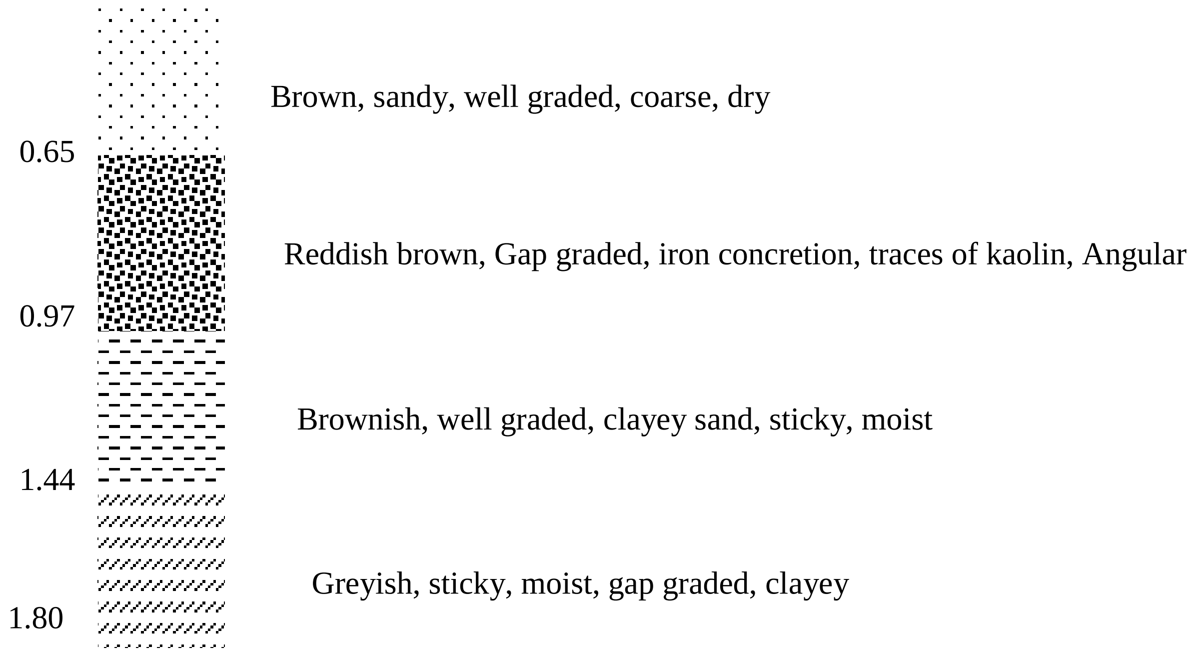


Figure 4.2 Typical Soil Profile at Lautech junction along the Road

4.3 Geochemical Studies

The geochemical analysis of two selected profile locations (L3& L6) revealed major oxides such as SiO_2, Al_2O_3, Fe_2O_3, K_2O, Na_2O, TiO_2, CaO and MnO as presented in the Table 4.1.The result shows that the saprolite, weathered unit and topsoil were dominantly rich in silica (SiO_2), alumina (Al_2O_3) with high percentage of iron oxide (Fe_2O_3) in the mottled horizon (Table 4.1). These three oxides constitute about 70-80 % of the soils, their values show general weathering trends. Fe_2O_3 is enriched at the topsoil, weathered unit (laterite) and mottled horizon while there was depletion of iron oxide within the saprolite horizon or transitional zone (L3SP5). The enrichment of Fe_2O_3 in each of the horizon can be attributed to chemical weathering of mafic mineral composition of the parent rock, abundant sunshine leading to oxidation and ferruginization of Fe bearing mineral. Al_2O_3 enrichment increases from the topsoil down the profile with the silica (SiO_2) behaving in similar manner. This enrichment of Al_2O_3 can be attributed to the weathering alteration of feldspar to clay mineral (Elueze, Ekengele and Bolarinwa, 2004), causing leaching of Al_2O_3 by infiltrating acid rain/recharge water into the ground. Oxides such as K_2O, Na_2O, MgO, Cao, V_2O_5 are decreasing down the profile due to leaching of the soil. TiO_2 also increases down the profile except in L6 where the value is more at the topsoil and weathered unit due to the presence of leucoxene and anatase which are weathering product of illmenite.

Table: 4.1 Percentage of Oxides in analysed lateritic soil samples

Sample No.	SiO_2 %	Al_2O_3 %	K_2O %	Na_2O %	MgO %	TiO_2 %	CaO %	V_2O_5 %	Cr_2O_5 %	MnO %	Fe_2O_5 %	CuO %	BaO %	Eu_2O_3 %	LOI %
Loc3Sp1	50.80	16.60	4.15	1.02	0.30	1.55	0.1	0.081	0.033	0.03	20.27	0.02	0.32	0.15	4.60
Loc3Sp2	52.40	16.40	2.50	1.09	0.48	1.19	0.1	0.077	0.057	0.046	17.14	0.02	0.25	0.17	8.13
Loc3Sp3	51.40	18.10	2.30	0.93	0.11	2.08	0.71	0.092	0.029	0.01	18.13	0.02	0.25	0.17	5.58
Loc3Sp4	39.50	13.00	0.41	0.15	0.04	1.13	0.22	0.13	0.051	0.15	33.72	0.036	0.17	0.29	7.43
Loc3Sp5	67.50	15.30	2.73	1.13	0.89	0.61	2.50	0.028	0.012	0.036	6.280	0.01	0.22	0.094	2.63
Loc6Sp2	85.40	0.33	0.76	0.45	0.35	2.64	0.46	0.095	0.015	0.19	6.429	-	-	0.99	2.80
Loc6Sp3	43.10	13.00	0.85	0.022	0.013	1.49	0.24	0.12	0.044	0.068	27.83	0.03	0.20	0.25	7.91
Loc6Sp4	42.10	20.10	0.61	0.06	0.03	2.02	0.36	0.11	0.027	0.03	23.23	0.037	-	0.22	11.01

4.4 Geotechnical Studies

4.4.1 Index test

4.4.1.1 Specific Gravity

The value of specific gravity ranges between 2.50 and 2.74 with an average value of 2.72 (Table 4.2). These values are normal in accordance with Bello and Adegoke (2010) and Wright (1985) which shows that the standard range of value of specific gravity of soils lies between 2.6 and 2.80. Lower specific gravity values indicate coarse soil, while higher values indicate fine grained soils (Bs 1377). The highest values of 2.74 was obtained at L6SP3 which is B horizon composed of lateritic concretions. Since a good lateritic material should have good specific gravity ranging between 2.5 and 2.75 all values obtained fall within the ranges of value of the specific gravity indicate that the samples have been produced through in-situ weathering of felsic mineral rich granitic rocks.

Table: 4.2 Summaries of the Laboratory Test Results

Samples	Natural Moisture Content (%)	Bulk Density (g/cm^3)	Dry Density (g/cm^3)	E	n(%)	S(%)	S.G (Gs)	Cc	Cu	LL (%)	PL (%)	PI (%)	LS (%)	MDD (g/cm^3)	OMC (%)	CBR (%) Unsoaked	CBR (%) Soaked	Angle of Friction (Ǿ)	Cohesion C (KN/m^2)
L1SP1	17.7	1.54	1.31	0.978	49.4	46.9	2.59	6.8	1.5	50	22	27.9	13	1.77	14	45	24	22	30
L1SP2	16.8	1.55	1.33	0.950	48.7	49.4	2.56	6.2	0.9	54	19	35	14	1.68	16	42	20	24	20
L2SP1	9.3	1.77	1.62	0.636	63.6	40.9	2.65	4	0.7	NL	NP	NL NP	4.4	1.87	11	62	47	32	10
L2SP2	9.9	1.63	1.48	0.799	44.4	33.1	2.67	2.4	0.7	53	27	26	9.4	1.78	14	57	42	30	15
L2SP3	8.2	1.62	1.5	0.759	43.2	28.1	2.64	3.5	1.0	49	24	25.3	8.2	1.78	14	56	38	31	20
L2SP4	7.2	1.6	1.49	0.748	42.8	25.1	2.6	2.9	1.0	NL	NP	NL NP	3.3	1.8	13	59	44	30	10
L3SP1	9.8	1.69	1.54	0.702	41.2	36.6	2.62	3.5	1.5	51	23	28.1	9.8	1.8	14	53	25	28	10
L3SP2	11.5	1.76	1.58	0.695	41.0	43.6	2.67	5.5	1.3	50	18	31.9	11	1.82	14	51	35	30	30
L3SP3	17	1.62	1.38	0.828	45.3	52.2	2.53	3.3	1.2	58	25	32.2	14	1.71	16	51	27	24	20
L3SP4	3.1	1.44	1.40	0.949	48.6	9.4	2.72	9.4	1.7	44	22	22.2	5.0	1.84	13	54	47	27	30
L3SP5	7.6	1.58	1.47	0.803	44.5	25.0	2.64	2.9	1.5	46	25	21.5	6.6	1.89	13	46	24	30	10
L4SP1	11.8	1.54	1.38	0.823	45.1	35.2	2.52	2.4	1.5	43	20	22.9	5.5	1.78	15	61	31	24	20
L4SP2	12.8	1.58	1.40	0.847	45.8	39.0	2.59	3.2	1.3	40	22	17.7	5.4	1.76	15	65	27	25	30
L4SP3	10.9	1.58	1.42	0.791	44.2	35.1	2.55	2.4	1.5	46	22	23.2	6.0	1.74	15	59	21	26	20
L4SP4	14.2	1.58	1.39	0.803	44.5	42.5	2.50	3.3	1.2	44	22	22.2	5.4	1.75	15	57	17	26	30
L6SP1	6.6	1.64	1.54	0.943	48.5	20.5	2.69	5.3	1.3	NL	NP	NL NP	3.3	1.78	17	71	52	27	20
L6SP2	5.6	1.58	1.50	0.849	45.9	25.5	2.65	6.0	1.1	NL	NP	NL NP	3.3	1.65	15	66	46	27	25
L6SP3	3.2	1.56	1.51	0.809	44.7	29.7	2.74	6.7	0.8	43	17	26	5.6	1.82	13	74	53	31	10
L6SP4	11.9	1.54	1.37	0.866	46.4	22.3	2.66	5.8	1.3	51	18	33.4	11	1.57	16	65	42	23	20
L7SP1	7.2	1.48	1.38	0.931	48.2	18.5	2.56	4.4	2.1	45	21	23.6	11	1.73	14	65	61	23	30
L7SP2	8.2	1.53	1.41	0.819	48.0	26.5	2.50	4.1	1.6	48	21.4	23.6	11	1.78	14	66	66	28	20
L7SP3	9.3	1.57	1.43	0.743	42.6	23.7	2.59	3.5	0.9	48	25	22.4	9.9	1.73	14	65	58	23	20
L8SP1	7.4	1.50	1.40	0.773	43.5	19.6	2.61	4.6	1.2	44	20	23.5	8.2	1.82	14	74	61	27	20
L8SP2	6.5	1.46	1.37	0.815	44.9	10.6	2.65	2.4	1.5	40	18	22	8.6	1.80	12	73	65	28	25
L8SP3	8.4	1.55	1.43	0.938	48.4	33.4	2.69	3.8	0.9	45	19	26.5	9.3	1.69	15	68	62	27	25

4.4.1.2 Grain Size Analysis

APPENDIX A shows particles size distribution curve and grain size fraction conducted on particle size greater than 0.075mm diameter respectively. The fine sand ranges between 0 to 87%, medium grained sand 10 to 90%, and coarse grained sand 2 to 62% and Fine gravel between 1 to 40% while very few samples contained medium grained gravel of 5 to 18%. The composition indicated well to poorly sorted sand with gravel (Table 4.3).

4.4.1.3 Atterberg Consistency Limit

The liquid limit (LL) values range between 40 and 58% with an average of 49% while plastic limits (PL) range between 3.3 and 14% (Table 4.2). Plasticity index (PI) is the difference between the LL and PL and a range of value between 17.7 – 35% were obtained which indicate low to medium plasticity, However soils with plasticity index of 20% shows that considerable amount of water can be added before the soil can become liquid, such soil is a desirable foundation material and greater than 35% may have high swelling capacity. Federal Ministry of Works and Housing (FMWH) recommends liquid limit of 35%, plasticity index of 12%as maximum for subgrade, LL 30%, PI 12 as maximum for sub-base and LL 30%, PI 10 as maximum for base course. It is noted that all samples have liquid limit greater than the set standard by FMWH specifications. The LS values from L2SP1, L2SP1, L2SP4, L3SP4, L3SP5, L4SP1, L6SP1 to 3 all fall below 8% standard value. Other samples obtained from L1SP1 &2, L2SP2&3, L3SP1,2&3, L6SP4, L7SP1,2&3, L8SP1, 2 and 3 did not conform and could be assessed as unsuitable for sub – base material, hence majority of the materials are acceptable as sub – base materials.

Table:4.3 Particle size distribution

Samples	Fine sand %	Medium sand %	Coarse sand %	Fine gravel %	Medium gravel %	Description
L1SP1	20	30	48	2		Sand, well sorted
L1SP2	15	30	45	10		Sand with gravel
L2SP1	48	24	26	2		Sand with gravel
L2SP2	45	30	20	5		Sand with gravel
L2SP3	62	26	7	5		Fine sand
L2SP4	65	25	5	5		Fine sand
L3SP1	30	25	35	8	2	Sand, well sorted
L3SP2	30	37	18	12	3	Sand with gravel
L3SP3	0	20	70	6	2	Sand poorly sorted
L3SP4	8	12	52	22	6	Gravelly sand
L3SP5	25	65	8	2		Sand
L4SP1	12	80	7	1		Sand
L4SP2	18	72	8	2		Sand
L4SP3	70	27	3			Fine sand
L4SP4	88	10	2			Fine sand
L6SP1	20	50	28	2		Sand, well sorted
L6SP2	20	50	28	2		Sand , well sorted
L6SP3	0	10	50	22	18	Gravelly sand
L6SP4	18	52	25	5		Sand
L7SP1	0	25	40	35		Gravelly sand
L7SP2	3	30	40	25	2	Gravelly sand
L7SP3	5	35	42	13	5	Gravelly sand
L8SP1	2	12	60	25	1	Gravelly sand
L8SP2	0	12	50	35	3	Gravelly sand
L8SP3	5	15	62	13	5	Gravelly sand

Figure 4.3 showing Plot of PI against LL shows that all samples fall within the CL zone above A-line which classify the soil as inorganic clay of moderate plasticity, which could be gravelly clay, silty clay, sandy clay or lean clay based on Unified Soil Classification System (USCS).Soils with liquid limit values less than 35% are grouped as low plasticity while those with values between 35 and 50 are classified as intermediate plasticity.

4.4.2 Performance Test

4.4.2.1 Compaction Test

The maximum dry density (MDD) for the soil samples varied between 1.57 and 1.87g/cm^3 at standard proctor compaction energy while the maximum moisture content (OMC) range between 11.0 and 16.5% (Table 4.2). This indicates a reduction in shear strength due to high pore pressure without an increase in density even with an increase in compactive effort. According to O'Flaherty (1988), Adeyemi (2001), Ogunsanwo (2009), Ojo *et al.* (2012) range of value that may be anticipated when using the standard proctor method are:

For Clay, MDD may fall before 1.44 and 1.685mg/m^3, OMC may fall between 20 and 30%

For silty clay, MDD range between 1.60 and 1.845mg/m^3 and OMC of 15 and 20%,

For sandy clay, MDD usually range between 1.76 and 2.16 mg/m^3 and OMC 8 – 15% thus, looking at the results of the soil samples, it could be noticed that they are sandy clays, since their MDD values fall between 8 and 15%.

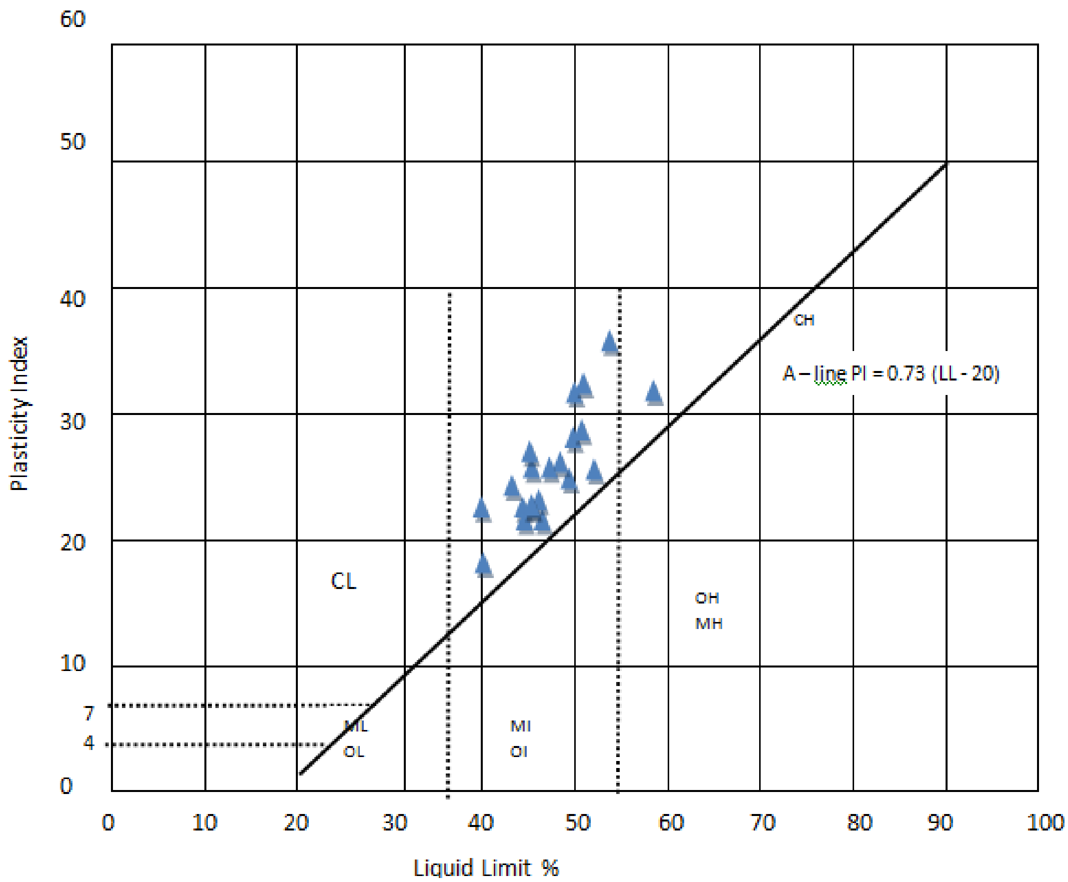


Figure: 4.3 Cassagrande soil Plasticity Chart for soil classification

All samples except L1SP2, L6SP2, L6SP4 and L8SP3 will be more suitable for subgrade and easily compactable since high clay content must be responsible for their low MDD value (APPENDIX B). The decrease in MDD values of C horizons are due high percentage of clay content in the mottled zone. However the values generally fall within the recommendation of previous researcher for purpose of fills in dams, building, and base course in road and liner in landfill.

4.4.2.2 California Bearing Ratio (CBR)

All compacted samples show some difference in their soaked and unsoaked CBR values ranging between 17 and 66% for soaked and 42 and 74% for unsoaked (APPENDIX C). Nigerian Federal Ministry of Works and Housing recommends for road works less than or equal to 10% for subgrade, 30% for sub-base and 80% for base course soil respectively. Thus, all the samples satisfy the condition for subgrade and sub-base material for road construction. Asphalt Institute (1962) also recommended a CBR value of 7 - 20% and 0 - 7% for highway sub –base and subgrade materials (Table 4.4). Since all the samples met the recommendation by Asphalt Institute (1962) the soils is thus qualified for use as subgrade and sub – base materials.

Table: 4.4 General Rating for CBR Asphalt Institute (1962)

% CBR	General rating	Uses	Unified Soil Classification	AASHTO soil Classification
0 – 3	Very Poor	Subgrade	OH, CH, MH, OL	A5, A6, A7
3 – 7	Poor to Fair	Subgrade	OH, CH, MH, OL	A4, A5, A6, A7
7 – 20	Fair	Subbase	OL, CL, ML, Sc, Sm, Sp	A2, A4, A6, A7
20 – 50	Good	Subbase, Base	Gm, Gc, Sw, Sp, Gp	A1b, A2 – 5, A3, A2 – 6
50 – 80	Very Good	Good Base	Gm, Gc, Sw, Sp, Gp	A1b, A2 – 5, A3, A2 – 6
>80	Excellent	Base	Gw, Gm	A1a, A2 – 4, A3

4.4.2.3 Shear Test

The direct shear test is used to determine the angle of internal friction of a soil (ϕ), the cohesion of the soil (C) and effective pressure. The angle of friction (ϕ) range between 22 - 32°, while the cohesion (C) range between 10 - 30KN/m^2 (APPENDIX D).

According to the USCS the results obtained from the shear box test can be use to classify the soils based on angle of internal friction. A soil having angle internal friction less than 20° are classified as soft, between 20 - 35° are classified as hard and above or greater than 35° are classified as stiff. The shear box test shows that the soils are of high strength (Table

4.5).The results obtained favourably compared with other authors as they fall within the same range of value with (Alao, 1999; Ogunsanwo 1989).

Table:4.5 Representative value of ϕ after Unified Soil Classification

Soil	**Φ**
Sand, round grain, uniform	27° to 34°
Sand angular, well graded	33° to 45°
Sandy Gravel	35° to 50°
Silty Sand	27° to 34°
Inorganic Silt	27° to 35°

4.4.2.4 Consolidation Test

The coefficient of compressibility (Av) value range from 1.0 –3.0 × 10^{-2}KN/m^2 with an average value of 1.85 × 10^{-2}KN/m^2, coefficient of volume compressibility (Mv) value range between 1.1 × 10^{-3} and 9.0 × 10^{-3}KN/$m^3$and settlement consolidation between 5.6 – 15mm/m (APPENDIX E). The soil samples have higher average coefficient of volume compressibility due to expelling water continuously with time which makes the sample suitable for construction.

4.5 Influence of parent rock on soil characteristics

The influence of parent rock on soil engineering characteristics was based on the mineralogy and weathering process of the parent migmatite. The high percentage of easily weathered minerals under tropical climatic conditions (plagioclase feldspar and hornblende) that are over 48% of the mineral composition in the parent migmatite has contributed to the fine grained lateritic soil.

The position of the horizon within the profile has varying influence on the plasticity, moisture density relationship, CBR, and shear strength. This study has proven that the influence of the parent rock factor on engineering index properties such as grain size distribution, plasticity index, CBR, shear strength, dry density determination of the soil studied were significant(Table 4.6). It is thus necessary to take cognizance of the migmatite rock characteristic for adequate understanding of the engineering properties and behaviour of its residual soils.

Table: 4.6 Influence of horizon position on the engineering characteristics of laterite derived from migmatite.

Parameters	**Coefficient of variation**	**Natural Influence**	**Specified values**
Liquid limit(LL)	47.24%	Significant	≥30%
Plasticity index (PI)	25.58%		
Shrinkage limit (SL)	8.09%	Insignificant	<8%
Specific gravity (SG)	2.62	Significant	
Maximum dry density (MDD)	2.37g/cm^3	Significant	
Moisture contents(OMC)	14.24%	Insignificant	
California bearing ratio (CBR)	60.20%	Significant	7 – 20%
Angle of internal friction (ϕ)	26.92°	Significant	
Cohesion (C)	20.80KN/m^2	Significant	

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The rock of the study area belongs to the south western Basement Complex terrain of Nigeria which includes granite gneiss, porphyroblastic gneiss, biotite granite pegmatite, quartzite and migmatite, in which migmatite is the dominating rock type and studied in this research. The dominant mineral assemblage from petrographic studies include feldspar, quartz, muscovite, hornblende, biotite, albite and other accessory mineral which exhibit different properties.

The position of the horizon in a lateritic soil profile has been found to significantly control the plasticity, specific gravity, grain size distribution, shear strength, California bearing ratio, and consolidation characteristics. Based on the plasticity chart the samples fall above A- line within the CL zone which classified the soils as inorganic clay of moderate plasticity which could be gravelly clay, silty clay, sandy clay or lean clay. The A horizon which is the top soil in the profile and C horizon which is the mottled zone of the profile overlying the migmatite rock revealed low values in bulk density (1.44 and 1.48 location L3SP4 and L7SP1), High natural moisture content 17.7% (L1SP1), high porosity of 63.6% (L2SP1). Low SG, CBR, value and angle of friction of 45, 42, and 46 L1SP1 and 2, L3SP5 are associated with A and C horizon since the density of the soil mass affect the strength of the soil. Generally strength increases as the dry density increases, hence, the soil with low CBR value and high moisture are not suitable for road construction. Since A horizon is normally scraped, evacuated and often stacked for slope grazing, road cuts intercepting the

C horizons should be given special attention as subgrade material during cut and fill in road construction. B horizon which is lateritic concretion is a good construction material.

The major oxides include SiO_2, Al_2O_3, Fe_2O_3, K_2O, Na_2O, TiO_2, CaO and MnO. The compositional study of these results shows that SiO_2, Al_2O_3 and Fe_2O_3, constitute about 70 to 80% of the oxides. High alumina content, iron and silica content can be attributed to weathering, MgO, CaO and Na_2O are maintained or slightly increased with the weathering process, however, K_2O is immediately decreases due to decomposition of K – Feldspar. In conclusion, the high percentage of feldspathic mineral (plagioclase) and micaceous mineral (biotite and muscovite) coupled with the presence of foliations in the rock resulted in the low resistance weathering and other magmatic alteration processes which may include partial melting and recrystallization during metamorphism.

5.2 Recommendation

Laterite profiles are residual soil as revealed in this research project. Since the profile indicated varied geotechnical and geochemical properties of the horizons characteristics, strict selection of foundation and construction materials for subgrade, sub – base and base course should be taken into consideration. It is recommended that soil with high moisture content and low CBR values should be excavated since it is not good for road construction and replaced with suitable soil. Good drainage systems should also be provided along the road and proper investigation should be carried out before any construction to avoid failures such as pot-holes and cracks which might be caused by the clay content.

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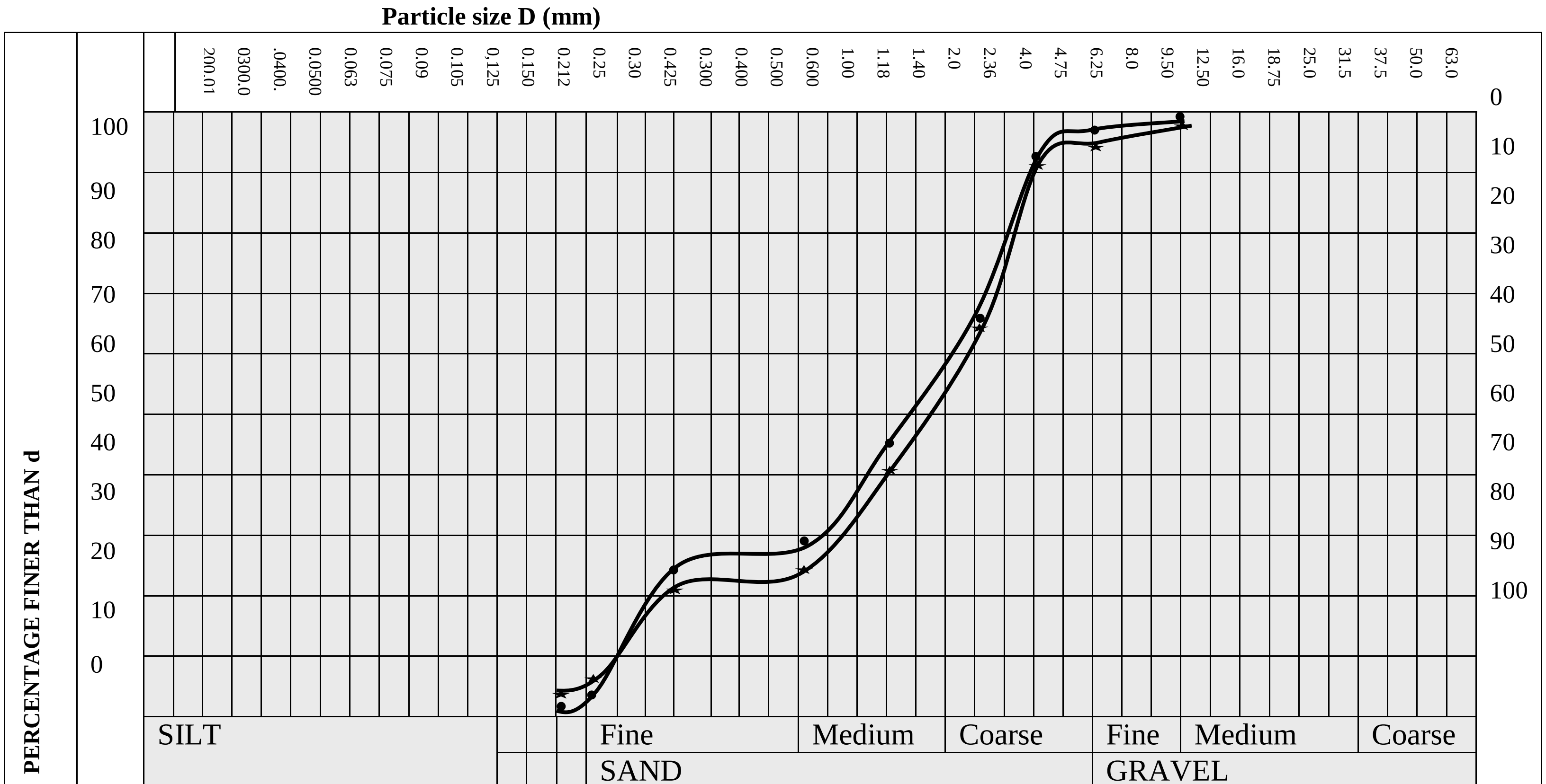
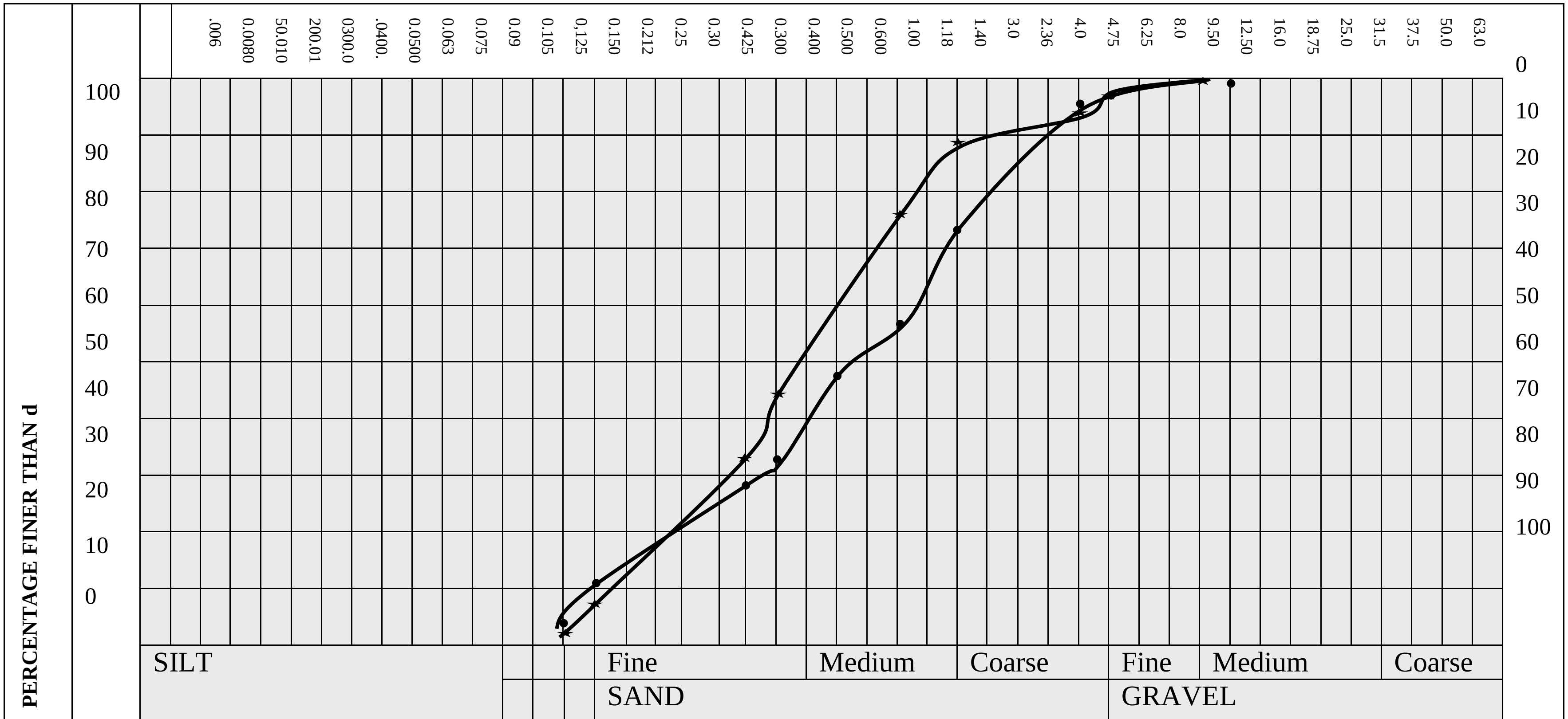
APPENDIX A   PARTICLE SIZE DISTRIBUTION CURVE



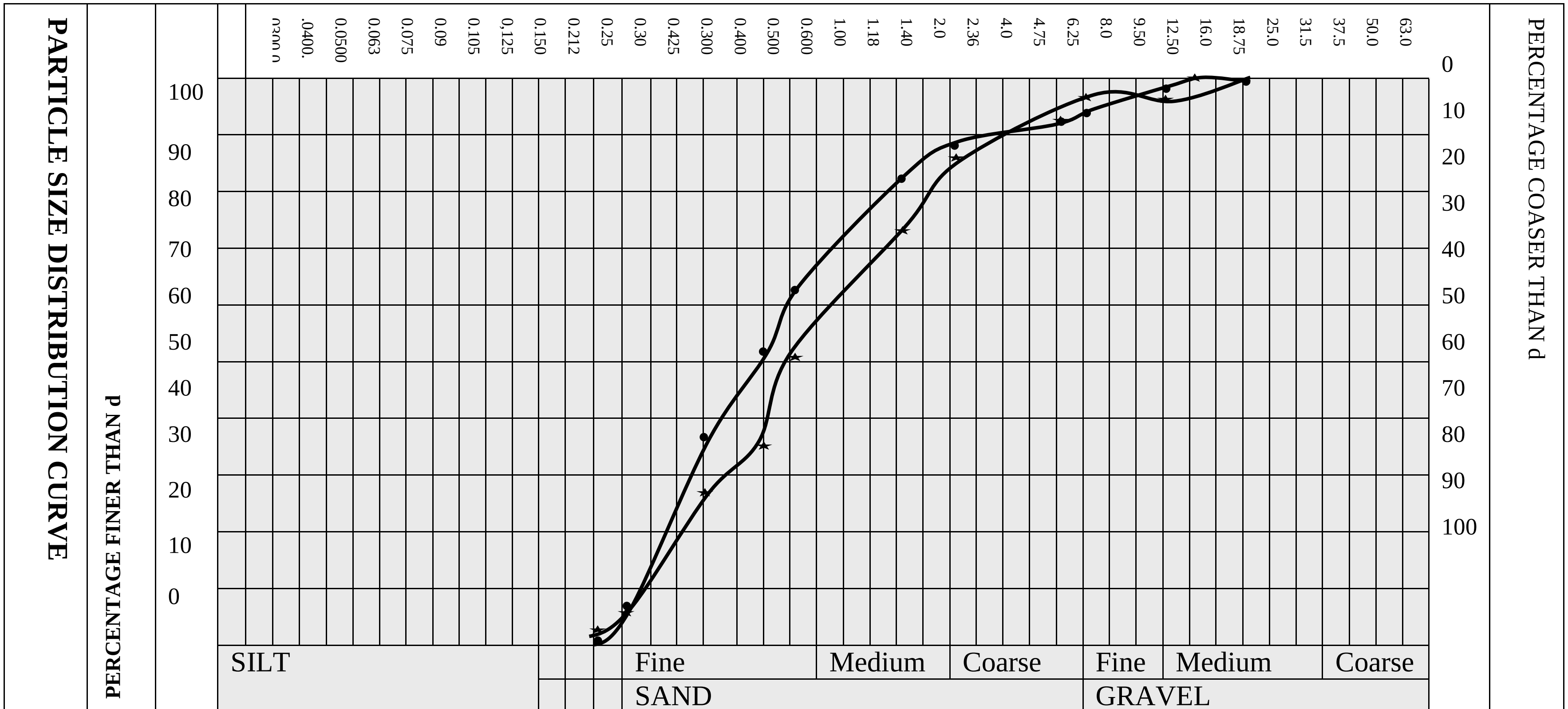
LOC 1   SP 1 & 2

KEY	**SAMPLE**	**DEPTH**	**D_{10}**	**D_{30}**	**D_{60}**	**CU**	**CC**
●	Loc 1 SP1		0.125	0.40	0.85	6.8	1.5
★	Loc1 SP2		0.130	0.30	0.80	6.2	0.9



LOC 2 SP1 &2

KEY	SAMPLE	DEPTH	D_{10}	D_{30}	D_{60}	CU	CC
●	Loc 2 SP1		0.180	0.30	0.72	4.0	0.7
★	Loc 2 SP2		0.180	1.23	0.425	2.4	0.7



LOC 2 SP 3&4

KEY	SAMPLE	DEPTH	D_{10}	D_{30}	D_{60}	CU	CC
●	LoC2 SP3		0.115	0.212	0.40	3.5	1.0
★	Loc2 SP4		0.105	0.180	0.30	2.9	1.0

APPENDIX A

Grain Size Analysis- Mechanical Method

	LOC 1 SP1		
Diameter (mm)	Wt retained	% Retained	% Passing
16			
8	0	0	100
4	1.5	1.1	98.9
2.36	9	6.4	92.5
1	36.5	26.1	66.4
0.5	36.5	26.1	40.3
0.3	23	6.4	23.9
0.25	6	4.3	19.6
0.9	20	14.3	5.3
0.075	3.5	2.5	2.8
PAN	1	0.7	2.1
% Passing= 100-∑%retained			

	LOC 1 SP2		
Diameter (mm)	Wt retained	% Retained	% Passing
16			
8	0	0	100
4	1.5	1.1	98.9
2.36	10	7.1	91.8
1	35	25	66.8
0.5	29.5	21.1	45.7
0.3	24	17.1	28.6
0.25	8	5.7	22.9
0.9	26.5	18.9	4
0.075	4	2.9	1.1
PAN	1.5	1.1	0
% Passing=100-∑%retained			

	LOC 2 SP1		
Diameter (mm)	Wt retained	% Retained	% Passing
16			
8	0	0	100
4	2	1.9	98.1
S2.36	2	1.9	96.2
1	25	23.8	72.4
0.5	24.5	23.3	49.1
0.3	18.5	17.6	31.5
0.25	6	5.7	25.8
0.9	17	16.2	9.6
0.075	5	4.8	4.8
PAN	3.5	3.3	1.5
% Passing= 100-∑%retained			

	LOC 2 SP2		
Diameter (mm)	Wt retained	% Retained	% Passing
16			
8	0	0	100
4	2.5	1.9	98.1
2.36	4	3.1	95
1	8.5	6.5	88.5
0.5	18.5	13.8	74.7
0.3	40	30.8	43.9
0.25	14.5	11.2	32.7
0.9	34.5	26.5	6.2
0.075	4.5	3.5	2.7
PAN	2.5	1.9	0.8
% Passing= 100-∑%retained			

APPENDIX A Cont'd

	LOC 2 SP3		
Diameter (mm)	Wt retained	% Retained	% Passing
16			
8	0	0	100
4	2.5	2.1	97.9
2.36	5.5	4.6	93.3
1	10	8.3	85
0.5	13.5	11.3	73.7
0.3	28	23.3	50.4
0.25	18.5	15.4	35
0.9	34.5	28.8	6.2
0.075	3	2.5	3.7
PAN	2.5	2.1	1.6
% Passing= 100-∑%retained			

	LOC 2 SP4		
Diameter (mm)	Wt retained	% Retained	% Passing
16	0	0	100
8	2	1.5	98.5
4	3	2.3	96.2
2.36	4	3.1	93.1
1	6.5	5	88.1
0.5	7.5	5.8	82.3
0.3	26	20	62.3
0.25	15	11.5	50.8
0.9	58	44.6	6.2
0.075	3.5	2.7	3.5
PAN	4.5	3.5	0
% Passing= 100-∑%retained			

	LOC 3 SP1		
Diameter (mm)	Wt retained	% Retained	% Passing
16			
8	0	0	100
4	2	1.8	98.2
2.36	1.5	1.4	96.8
1	10	9	87.8
0.5	36	32.4	55.4
0.3	33.5	30.2	25.2
0.25	8	7.2	18
0.9	16.5	14.9	3.1
0.075	2	1.8	1.3
PAN	1.5	1.4	-0.1
% Passing= 100-∑%retained			

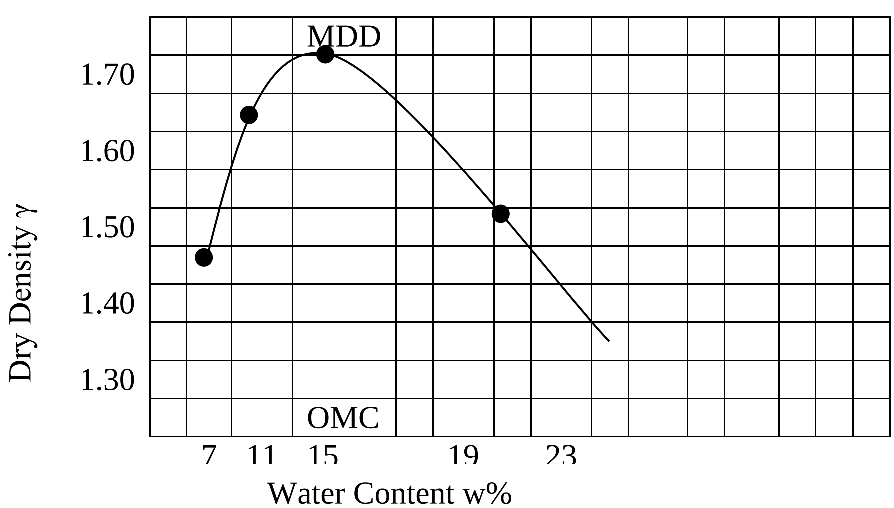
	LOC 3 SP2		
Diameter (mm)	Wt retained	% Retained	% Passing
16			
8	0	0	100
4	7.5	5.6	94.4
2.36	12	8.9	85.5
1	28	20.7	64.8
0.5	31	23	41.8
0.3	26.5	19.6	22.2
0.25	8	5.9	16.3
0.9	16	11.9	4.4
0.075	2	1.5	2.2
PAN	3	2.2	
% Passing= 100-∑%retained			

APPENDIX B

Compaction Test

Project….M.tech Project……………………… `
Location…Ogbomoso-ilorin highway……………
Tested by……Dindey Azeezat
Sample No……L1SP1………

COMPACTION TEST								
SAMPLE NO	LOC 1 SPI							
BLOWS LAYER 25	NO OF LAYERS 3		WT OF HAMMER 2.5KG					
MOLDDIMMENSION: 10.5CM **AT** 11.5CM **VOL** 1000CM^3								
WATER CONTENT DETERMINATION								
Sample no	1		2		3		4	
Moisture can no								
Wt of wet soil + can	51.0	48.5	69.8	75.6	82.5	90.0	91.0	91.4
Wt of dry soil + can	47.5	45.5	65.0	70.6	74.5	81.5	77.5	78.0
Wt of Water	3.5	3.0	4.8	5.0	8.0	8.5	13.5	13.4
Wt of can	4.0	7.5	13.0	15.6	18.0	21.5	13.5	14.5
Wt of dry soil	43.5	38.0	52.0	55.0	56.5	60.0	64.0	63.5
Water content, w %	8.0	7.9	9.2	9.1	14.2	14.2	21.1	21.1
DENSITY DETERMINATION								
Assumed Water Content		8.0	9.0	14.0	21.0			
Average Water Content w%(m)		8.0	9.0	14.2	21.1			
Wt of soil + mold		3442	3576	3792	3717			
Wt of mold		1854	1854	1854	1854			
Weight of soil in mold		1588	1722	1938	1863			
Wet Density, Dw		1.588	1.722	1.938	1.863			
Dry Density Y = Dw 100/100 + m		1.47	1.58	1.70	1.54			



Optimum moisture = 14.0%   Maximum dry density = 1.70g/cm³

APPENDIX B Cont'd

Compaction Test`

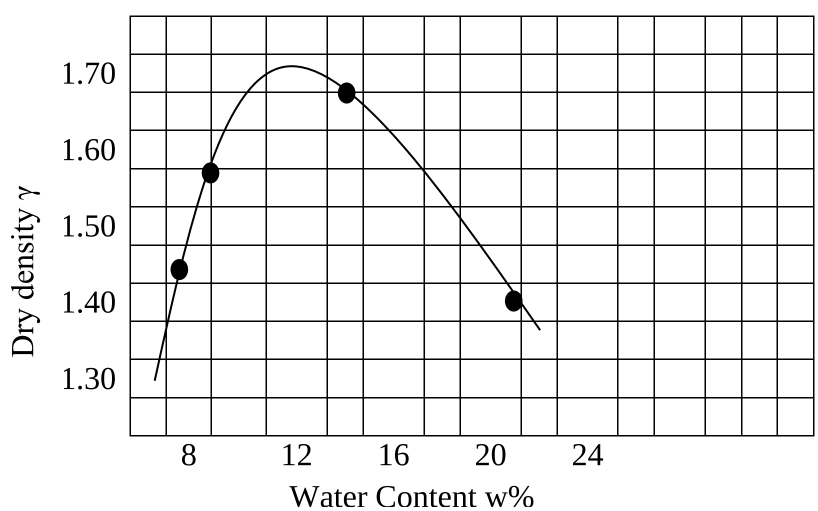
Location…Ogbomoso-ilorin highway……………

Tested by……Dindey Azeezat

Sample No……L1SP2………

SAMPLE NO	LOC 1 SP2							
BLOWS LAYER 25	NO OF LAYERS 3		WT OF HAMMER 2.5KG					
MOLDDIMMENSION: 10.5CM **AT** 11.5CM **VOL** 1000CM^3								
WATER CONTENT DETERMINATION								
Sample no	1		2		3		4	
Moisture can no								
Wt of wet soil + can	79.5	70.5	89.5	95.3	98.0	111.0	96.5	97.0
Wt of dry soil + can	74.0	65.5	82.5	88.1	87.0	99.5	83.5	83.0
Wt of Water	5.5	5.0	7.0	7.2	11.0	11.5	13.0	14.0
Wt of can	14.0	10.5	12.5	16.6	18.5	20.0	25.0	19.5
Wt of dry soil	60.0	55.0	70.0	71.5	68.5	77.5	58.5	63.5
Water content, w %	9.2	9.1	10.0	10.1	16.1	14.8	22.2	22.0

DENSITY DETERMINATION				
Assumed Water Content	9.0	10.0	15.5	22.0
Average Water Content w%(m)	9.2	10.1	15.5	22.1
Wt of soil + mold	3424	3559	3794	3708
Wt of mold	1854	1854	1854	1854
Weight of soil in mold	1570	1705	1940	1854
Wet Density, Dw	1.570	1.705	1.940	1.854
Dry Density Y = Dw 100/100 + m	1.44	1.55	1.68	1.52



Optimum moisture = 15.5%   Maximum dry density = 1.68g/cm³

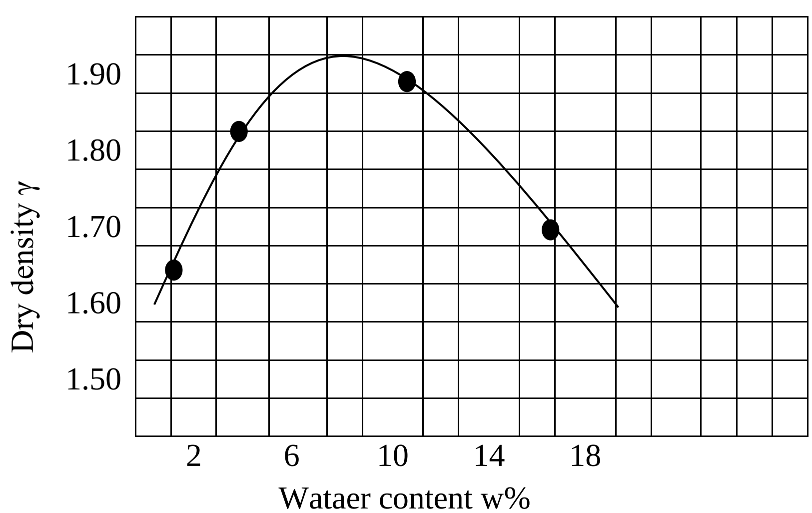
APPENDIX B Cont'd

Compaction Test

Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L2SP1………

SAMPLE NO	LOC 2 SP1							
BLOWS LAYER 25	NO OF LAYERS 3		WT OF HAMMER 2.5KG					
MOLDDIMMENSION: 10.5CM **AT** 11.5CM **VOL** 1000CM^3								
WATER CONTENT DETERMINATION								
Sample no	1		2		3		4	
Moisture can no								
Wt of wet soil + can	61.7	60.0	91.5	98.4	93.0	101.6	90.5	104.4
Wt of dry soil + can	60.7	59.0	88.0	94.4	85.0	92.6	78.5	91.4
Wt of Water	1.0	1.0	3.5	4.0	8.0	9.0	12.0	13.0
Wt of can	15.0	13.0	18.0	17.9	12.5	11.6	15.5	16.4
Wt of dry soil	45.7	46.0	70.0	76.5	72.5	81.0	63.0	75.0
Water content, w %	2.2	2.2	5.0	5.2	11.0	11.1	19.0	17.3

DENSITY DETERMINATION					
Assumed Water Content		2.0	5.0	11.0	18.0
Average Water Content w%(m)		2.2	5.1	11.1	18.2
Wt of soil + mold		3506	3744	3929	3825
Wt of mold		1854	1854	1854	1854
Weight of soil in mold		1652	1890	2075	1971
Wet Density, Dw		1.652	1.89	2.075	11.97
Dry Density Y = Dw 100/100 + m		1.62	1.80	1.87	1.67



Optimum moisture = 11.0%   Maximum dry density = 1.87g/cm³

APPENDIX B Cont'd

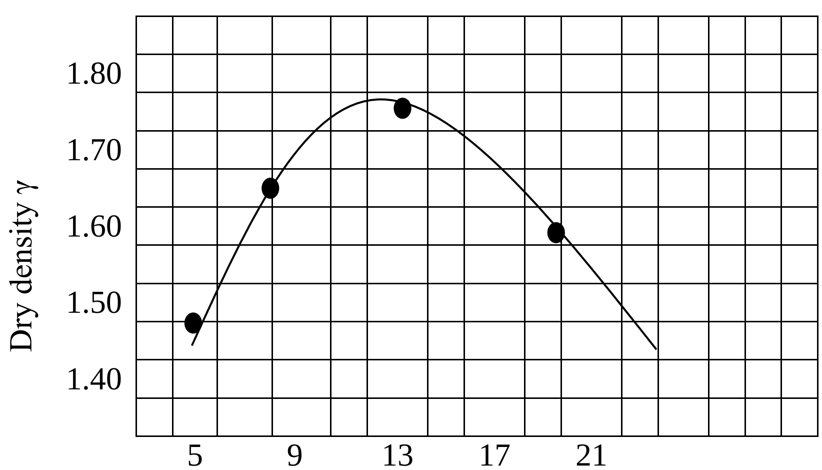
Compaction Test

Location…Ogbomoso-ilorin highway……………………….

Tested by……Dindey Azeezat

Sample No……L2SP2………

SAMPLE NO		LOC 2 SP2							
BLOWS LAYER 25		NO OF LAYERS 3		WT OF HAMMER 2.5KG					
MOLDDIMMENSION: 10.5CM **AT** 11.5CM **VOL** 1000CM^3									
WATER CONTENT DETERMINATION									
Sample no		1		2		3		4	
Moisture can no									
Wt of wet soil + can		74.0	78.2	90.2	9.7	101.5	95.7	107.0	111.6
Wt of dry soil + can		70.0	74.0	83.6	84.0	91.0	85.7	91.0	95.3
Wt of Water		4.0	4.2	6.6	6.7	10.5	10.0	16.0	16.3
Wt of can		3.5	7.0	11.6	12.0	16.0	14.9	15.0	17.8
Wt of dry soil		66.5	67.0	72.0	72.0	75.0	70.8	76.0	77.5
Water content, w %		6.0	6.3	9.2	9.3	14.0	14.1	21.1	21.0
DENSITY DETERMINATION									
Assumed Water Content			6.0	9.0	14.0	21.0			
Average Water Content w%(m)			6.2	9.3	14.1	21.1			
Wt of soil + mold			3444	3685	3883	3814			
Wt of mold			1854	1854	1854	1854			
Weight of soil in mold			1590	1831	2029	1960			
Wet Density, Dw			1.590	1.831	2.029	1.96			
Dry Density Y = Dw 100/100 + m			1.50	1.68	1.78	1.62			



Optimum moisture = 14.0%   Maximum dry density = 1.78g/cm³

APPENDIX C

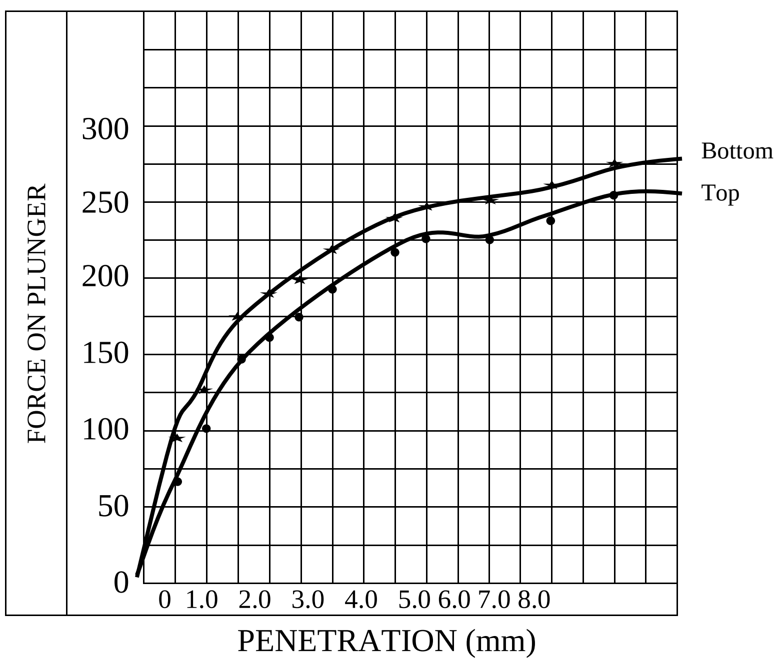
California Bearing Ratio (CBR TEST)

SAMPLE No……L1SP1……… NO OF LAYERS 3……;
BLOWS PER LAYER..25….
WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm..

BEARING VALUE AT
2.5mm…..45%...
5.0mm..37..%..
C.B.R VALUE…45%...

UNSOAKED ACCEPTED

	TOP		BOTTOM	
Penetration plunger(mm)	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	23	67	29.5	86
0.75				
1.00	34	100	39.5	116
1.50	46	135	50	147
2.00	53	155	61	179
2.50	61	179	67	196
3.00	65	190	73	214
4.00	70	205	78	229
5.00	74	217	83	243
6.00	80	234	87	255
7.00	83.5	245	94	275
8.00	87	255	97	284



APPENDIX C Cont'd

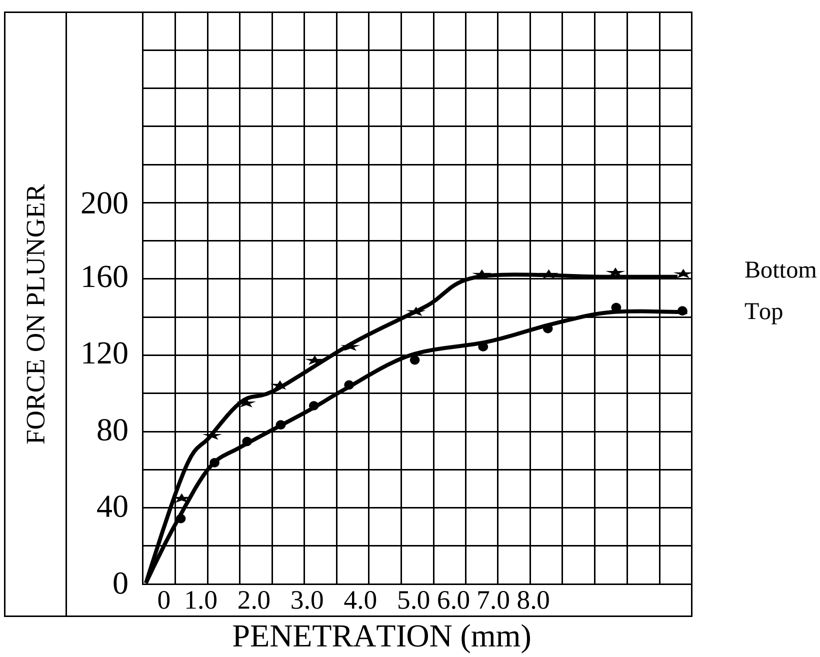
California Bearing Ratio(CBR Test)

Project….M.tech Project……………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L1SP1……… NO OF LAYERS 3……;
BLOWS PER LAYER..25….
WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm.

BEARING VALUE AT
2.5mm…..24%...
5.0mm….22%..
C.B.R VALUE…24%...

SOAKED ACCEPTED

	TOP		**BOTTOM**	
Penetration plunger(mm)	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	11	32	15	44
0.75				
1.00	19	56	25	73
1.50	24	70	30	88
2.00	27	79	34	100
2.50	31	91	38	111
3.00	35	103	42	123
4.00	39	114	47	138
5.00	42	123	50	147
6.00	44.5	130	53	155
7.00	48	141	55	161
8.00	49	144	57.5	169



APPENDIX C Cont'd

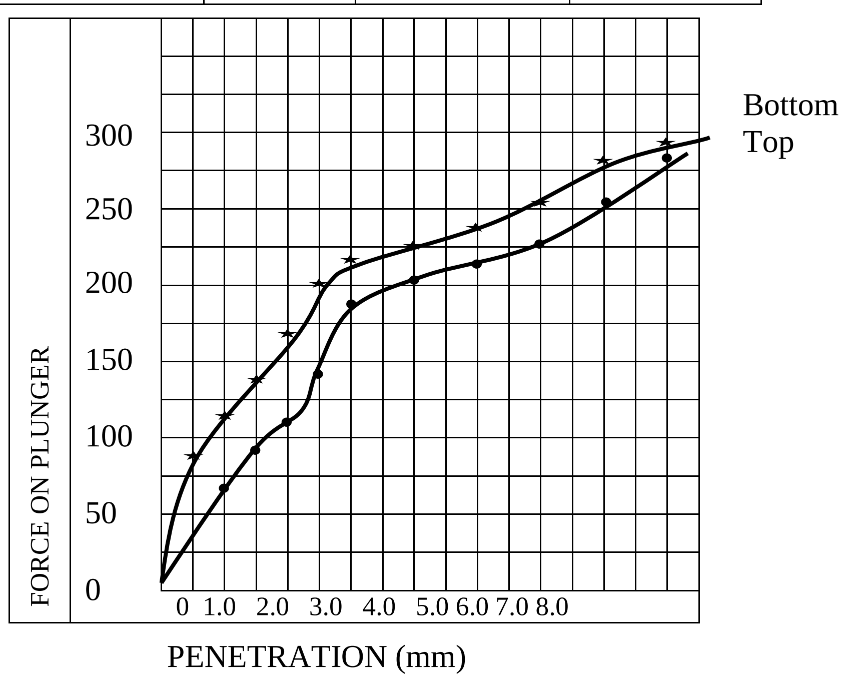
California Bearing Ratio(CBR TEST)

Project….M.tech Project……………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L1SP2……… NO OF LAYERS 3……;
BLOWS PER LAYER..25….
WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm..

BEARING VALUE AT
2.5mm…..42%...
5.0mm..35..%..

C.B.R VALUE…42%...UNSOAKED ACCEPTED

	TOP		**BOTTOM**	
Penetration plunger(mm)	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	19	56	26	76
0.75				
1.00	27	79	38	111
1.50	38	111	47	138
2.00	49	144	59	173
2.50	55.5	163	60.5	201
3.00	60	176	73	214
4.00	64	188	78	229
5.00	67	196	82	240
6.00	70	205	86	252
7.00	73	214	90	264
8.00	77	226	93.5	274



APPENDIX C Cont'd

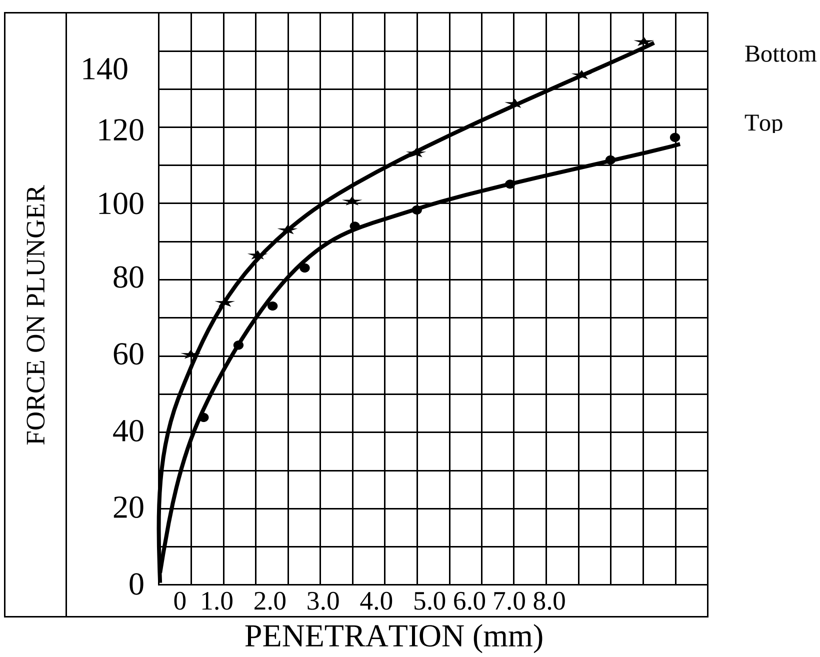
California Bearing Ratio(CBR TEST)

Project….M.tech Project……………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L1SP2……… NO OF LAYERS 3……;
BLOWS PER LAYER..25….
WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm..

BEARING VALUE AT
2.5mm…..20%...
5.0mm…..18..%..
C.B.R VALUE…20..%..

SOAKED ACCEPTED

	TOP		BOTTOM	
Penetration plunger(mm)	**SurchargeDial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	11	32	17	50
0.75				
1.00	16	47	24	70
1.50	20	59	27	79
2.00	23	67	29	85
2.50	26	76	31.8	93
3.00	29	85	34	100
4.00	32	94	38	111
5.00	34	100	42	123
6.00	36.5	107	45	132
7.00	38	111	47	138
8.00	40	117	50	147



APPENDIX C Cont'd

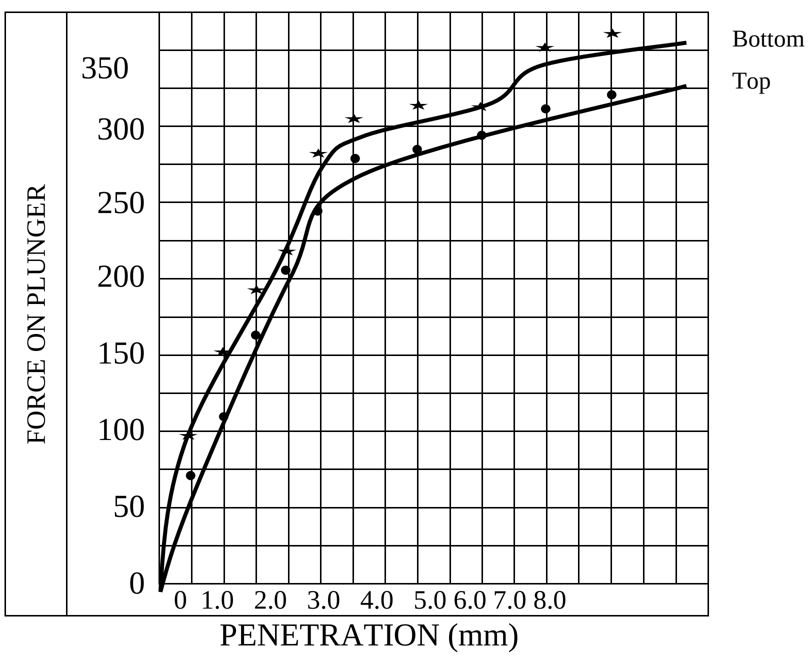
California Bearing Ratio(CBR TEST)

Project….M.tech Project……………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L2SP1……… NO OF LAYERS 3……;
BLOWS PER LAYER..25….
WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm..

BEARING VALUE AT
2.5mm…..62%...
5.0mm..50..%..
C.B.R VALUE…62%...

UNSOAKED ACCEPTED

	TOP		BOTTOM	
Penetration plunger(mm)	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	28	82	36.5	107
0.75				
1.00	40	117	52	152
1.50	56.5	166	66	193
2.00	72	211	80	234
2.50	85	249	95	278
3.00	91	267	101	296
4.00	95	278	107	314
5.00	100	293	115	337
6.00	105	308	119	349
7.00	110	322	124	363
8.00	113	331	127	372



APPENDIX C Cont'd

California Bearing Ratio(CBR TEST)

Project….M.tech Project……………………………………

Location…Ogbomoso-ilorin highway……………………….

Tested by……Dindey Azeezat

Sample No……L2SP1……… NO OF LAYERS 3……;

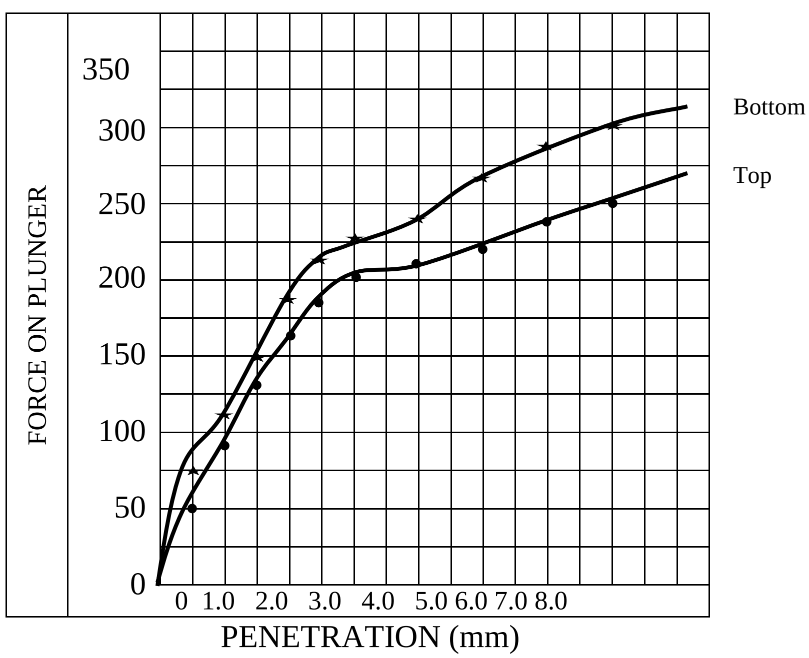
BLOWS PER LAYER..25….

WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm..

BEARING VALUE AT
2.5mm…..47%...
5.0mm..38..%..
C.B.R VALUE…47%...

SOAKED ACCEPTED

	TOP		BOTTOM	
Penetration plunger(mm)	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	18	53	27	79
0.75				
1.00	30	88	38	111
1.50	44	129	52	152
2.00	55	161	65	190
2.50	64.5	189	70.5	207
3.00	68	199	75	220
4.00	72	211	83	243
5.00	75	220	88	258
6.00	80	234	91	267
7.00	86	252	95	278
8.00	90	264	100	293



APPENDIX C Cont'd

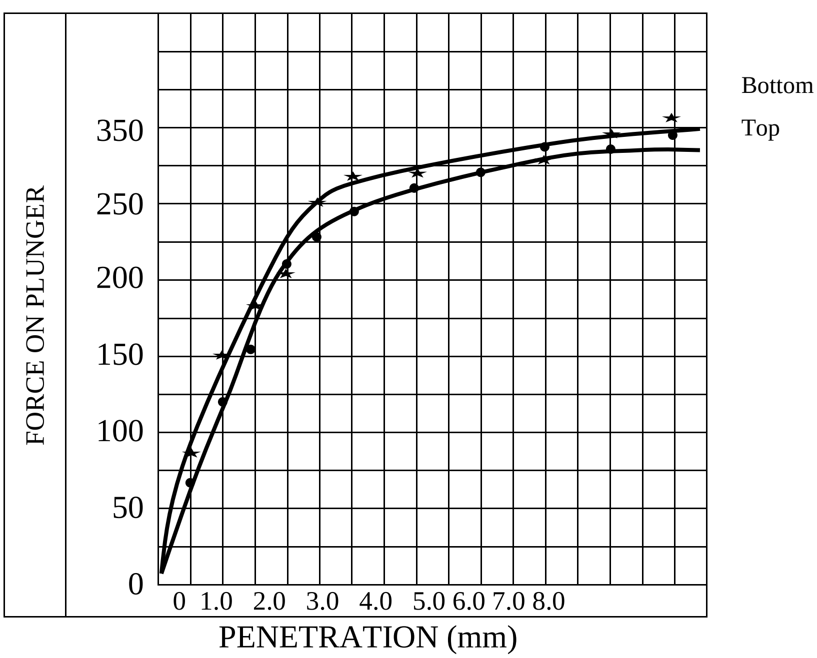
California Bearing Ratio(CBR TEST)

Project….M.tech Project……………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L2SP2……… NO OF LAYERS 3……;
BLOWS PER LAYER..25….
WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm..

BEARING VALUE AT
2.5mm…..57%...
5.0mm..46..%..
C.B.R VALUE…57%...

UNSOAKED ACCEPTED

	TOP		**BOTTOM**	
Penetration plunger(mm)	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	32	94	25	73
0.75				
1.00	50	147	41	120
1.50	62.5	183	56	129
2.00	71	208	70	205
2.50	78	229	86	252
3.00	84	246	93	272
4.00	90	264	98	287
5.00	96	281	103	302
6.00	100	293	107	314
7.00	107.5	315	113.5	333
8.00	114	334	121	355



APPENDIX C Cont'd

California Bearing Ratio(CBR TEST)

Project….M.tech Project……………………………………

Location…Ogbomoso-ilorin highway……………………….

Tested by……Dindey Azeezat

Sample No……L2SP2……… NO OF LAYERS 3……;

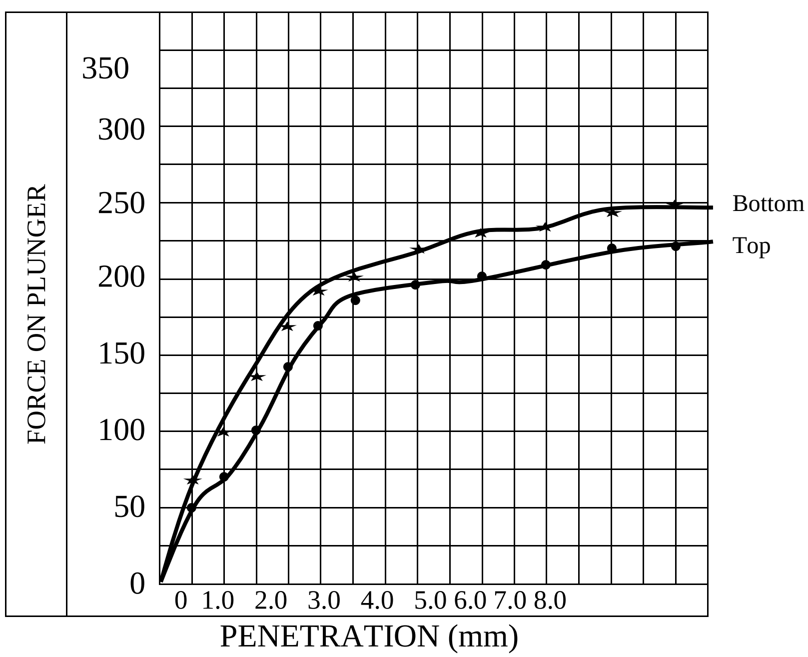
BLOWS PER LAYER..25….

WT OF RAMMER…4.5KG..; HT OF RAMMER..450mm..

BEARING VALUE AT
2.5mm…..42%...
5.0mm..34..%..
C.B.R VALUE…42%...

SOAKED ACCEPTED

	TOP		BOTTOM	
Penetration plunger(mm)	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**	**Surcharge Dial Reading*2.93**	**PISTON Load on plunger KN**
0.25				
0.50	18	53	24	70
0.75				
1.00	25	73	35	103
1.50	34	100	46	135
2.00	48	141	57	167
2.50	57	167	64	188
3.00	61	179	69	202
4.00	65	190	74	217
5.00	68	199	77	226
6.00	72	211	80	234
7.00	74	217	83	243
8.00	76	223	86	252



APPENDIX D

Shear Box Test

Project….M.tech Project……………………………………

Location…Ogbomoso-ilorin highway……………………….

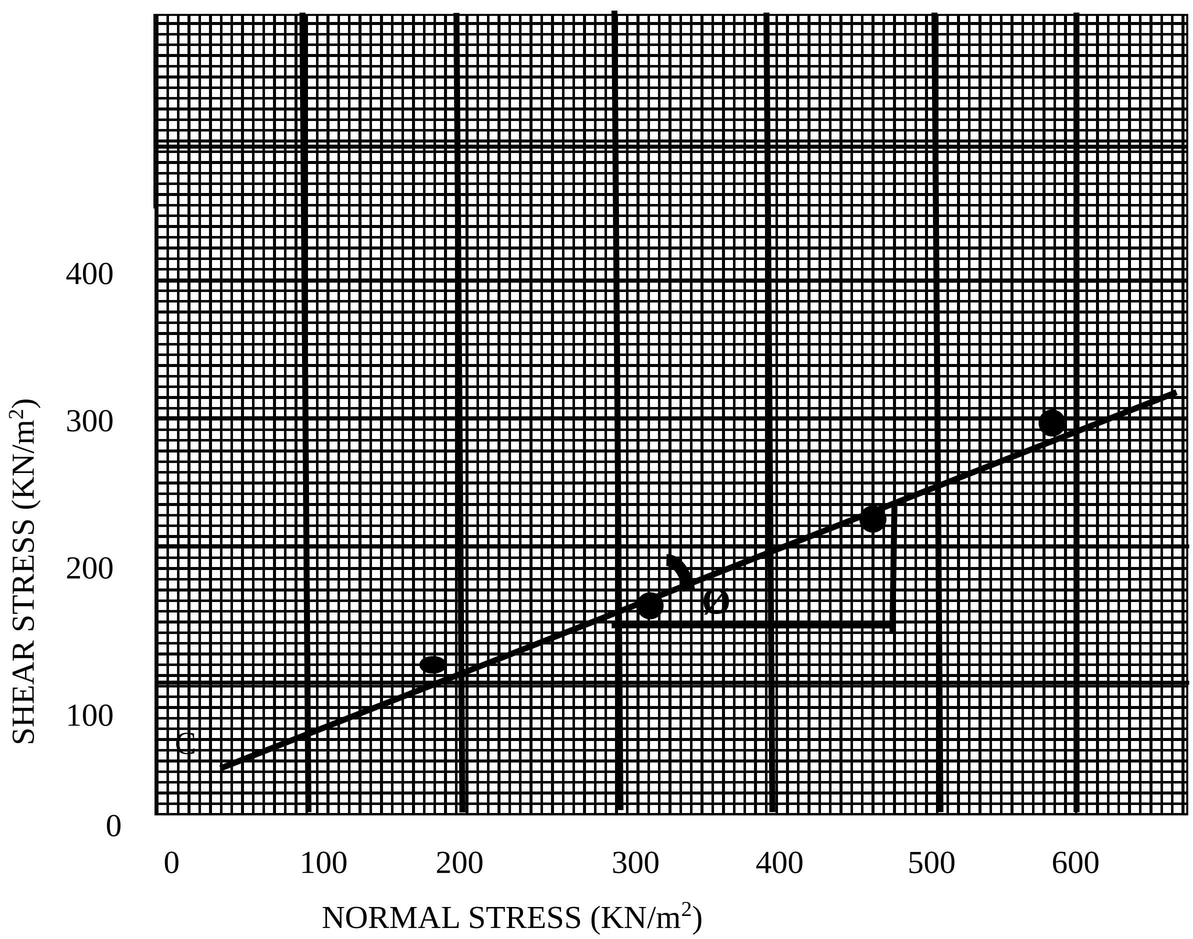
Tested by……Dindey Azeezat

Sample No……L1SP1………

Sample No……L1SP1

LOAD	NORMAL STRESS	LOAD DIAL	SHEAR STRESS
(KG)	**(KN/M^2)**	**READING**	**(KN/M^{-2})**
5	188.0	232	111.7
10	324.3	330	158.9
15	460.5	437	210.4
20	596.8	559	269.1

C =30KN/m² Ø = 22°



APPENDIX D Cont'd

Shear Box Test

Project….M.tech Project……………………………………

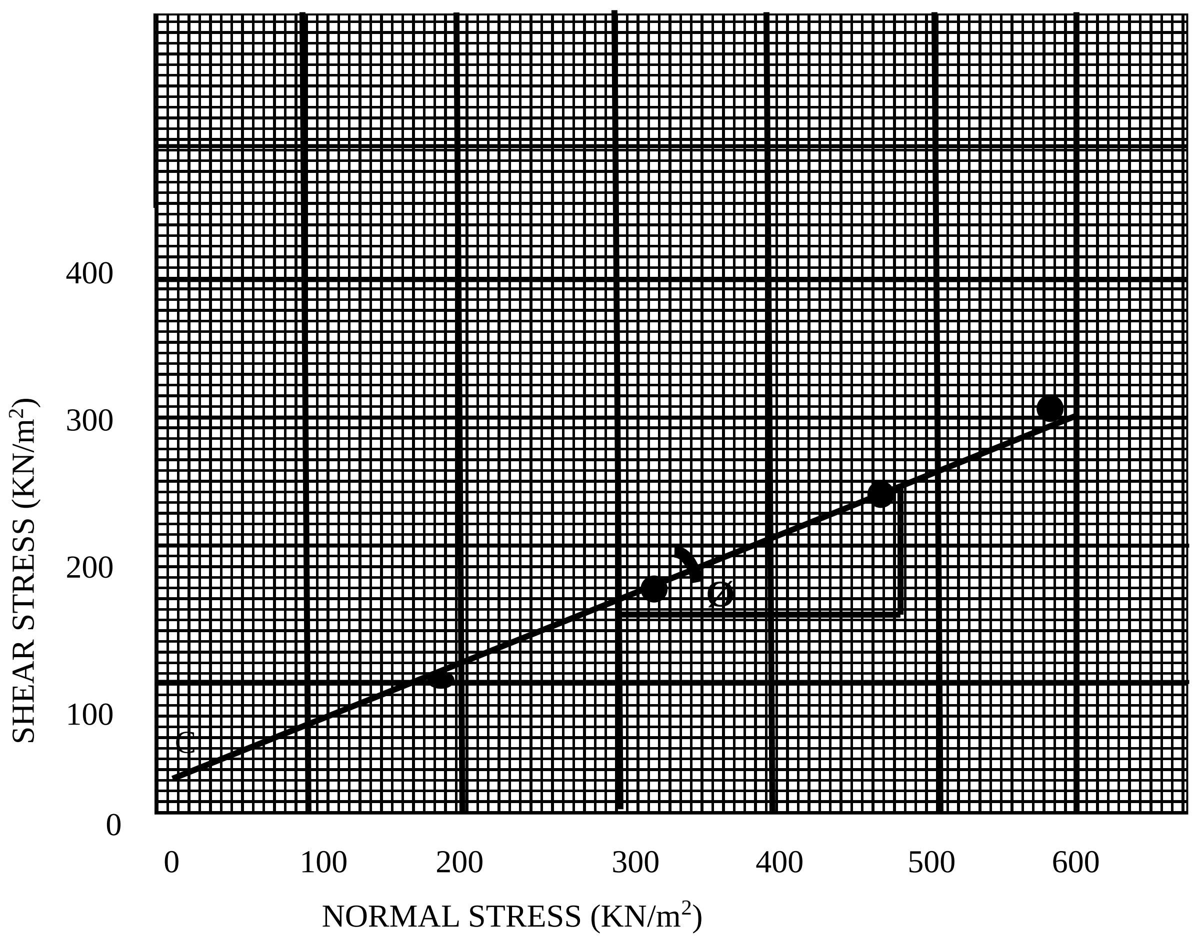
Location…Ogbomoso-ilorin highway……………………….

Tested by……Dindey Azeezat

Sample No……L1SP2………

LOAD	NORMAL STRESS	LOAD DIAL	SHEAR STRESS
(KG)	(KN/M^2)	READING	(KN/M^{-2})
5	188.0	211	101.6
10	324.3	335	161.3
15	460.5	457	263.3
20	596.8	582	280.2

$C = 20KN/m^2$ $Ø = 24°$



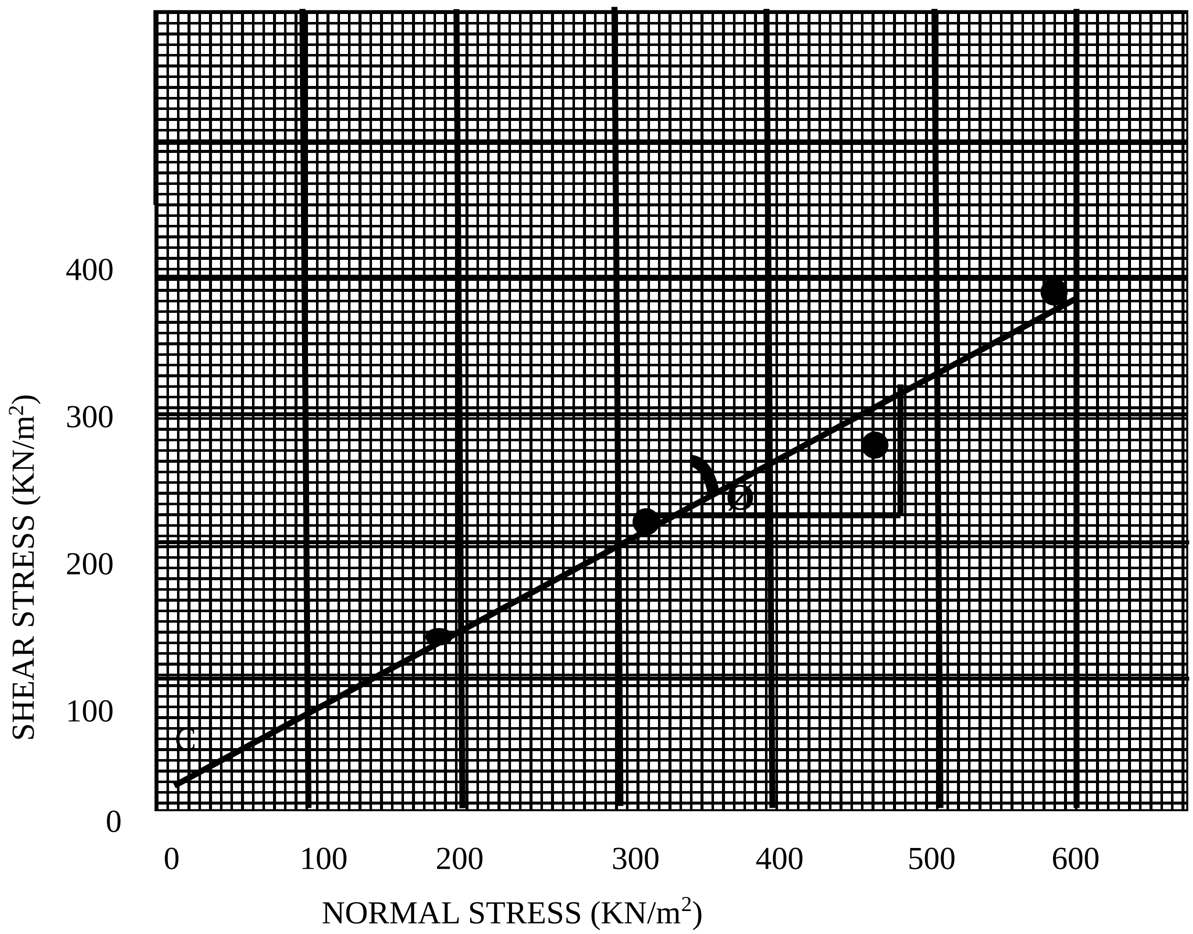
APPENDIX D Cont'd

Shear Box Test

Project….M.tech Project……………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L2SP1………

LOAD	NORMAL STRESS	LOAD DIAL	SHEAR STRESS
(KG)	(KN/M^2)	READING	(KN/M^{-2})
5	188.0	275	132.4
10	324.3	437	210.4
15	460.5	602	289.8
20	596.8	769	370.2

C = 10KN/m² **Ǿ = 32º**



APPENDIX D Cont'd

Shear Box Test

Project….M.tech Project……………………………………

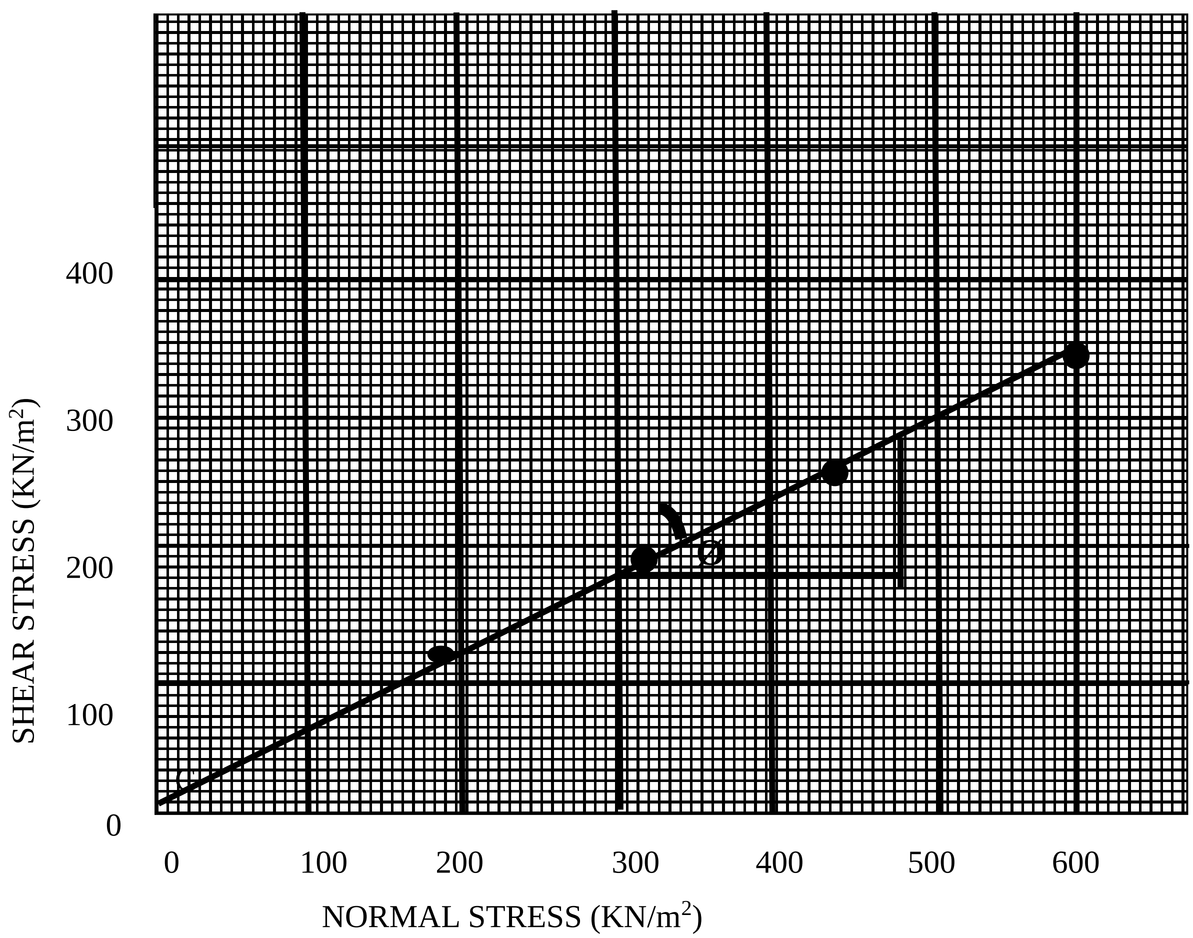
Location…Ogbomoso-ilorin highway……………………….

Tested by……Dindey Azeezat

Sample No……L2SP2………

LOAD	NORMAL STRESS	LOAD DIAL	SHEAR STRESS
(KG)	(KN/M^2)	READING	(KN/M^{-2})
5	188.0	249	119.9
10	324.3	396	190.6
15	460.5	542	260.9
20	596.8	709	341.3

C= 15KN/m² **Ǿ = 30º**



APPENDIX E

Consolidation Test

Sample No……L1SP2………

Void Ratio (e) …0.652…..; Change in void ratio…0.123………

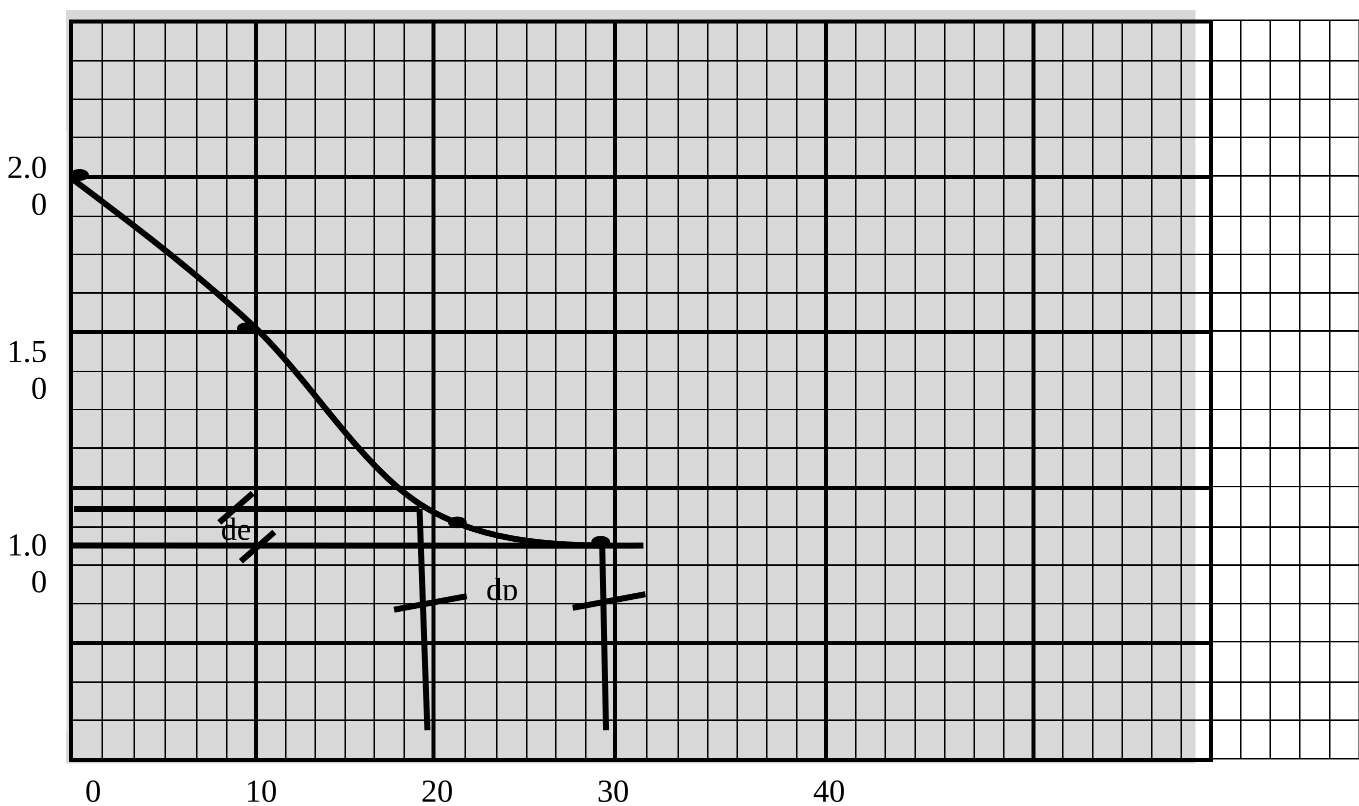
Sample thickness……13.41………

THICKNESS OF SOIL SAMPLE AFTER CONSOLIDATION

VERTICAL LOAD (Kg)	VERTICAL LOAD (KN/m²)	STRESS (KN/m²)	DIAL CHANGE READING AT 24Hrs	CHANGE IN THICKNESS AFTER CONSOLIDATION	THICKNESS OF SOIL CONSOLIDATION(mm)
5	0.049	11.10	120	1.20	18.80
10	0.098	22.20	224	2.24	16.56
15	0.147	33.30	315	3.15	13.41

THE VALUE OF THE VOID RATIO, e, AT THE END OF EACH CONSOLIDATION

PRESSURE (KN/m²)	H (mm)	Dh	De	E
0.00	20.00	6.59	0.811	2.034
11.10	18.80	3.44	0.423	1.223
` 22.20	16.56	1.20	0.148	0.800
33.30	13.41	0.00	0.000	0.652



CALCULATION FROM THE GRAPH

e at $20KN/m^2$……………………………….0.70

e at $30KN/m^2$……………………………….0.85

de = 0.85 – 0.70…………………………….0.15

dp = 30 – 20 ……………………………….$10KN/m^2$

COEFFICIENT OF COMPRESSIBILITY (av)

Av = de/dp

$0.15/10 = 0.015 = 1.5 * 10^{-2} m^2/KN$

$Av = 1.5 * 10^{-2} m^2/KN$

COEFFICIENT OF VOLUME COMPRESSIBILITY (Mv)
Mv = v/1+e

$1.5 * 10^{-2}/1.652 = 0.009 = 9 * 10^{-3} m^2/KN$

$Mv = 9 * 10^{-3} m^2/KN$

CONSOLIDATION SETTLEMENT (S)

S = mvDfdP

$S = 9 * 10^{-3} * 1.0 * 10 = 0.09m$

S = 9mm/m

APPENDIX E Cont’d

Consolidation Test

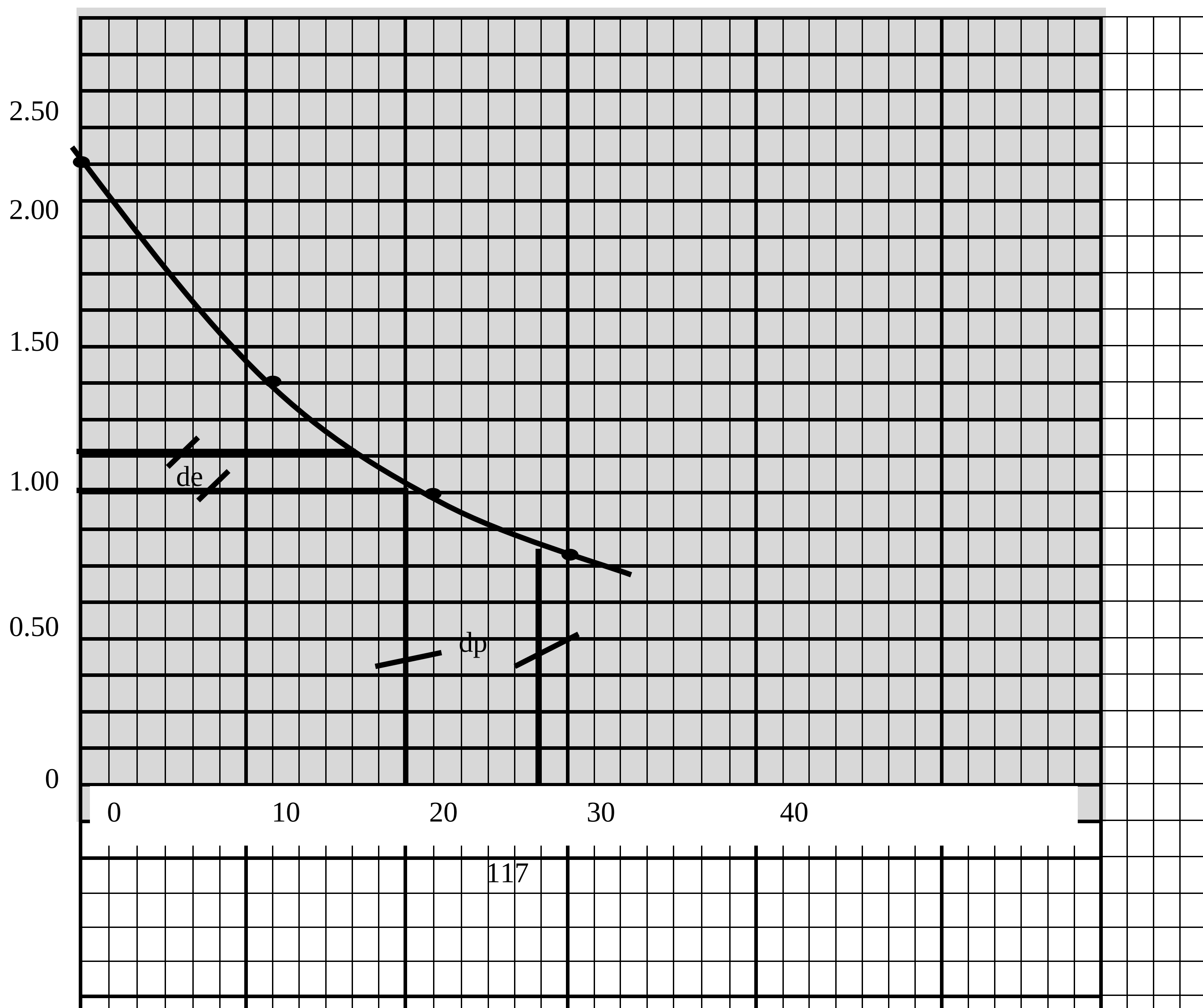
Tested by……Dindey Azeezat; Sample No……L2SP2………
Void Ratio (e) …0.804…..; Change in void ratio…0.132………
Sample thickness……13.67………

THICKNESS OF SOIL SAMPLE AFTER CONSOLIDATION

VERTICAL LOAD (Kg)	VERTICAL LOAD (KN/m^2)	STRESS (KN/m^2)	DIAL CHANGE READING AT 24Hrs	CHANGE IN THICKNESS AFTER CONSOLIDATION	THICKNESS OF SOIL CONSOLIDATION (mm)
5	0.049	11.10	116.5	1.17	18.83
10	0.098	22.20	217.0	2.17	16.66
15	0.147	33.30	298.5	2.99	13.67

THE VALUE OF THE VOID RATIO, e, AT THE END OF EACH CONSOLIDATION

PRESSURE (KN/m^2)	H (mm)	Dh	De	E
0.00	20.00	6.33	0.836	2.235
11.10	18.83	3.34	0.441	1.399
22.20	16.66	1.17	0.154	0.958
33.30	13.67	0.00	0.000	0.804



CALCULATION FROM THE GRAPH

e at 20KN/m²……………………………….0.85

e at 30KN/m²……………………………….1.05

de = 1.05 - 0.85……………………………..0.20

dp = 30 – 20 ……………………………….10KN/m²

COEFFICIENT OF COMPRESSIBILITY (av)

Av = de/dp

$0.20/10 = 0.02 = 2 * 10^{-2} m^2/KN$

COEFFICIENT OF VOLUME COMPRESSIBILITY (Mv)
Mv = v/1+e

$2 * 10^{-2}/1.804 = 0.011 = 1.1 * 10^{-2} m^2/KN$

CONSOLIDATION SETTLEMENT (S)

S = mvDfdP

S = $1.1 * 10^{-2} * 1.0 * 10 = 0.11$m

S = 11mm/m

APPENDIX E Cont'd

Consolidation Test

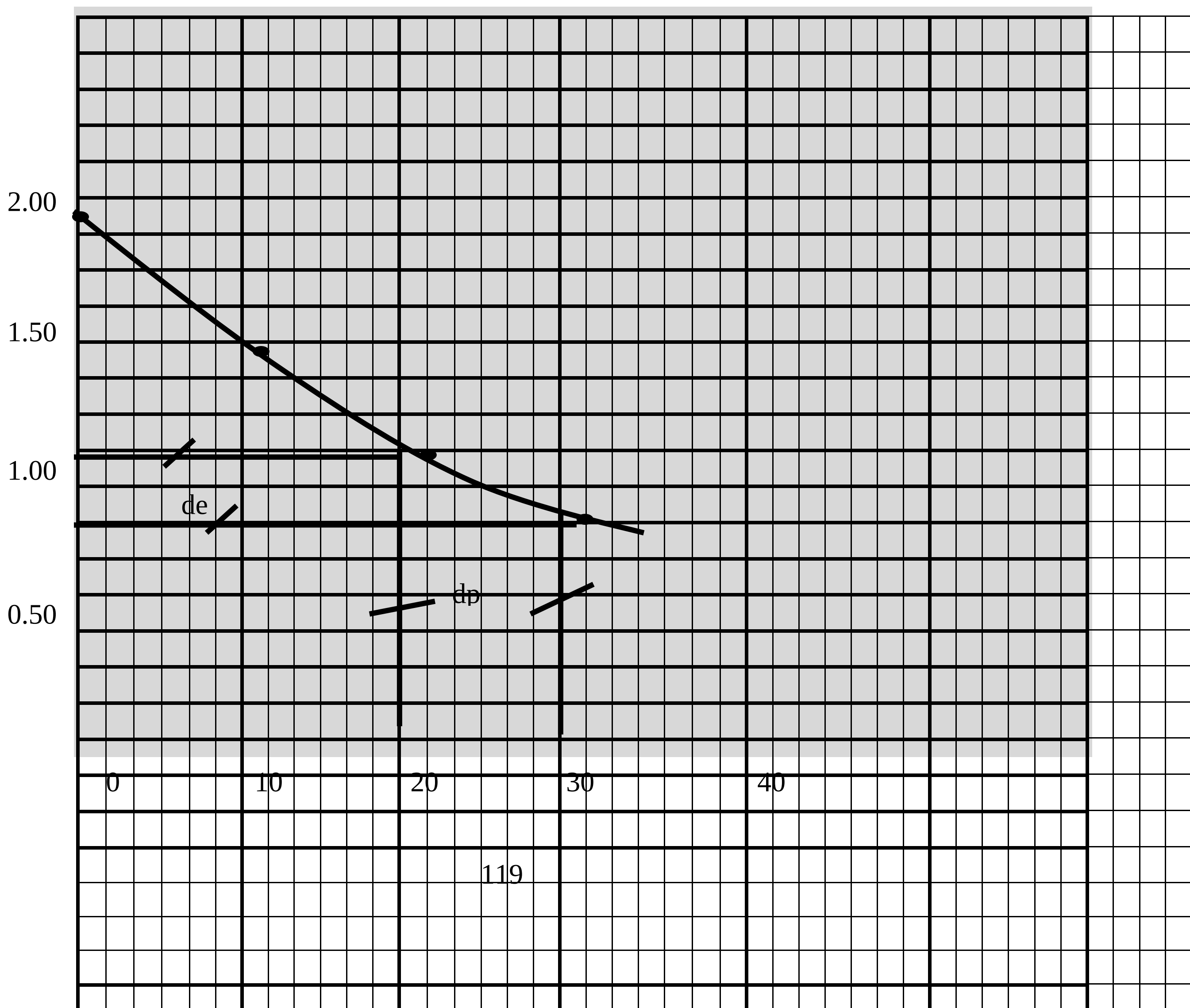
Tested by……Dindey Azeezat; Sample No……L3SP3………
Void Ratio (e) …0.833…..; Change in void ratio…0.122………
Sample thickness……15.01………

THICKNESS OF SOIL SAMPLE AFTER CONSOLIDATION

VERTICAL LOAD (Kg)	VERTICAL LOAD (KN/m²)	STRESS (KN/m²)	DIAL CHANGE READING AT 24Hrs	CHANGE IN THICKNESS AFTER CONSOLIDATION	THICKNESS OF SOIL CONSOLIDATION (mm)
5	0.049	11.10	96.0	0.96	19.04
10	0.098	22.20	158.0	1.58	17.46
15	0.147	33.30	245.0	2.45	15.01

THE VALUE OF THE VOID RATIO, e, AT THE END OF EACH CONSOLIDATION

PRESSURE (KN/m²)	H (mm)	Dh	De	E
0.00	20.00	4.99	0.609	1.869
11.10	19.04	2.54	0.310	1.260
22.20	17.46	0.96	0.117	0.950
33.30	15.01	0.00	0.000	0.833



CALCULATION FROM THE GRAPH

e at $20KN/m^2$……………………………….0.80

e at $30KN/m^2$……………………………….1.00

de = 1.00 - 0.80……………………………..0.20

dp = 30 – 20 ……………………………….$10KN/m^2$

COEFFICIENT OF COMPRESSIBILITY (av)

Av = de/dp

$0.20/10 = 0.02 = 2 * 10^{-2} m^2/KN$

COEFFICIENT OF VOLUME COMPRESSIBILITY (Mv)
Mv = v/1+e

$2 * 10^{-2}/1.833 = 0.011 = 1.1 * 10^{-2} m^2/KN$

CONSOLIDATION SETTLEMENT (S)

S = mvDfdP

S = $1.1 * 10^{-2} * 1.0 * 10 = 0.11m$

S = 11mm/m

APPENDIX F

NATURAL MOISTURE CONTENTS

The natural moisture contents determination shows the percentage (%) of water contained in the soil sample from L1SP1 – L8SP3 (APPENDIX F).

	LOC 1		LOC 2		
Boring No	1	2	1	2	3
Container No (cup)	1A	C3	F19	2D	J
Wt of cup+ wet soil	416.0	421.0	464.5	432.0	437.0
Wt of cup+ dry soil	357.5	364.8	426.5	395.0	406.0
Wt of cup	27.5	31.0	19.5	22.0	28.5
Wt of dry soil	330.0	333.8	407.0	373.0	377.5
Wt of water	58.5	56.2	38.0	37.0	31.0
Water content%	17.7	16.8	9.3	9.9	8.2

	LOC2/4	LOC3			
Boring No	4	1	2	3	4
Container No (cup)	M	N	H	Ti	K
Wt of cup+ wet soil	413.0	454.5	477.5	437.8	407.1
Wt of cup+ dry soil	386.9	416.5	432.0	378.8	396.1
Wt of cup	12.9	29.0	36.0	30.8	44.6
Wt of dry soil	374.0	387.5	396.0	348.0	351.0
Wt of water	27.0	38.0	45.5	59.0	11.0
Water content%	7.2	9.8	11.5	17.0	3.1

	LOC 3	LOC 4			
Boring No	5	1	2	3	4
Container No (cup)	R	E	W	A	JI
Wt of cup+ wet soil	422.8	423.0	460.0	471.0	436.0
Wt of cup+ dry soil	394.8	382.0	415.0	432.0	386.5
Wt of cup	26.0	35.0	62.5	74.0	37.5
Wt of dry soil	368.8	347.0	352.5	358.0	349.0
Wt of water	28.0	41.0	45.0	39.0	49.5
Water content%	7.6	11.8	12.8	10.9	14.2

APPENDIX F Cont'd

LOC 5

Boring No	1	2	3
Container No (cup)	U3	X4	O2
Wt of cup+ wet soil	435.5	428.3	438.5
Wt of cup+ dry soil	410.5	399.3	405.0
Wt of cup	62.5	43.8	45.0
Wt of dry soil	348.0	355.5	360.0
Wt of water	25.0	29.0	33.5
Water content%	7.2	8.2	9.3

APPENDIX F

LOC 6

Boring No	1	2	3
Container No (cup)	W5	V7	E6
Wt of cup+ wet soil	451.8	403.5	444.4
Wt of cup+ dry soil	425.8	381.0	414.4
Wt of cup	74.0	36.0	55.5
Wt of dry soil	351.8	345.0	358.9
Wt of water	26.0	22.5	30.0
Water content%	7.4	6.5	8.4

LOC 8

Boring No	1	2	3	4
Container No (cup)	P3	N7	M9	L
Wt of cup+ wet soil	428.5	422.4	409.9	411.5
Wt of cup+ dry soil	403.0	401.1	397.5	370.5
Wt of cup	15.0	24.9	18.0	25.5
Wt of dry soil	388.0	376.5	379.5	345.0
Wt of water	25.5	21.0	12.0	41.0
Water content%	6.6	5.6	3.2	11.9

APPENDIX G

Bulk Density

The bulk density determination of the soil samples L1SP1 – L8SP3 are shown in APPENDIX G.

	LOC 1				**LOC 2**	
Sample Label	1	2	1	2	3	4
WT OF MOULD + WET SOIL (g)	571.8	573.5	628.5	593.5	592.0	584.5
WT OF MOULD	183.5	183.5	183.5	183.5	183.5	183.5
WT OF WET SOIL	388.3	390.5	445.0	410.0	408.5	401.0
VOLUME OF MOULD (cm^3)	251.4	251.4	251.4	251.4	251.4	251.4
BULK DENSITY (g/cm^3)	1.54	1.55	1.77	1.63	1.62	1.60
WT OF PAN+ DRY SOIL (g)	730.0	730.5	807.0	773.0	777.5	774.0
WT OF PAN(g)	400.0	400.0	400.0	400.0	400.0	400.0
WT OF DRY SOIL	330.0	330.0	407.0	373.0	377.5	374.0
VOLUME OF MOULD(cm^3)	251.4	251.4	251.4	251.4	251.4	251.4
DRY DENSITY(g/cm^3)	1.31	1.33	1.62	1.48	1.50	1.49

APPENDIX G Cont'd

LOC 3

SAMPLE LABEL	1	2	3	4	5
WT OF MOULD + WET SOIL (g)	609.0	625.0	590.5	546.5	580.3
WT OF MOULD	183.5	183.5	183.5	183.5	183.5
WT OF WET SOIL	425.5	441.5	407.5	362.5	396.8
VOLUME OF MOULD (cm³)	251.4	251.4	251.4	251.4	251.4
BULK DENSITY (g/cm³)	1.69	1.76	1.62	1.44	1.58
WT OF PAN+ DRY SOIL	787.5	796.0	748.0	751.5	768.8
WT OF PAN	400.0	400.0	400.0	400.0	400.0
WT OF DRY SOIL	387.5	396.0	348.0	351.0	368.8
VOLUME OF MOULD(cm³)	251.4	251.4	251.4	251.4	251.4
DRY DENSITY(g/cm³)	1.54	1.58	1.38	1.40	1.47

LOC 4

Sample Label	1	2	3	4
WT OF MOULD + WET SOIL (g)	571.5	581.0	580.5	580.0
WT OF MOULD	183.5	183.5	183.5	183.5
WT OF WET SOIL	388.0	397.5	397.0	396.5
VOLUME OF MOULD (cm³)	251.4	251.4	251.4	251.4
BULK DENSITY (g/cm³)	1.54	1.58	1.58	1.58
WT OF PAN+ DRY SOIL	747.0	752.5	758.0	749.0
WT OF PAN	400.0	400.0	400.0	400.0
WT OF DRY SOIL	347.5	352.5	358.0	349.0
VOLUME OF MOULD(cm³)	251.4	251.4	251.4	251.4
DRY DENSITY(g/cm³)	1.38	1.40	1.42	1.39

APPENDIX G Cont'd

LOC 5

Sample Label	1	2	3
WT OF MOULD + WET SOIL (g)	556.0	567.5	576.5
WT OF MOULD(g)	183.5	183.5	183.5
WT OF WET SOIL(g)	373.0	384.5	393.5
VOLUME OF MOULD (cm^3)	251.4	251.4	251.4
BULK DENSITY (g/cm^3)	1.48	1.53	1.57
WT OF PAN+ DRY SOIL (g)	615.0	622.5	627.0
WT OF PAN(g)	267.0	267.0	267.0
WT OF DRY SOIL	348.0	355.0	360.0
VOLUME OF MOULD(cm^3)	251.4	251.4	251.4
DRY DENSITY(g/cm^3)	1.38	1.41	1.43

LOC 6

Sample Label	1	2	3
WT OF MOULD + WET SOIL (g)	560.8	550.5	571.9
WT OF MOULD(g)	183.5	183.5	183.5
WT OF WET SOIL(g)	377.8	367.5	388.9
VOLUME OF MOULD (cm^3)	251.4	251.4	251.4
BULK DENSITY (g/cm^3)	1.50	1.46	1.55
WT OF PAN+ DRY SOIL (g)	618.8	612.0	625.9
WT OF PAN(g)	267.0	267.0	267.0
WT OF DRY SOIL	351.8	345.0	358.9
VOLUME OF MOULD(cm^3)	251.4	251.4	251.4
DRY DENSITY(g/cm^3)	1.40	1.37	1.43

APPENDIX G Cont'd

LOC 8

Sample Label	1	2	3	4
WT OF MOULD + WET SOIL (g)	597.0	581.0	575.0	569.5
WT OF MOULD(g)	183.5	183.5	183.5	183.5
WT OF WET SOIL(g)	413.5	397.5	391.5	386.0
VOLUME OF MOULD (cm^3)	251.4	251.4	251.4	251.4
BULK DENSITY (g/cm^3)	1.64	1.58	1.56	1.54
WT OF PAN+ DRY SOIL (g)	788.0	776.5	779.5	745.0
WT OF PAN(g)	400.0	400.0	400.0	400.0
WT OF DRY SOIL	388.0	376.0	379.5	345.0
VOLUME OF MOULD(cm^3)	251.4	251.4	251.4	251.4
DRY DENSITY(g/cm^3)	1.54	1.50	1.51	1.37

APPENDIX H

SPECIFIC GRAVITY

LOC 1 **LOC 2**

SAMPLE LABEL	1	2	1	2	3	4
WT OF EMPTY BOTTLE (W1)	26.0	26.0	26.0	26.0	26.0	26.0
WT OF EMPTY BOTTLE+ 1/3 OF SOIL (W2)(G)	48.0	49.0	48.5	50.0	49.5	52.0
WT OF EMPTY BOTTLE+1/3 SOIL+WATER(W3)	87.5	88.0	88.0	89.0	88.6	90.0
WT OF EMPTY BOTTLE+WATER ONLY(W4)	74.0	74.0	74.0	74.0	74.0	74.0
SPECIFIC GRAVITY=W2-W1/(W4-W1)-(W3-W2)	2.59	2.56	2.65	2.67	2.64	2.60

LOC 3

SAMPLE LABEL	1	2	3	4	5
WT OF EMPTY BOTTLE (W1)	26.0	26.0	26.0	26.0	26.0
WT OF EMPTY BOTTLE+ 1/3 OF SOIL (W2)(G)	49.6	50.0	47.5	51.0	49.0
WT OF EMPTY BOTTLE+1/3 SOIL+WATER(W3)	88.6	89.0	87.0	89.8	88.3
WT OF EMPTY BOTTLE+WATER ONLY(W4)	74.0	74.0	74.0	74.0	74.0
SPECIFIC GRAVITY=W2-W1/(W4-W1)-(W3-W2)	2.62	2.67	2.53	2.72	2.64

LOC 4

SAMPLE LABEL	1	2	3	4
WT OF EMPTY BOTTLE (W1)	26.0	26.0	26.0	26.0
WT OF EMPTY BOTTLE+ 1/3 OF SOIL (W2)(g)	49.9	48.0	52.0	50.0
WT OF EMPTY BOTTLE+1/3 SOIL+WATER(W3)	88.4	87.5	89.8	88.4
WT OF EMPTY BOTTLE+WATER ONLY(W4)	74.0	74.0	74.0	74.0
SPECIFIC GRAVITY=W2-W1/(W4-W1)-(W3-W2)	2.52	2.59	2.55	2.50

APPENDIX H Cont'd

LOC 6

SAMPLE LABEL	1	2	3	4
WT OF EMPTY BOTTLE (W1)	26.0	26.0	26.0	26.0
WT OF EMPTY BOTTLE+ 1/3 OF SOIL (W2)(g)	47.5	48.5	52.0	51.0
WT OF EMPTY BOTTLE+1/3 SOIL+WATER(W3)	87.5	88.0	90.5	89.6
WT OF EMPTY BOTTLE+WATER ONLY(W4)	74.0	74.0	74.0	74.0
SPECIFIC GRAVITY=W2-W1/(W4-W1)-(W3-W2)	2.69	2.65	2.74	2.66

LOC 7

SAMPLE LABEL	1	2	3
WT OF EMPTY BOTTLE (W1)	26.0	26.0	26.0
WT OF EMPTY BOTTLE+ 1/3 OF SOIL (W2)(g)	49.0	48.5	48.0
WT OF EMPTY BOTTLE+1/3 SOIL+WATER(W3)	88.0	87.5	87.5
WT OF EMPTY BOTTLE+WATER ONLY(W4)	74.0	74.0	74.0
SPECIFIC GRAVITY= W2-W1/(W4-W1)-(W3-W2)	2.56	2.50	2.59

LOC 8

SAMPLE LABEL	1	2	3
WT OF EMPTY BOTTLE (W1)	26.0	26.0	26.0
WT OF EMPTY BOTTLE+ 1/3 OF SOIL (W2)(g)	49.5	48.5	47.5
WT OF EMPTY BOTTLE+1/3 SOIL+WATER(W3)	88.5	88.0	87.5
WT OF EMPTY BOTTLE+WATER ONLY(W4)	74.0	74.0	74.0
SPECIFIC GRAVITY=W2-W1/(W4-W1)-(W3-W2)	2.61	2.65	2.69

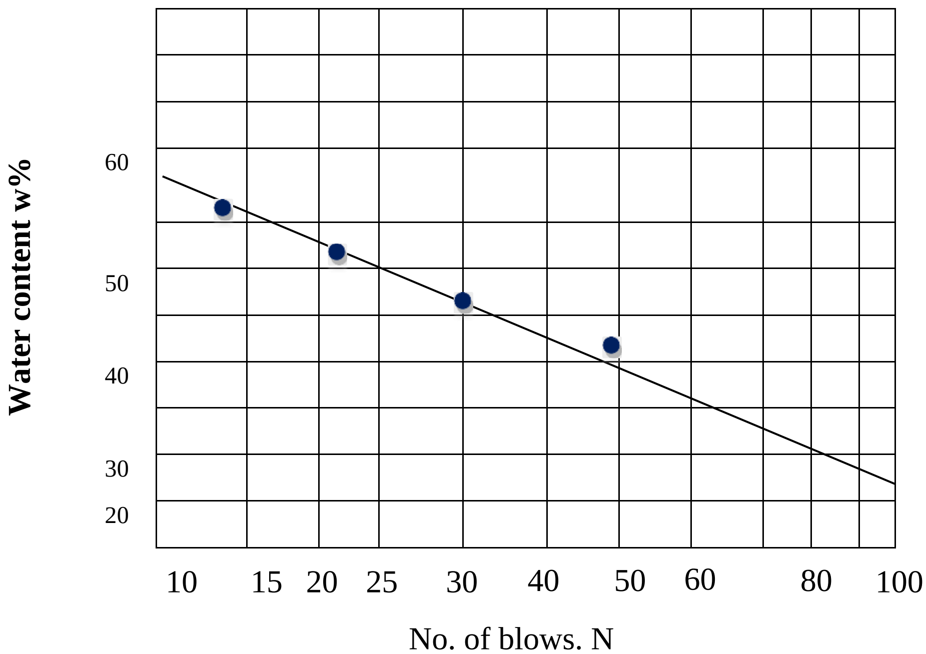
APPENDIX I

Atterberg Limit Determination

Project….M.tech Project…………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L1SP1

Liquiq Limit

Can No	**B**	**3**	**6**	**7**
Wt of wet soil + Can	35.1	30.2	32.4	27.8
Wt of Dry soil+ Can	29.0	24.6	27.4	22.3
Wt of can	14.5	13.0	17.9	12.8
Wt of Dry soil	14.5	11.6	9.5	9.5
Wt of Moisture	6.1	5.6	5.0	5.5
Water content, w%	42.1	47.4	52.6	57.9
No of Blows, N	50	30	22	14



Flow index F1 = <u>-28.7%</u>
Liquid limit = <u>50.0%</u>
Plastic limit = <u>22.1%</u>
Plasticity index Ip=<u>27.9%</u>
Toughness index= 1.0%

PLASTIC LIMIT

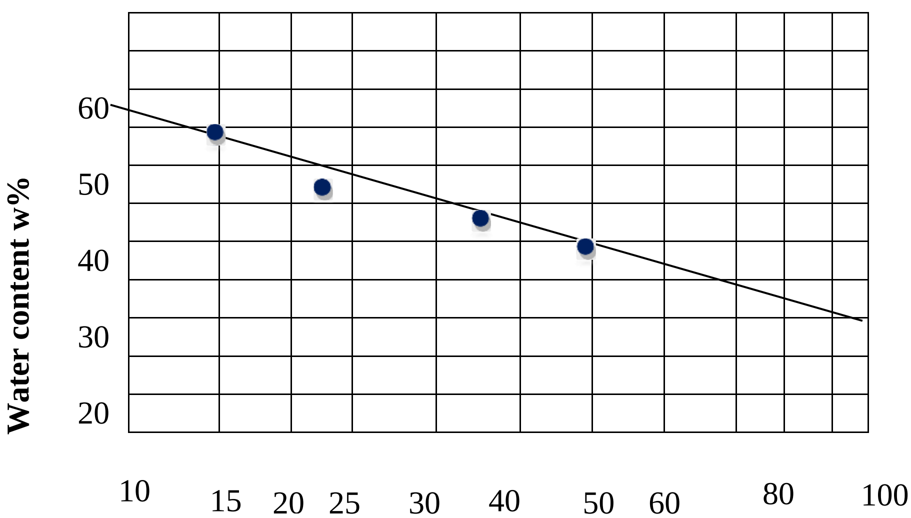
Can No	W3	14	E4	G5
Wt of wet soil + Can	20.2	24.8	20.3	17
Wt of Dry soil+ Can	18.5	22.8	18.7	16
Wt of can	10.5	14	11.5	11.5
Wt of Dry soil	8.0	8.8	7.2	4.5
Wt of Moisture	1.7	2.0	1.6	1.0
Water content, w%	21.3	22.7	22.2	22.2

APPENDIX I Cont'd

Atterberg Limit Determination
Project….M.tech Project……………………………………
Location…Ogbomoso-ilorin highway……………………….
Tested by……Dindey Azeezat
Sample No……L2SP2………

LIQUID LIMIT

Can No	M1	M2	M3	M4
Wt of wet soil + Can	34.0	31.2	29.4	37.1
Wt of Dry soil+ Can	29.0	26.0	23.0	30.6
Wt of can	18.0	15.5	11.0	19.6
Wt of Dry soil	11.0	10.5	12.0	11.0
Wt of Moisture	5.0	5.2	6.4	6.5
Water content, w%	45.5	49.5	53.3	59.1
No of Blows, N	50	35	23	15



Flow index F1 = -26.2%
Liquid limit = 52.5%
Plastic limit = 26.5%
Plasticity index Ip=26.0%
Toughness index= 1.0%

PLASTIC LIMIT

Can No	MS	M6	M7	M8
Wt of wet soil + Can	15.2	27.0	21.5	25.5
Wt of Dry soil+ Can	13.0	24.5	19.5	22.5
Wt of can	4.5	13.0	12.0	11.5
Wt of Dry soil	8.5	9.5	7.5	11.0
Wt of Moisture	2.2	2.5	2.0	3.0
Water content, w%	25.7	26.3	26.7	27.3

APPENDIX I Cont'd

Linear Shrinkage Limit

Sample label	L1SP1	L1SP2	L2SP1	L2SP2	L2SP3	L2SP4
Original length of sample(cm)	18.0	18.4	18.0	18.0	18.2	18.4
Final length of sample(cm)	15.7	15.9	17.2	16.3	16.7	17.8
Change in length(cm)	2.3	2.5	0.8	1.7	1.5	0.6
Shrinkage limit,%	12.8	13.6	4.4	9.4	8.2	3.3

Sample label	L3SP1	L3SP2	L3SP3	L3SP4	L3SP5
Original length of sample(cm)	18.4	18.0	18.4	18.0	18.2
Final length of sample(cm)	16.6	16.1	15.8	17.1	17.0
Change in length(cm)	1.8	1.9	2.6	0.9	1.2
Shrinkage limit,%	9.8	10.6	14.1	5.0	6.6

Sample label	L4SP1	L4SP2	L4SP3	L4SP4	L6SP1	L6SP2
Original length of sample(cm)	18.2	18.6	18.2	18.6	18.4	18.2
Final length of sample(cm)	17.2	17.6	17.1	17.6	17.8	17.6
Change in length(cm)	1.0	1.0	1.1	1.0	0.6	0.6
Shrinkage limit,%	5.5	5.4	6.0	5.4	3.3	3.3

Sample label	L6SP3	L6SP4	L7SP1	L7SP2	L7SP3
Original length of sample(cm)	18.0	18.0	18.4	18.6	18.2
Final length of sample(cm)	17.0	16.0	16.4	16.5	16.4
Change in length(cm)	1.0	2.0	2.0	2.1	1.8
Shrinkage limit,%	5.6	11.1	10.9	11.3	9.9

Sample label	L8SP1	L8SP2	L8SP3
Original length of sample(cm)	18.4	18.6	18.2
Final length of sample(cm)	16.9	17.0	16.5
Change in length(cm)	1.5	1.6	1.7
Shrinkage limit,%	8.2	8.6	9.3