LIGHTLY LOADED FOUNDATION FAILURES AT SOKOTO

- INVESTIGATIONS AND SOLUTIONS -

BY

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A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, AHMADU BELLO UNIVERSITY, IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF MASTER OF SCIENCE DEGREE IN CIVIL ENGINEERING CONSTRUCTION MATERIALS.

DEPARTMENT OF CIVIL ENGINEERING

FACULTY OF ENGINEERING

AHMADU BELLO UNIVERSITY ZAIRA.

OCTOBER, 1989.

DECLARATION

I hereby declare that this thesis is a record of my own research work. It has not been presented by me in any previous application for a higher degree.

E.C. Onyeiwu



DEDICATION

This work is dedicated to the pursuit of excellence in engineering practice.

CERTIFICATION

This Thesis entitled Design Criteria for Lightly Loaded Foundations on Collapsible Soils - The Sokoto Experience by Eric C. Onyeiwu meets the regulations governing the award of the degree of Master of Science (Civil Engineering Construction Materials) of Ahmadu Bello University, and is approved for its contribution to knowledge and literary presentation.

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ACKNOWLEDGEMENT

I hereby express my gratitude to my supervisors Dr. O. Adebisi and Mr. B.K. Yanney for their assistance and encouragement all through the period of this work. In particular, I wish to state that the personal attention and scrutiny which this work underwent with Dr. O. Adebisi was a source of encouragement to me. I consider myself privileged to have worked with him.

I am also grateful to the Sokoto Rima River Basin Development Authority for permission to carry out excavations within their premises. Their willingness to provide me with labourers and excavation tools are highly appreciated. Many thanks are due to the technical staff of Soil Mechanics Laboratory and in particular Mr. Okereke Ariba for their assistance all through the period of laboratory research.

I also wish to acknowledge the assistance of Mr. D.P. Katale who went through the manuscript and made very relevant criticisms. Particular thanks is due to my wife for her support and encouragement all through the period of this course.

Finally, I wish to express my sincere appreciation to the Department of Civil Engineering Ahmadu Bello University Zaira for the opportunity to have embarked on this unique course.

E.C. Onyeiwu.

ABSTRACT

The causes of severe cracks on lightly loaded buildings within the Sokoto Rima River Basin Development Authority Heaaquarters Complex have been found to be due to inadequate foundation aesign. The soil within the complex is founa to be predominantly Reddish Brown Well Graded Clayey Gravel exhibiting collapsing characteristics. The soil has been shown to exhibit large settlement and reduction in bearing capacity on inundating with water. It has been shown that while reduction in bearing capacity is observed as moisture content is increased, the foundation failures observed were mainly, due to large excessive total and differential settle-ments. It has also been observed that the strip type of foundation used for most lightly loaded buildings on the site, lacked the rigidity needed for foundation on collapsible soils. Furthermore it has been shown that variation in moisture content of the soil across the seasons is minimal from 2.5m downwards. A design criteria for lightly loaded foundations on colla-psible soils is therefore put forward.

A survey of alternative foundation types has been undertaken and the most favoured of these foundations was that using spread footings at depths not. suqceptible to Tuch variation in moisture content. It has also been found that attention need also be focussed on environmental factors such as land-scaping and long term site and plumbing maintenance, if foundation stability is to be guarranteed. A typical design of the most favoured foundation was undertaken and found to impose a bearing pressure of 47KN/m^2 and an angular distortion of 1:556 which are less than the permissible values.

TABLE OF CONTENTS

Chapter		Page
	Title page	i
	Declaration	ii
	Dedication	iii
	Certification	iv
	Acknowledgements	v
	Abstract	vi
	Table of Contents	vii
	List of Tables	x
	List of Figures	xi
	List of Plates	xii
	List of Appendices	xiii
	Notations	xiv
1.	INTRODUCTION	
1.1	Background Information	1
1.2	Statement of the problem	11
1.3	Objective of study	11
1.4	Justification for the study	12
2.	LITERATURE REVIEW	13
2.1	Definition of Terms	13
2.1.1.	Foundations	13
2.1.2.	Lightly Loaded Foundations	13
2.1.3.	Design Criteria	13
2.2	Review of Literature	14

Chapter		Page
2.2.1	Problem soils of Nigeria	14
2.2.2.	Foundation design problems in Sokoto	17
2.2.3.	Collapsible and Expansive soils	22
2.2.4	Foundation on collapsible soils	24
2.2.5.	Foundation on Expansive soils	31
3.	LABORATORY TEST PROGRAMME AND PRESENTATION OF EXPERIMENTAL RESULTS	42
3.1	Test Programme	42
3.2	Classification and Identification tests	43
3.2.1.	Moisture Content determination	43
3.2.2.	Specific gravity determination	47
3.2.3.	Atterberg limits determination	49
3.2.4.	Particle size analysis	52
3.2.5.	Coefficient of concavity and uniformity	53
3.2.6.	Identification and classification of soil	57
3.3.	Strength and deformation tests	59
3.3.1	Standard proctor compaction test	59
3.3.2.	Triaxial tests	62
3.3.3.	Oedometer consolidation tests	67
4.	ANALYSIS AND DISCUSSION OF EXPERIMENTAL RESULTS	72
4.1	Classification Tests results	72
4.2.	Strength and deformation test results	74
5.	SOLUTIONS AND RECOMMENDATIONS	86
5.1	Solutions	86

Chapter		Page
5.1.1.	Improvement of soil properties	86
5.1.2.	Design of effective foundation type	88
5.1.3.	Consideration for environmental and constructional factors	91
5.2.	Recommendations	94
6.	CONCLUSION	100
6.1	Conclusions	100
	References	103
	Appendices	

LIST OF TABLES

Tables		Page
2.1	Triaxial shear test for a collapsing sand profile studied by Jennings and Knight (1957).	27
2.2	Categorization of potential swell by Snethen (1984)	34
2.3.	Criteria for identifying and classifying expansive soils by Snethen (1984)	35
3.1	Variation of Moisture content with depth during Dry season.	45
3.2	Variation of Moisture content with depth during Rainy season	45
3.3.	Specific gravity of soil particles	48
3.4.	Summary of Atterberg limit test results.	50
3.5.	Summary of values of coefficient of uniformity C_u and Coefficient of concavity C_c	56
3.6	Summary of compaction test results	61
3.7.	Water content Vs Zero - Air - Void values	61
3.8.	Summary of Triaxial test results.	64

LIST OF FIGURES

Figure		Page
1.1	Soil profile for test pits 1, 2, 3 and 4	3
1.2	Site plan showing location of test pits	10
2.0	Generalized geological section in a North-Westerly Direction of Sokoto state (After Kogbe 1975)	21
2.1	Behaviour of soil in double Oedometer test	25
2.2	Loaded soil structure before inundating	25
2.3	Loaded soil structure after inundating	25
2.4	Profiles and soil characteristics of selected samples of collapsing soil studied by Jennings and Knight (1957)	26
2.5	Typical compression curve for collapsing soil	28
3.1	Variation of Moisture content with depth	46
3.2	Plasticity chart for the classification of fine soils and the finer part of coarse soils	51
3.3	Particle size distribution curve for pit 2 at 2.5m depth	52
3.3a	Compression Vs Time (log Scale) for moisture content m = 9.67%	68
3.3b.	Compression Vs Time (log scale) for moisture content m = 13.68%	69
3.3c	Compression Vs Time (log scale) for moisture content m = 15.89%	70
3.4	Graph of void ratio versus pressure for various moisture content values	71
4.1	Section through a building on a lightly loaded foundation	78
4.2	Section through a collapsible soil layer on which a structure load of 50KN/m^2 is imposed.	81
4.3	Moisture content versus Settlement Curve.	84
5.1	Ground floor/foundation layout for a typical lightly loaded structure.	96
5.2	Section through a lightly loaded structure.	97

LIST OF PLATES

Plate		Page
I	Severely cracked portion of one of the staff quarters.	4
II	Part of the front elevation to the main administrative building	4
III	Cracks above the front door to the office of the Assistant General Manager (Engineering Services)	5
IV	Severe cracks on one of the staff houses still being habited	5
V	Severe cracks on one of the staff houses already abandoned	6
VI	Cracks on another staff house	6
VII	Severe cracks at the backside of the main administrative building	7
VIII	Severe cracks on one of the staff houses. Despite repair efforts these cracks resurface again	7
IX	Attempts at rectification by patching up cracks	8
X	Excavation of Pit 3 in progress.	8

LIST OF APPENDICES

APPENDICES

APPENDIX 1	Results of moisture content tests for test pits 1,2, 3 and 4.
APPENDIX 2	Results of Specific Gravity test on pits 1, 2 and 3.
APPENDIX 3	Results of Atterberg Limit tests on samples from pit 1, 2, and 3.
APPENDIX 4	Results of Sieve Analysis.
APPENDIX 5	Results of standard proctor compaction tests.
APPENDIX 6	Results of Triaxial tests with stress/strain and Mohr circle diagrams.
APPENDIX 7	Results of consolidation test on samples at various moisture content values.

LIST OF NOTATIONS

B	=	Width of foundation
C	=	Cohesion
C_u	=	Coefficient of Uniformity
C_c	=	Coefficient of Concavity
D	=	Effective diameter of particles
D_{10}	=	Effective size of soil samples
D_{30}	=	Maximum particle size of smallest 30% of soil sample
D_{60}	=	Maximum particle size of smallest 60% of soil sample.
d	=	Length of drainage parth length
e	=	Void ratio
e_i	=	Initial Void ratio
e_f	=	final void ratio
F_s	=	factor of safety
G_s	=	Specific gravity
h or H	=	Height of sample
I_p	=	Plasticity Index
LL	=	Liquid Limit
m	=	moisture content or water content
mv	=	Coefficient of compressibility
N_c	=	Bearing capacity factor
N_q	=	" " "
N_γ	=	" " "
PL	=	Plastic limit

P = Pressure on soil

q_u = Ultimate Bearing Capacity

γ = Bulk Density of soil sample

γ_d = Dry Density of soil sample

S = Settlement

S_r = Degree of Saturation

S.B.C. = Safe Bearing Capacity

SL = Shrinkage Limit

t = time of consolidation

V = Total volume of soil sample

W = Total weight of soil sample

Z = Depth of foundation

ϕ = Angle of internal friction.

CHAPTER ONE

1. INTRODUCTION

1.1 BACKGROUND INFORMATION

Sokoto is the capital of Sokoto State in Northwestern Nigeria. It experiences numerous foundation problems, which results in the cracking and failure of several buildings within Sokoto metropolis. Sokoto is in the semi arid region and is succesptible to long dry seasons. The topsoil within this area is largely wind blown sands which lack good bonds, and therefore create enormous foundation problems.

This project was neccessitated by the severe damages observed on buildings within Sokoto metropolis in general and the Sokoto Rima River Basin Development Authority Headquarters in particular. It is easy to observe that most lightly loaded buildings within Sokoto metropolis are severely cracked. Many of these buildings have been abandoned and many more are very unsafe for human habitation. It was therefore thought that an investigation into the causes of failure of these buildings will be a worthwhile contribution towards solving one of society's numerous engineering problems

Although damage to buildings is quite prevalent in Sokoto, it was considered appropriate to choose a particular location as a test case for the study, so as to undertake an indepth study. In the course of the reconnaissance study, it was decided that the Sokoto Rima River Basin Development Authority Headquarters, which has many severely damaged buildings be used as a case study.

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At the beginning of the study, it was thought that the observed cracks were due to the existence of an expansive layer of soil immediately below the foundations, but preliminary investigation showed the presence of lateritic soils with very minimal clay content. Thus, expansion of the immediate soil strata was not likely.

Reconissance survey showed that virtually all the buildings within the Sokoto Rima River Basin Development Authority Headquarters had severe cracks. Also, most of the staff quarters within the compound had been badly damaged and infact many houses have been condemned with the resident staff moved out of them. The extent of damage within this compound can be best assessed from plates I to X. These plates are photographs taken to further highlight the problems on site.

In carrying out the preliminary investigation, four test pits were excavated. Each of these excavations was carried to a depth of 3.0m from the natural gound level. For the four tests pits, the soils were found to be basically the same. The topmost layers were observed to be reddish brown topsoil of about 0.7m depth. The top soil is underlain by a layer of reddish brown loosely packed gravel with sand up to a depth of about 2.0m. While further down the excavation, a layer of lateritic soil in fairly dense state was observed. This layer comprises gravels, cobbles, boulders and slabs of cellular lateritic and iron cemented sandstones in a matrix of sand.

This observation further strengthened the view that observed failures were due to collapse of loosely packed coarse sand on application of load and or on inundating with water, rather than expansion of a clay layer. Fig. 1.1 shows a profile of the soil in each of the four test pits.

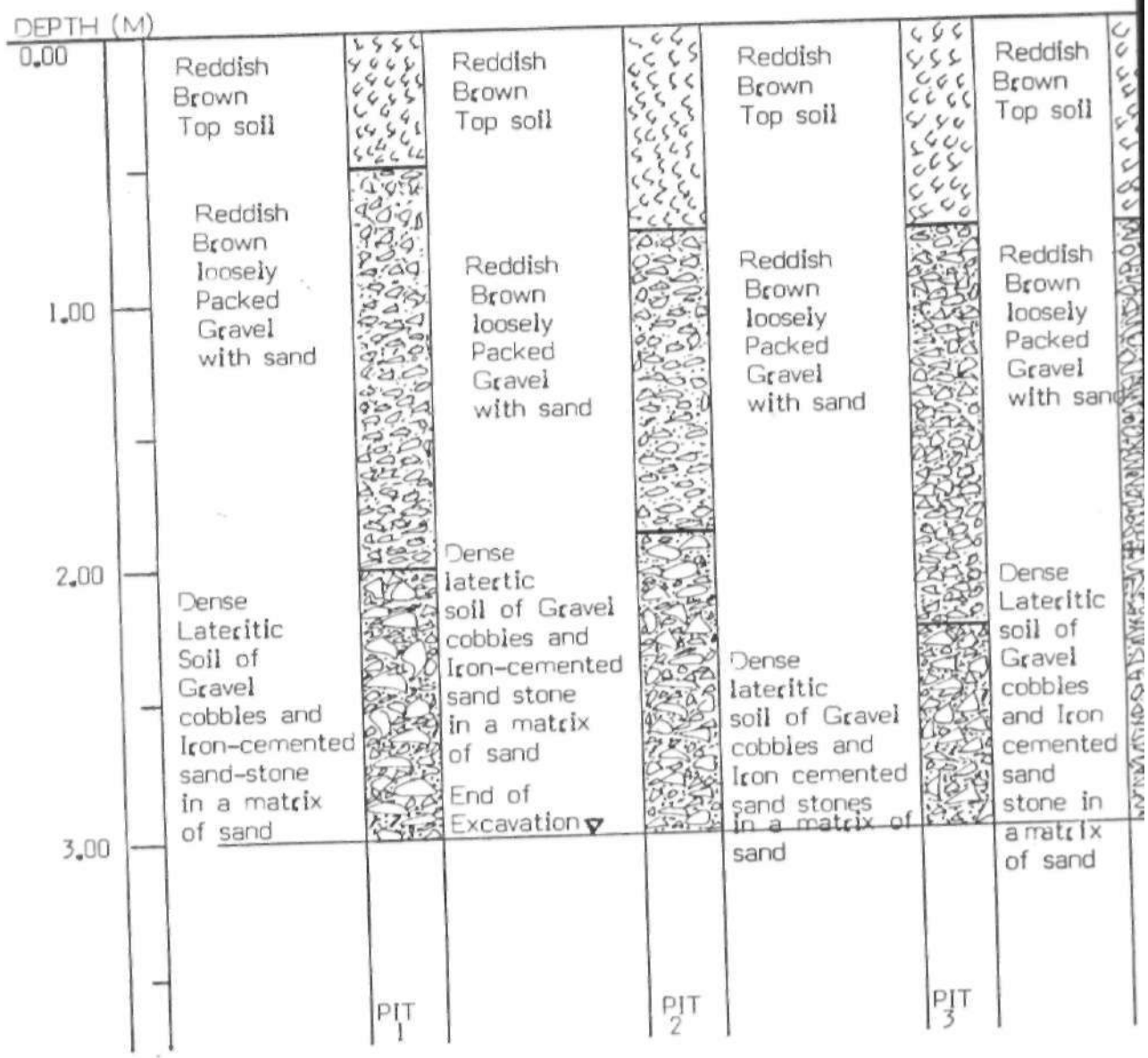


FIG: 1.1

Subsurface Soil Profile for Test pits 1,2,3, and 4.



PLATE I

Shows a severely cracked portion one of the staff quarters. Alread the occupants of this house have been moved out.



PLATE II

Shows part of the front elevation to the main administrative buildings. This kind of cracks are very common on the administrative building.



PLATE III

Shows the extent of cracks above the front door to the office of the Assistant General Manager (Engineering Services).



PLATE IV

Shows severe cracks on one of the Staff houses still being habited.



PLATE V

Shows severe cracks on one of the staff houses already abandoned.



PLATE VI

Shows cracks on one of the staff houses still being habited.



PLATE VII

Shows severe cracks at the backside of the main administrative building.



PLATE VIII

Shows severe cracks on of the staff houses. Despite repairs efforts, cracks resurface again.



PLATE IX

Shows attempt at rectification by patching up cracks did not yield any positive results as cracks re-appear after a while.



PLATE X

Excavation of pit 3 in progress.

In carrying out the investigation, three of the four test pits were excavated during the dry season while the fourth test pit was excavated during the rainy season, this was with a view to comparing the variation of moisture content with depth across the two seasons. The test pits used were chosen such that a fair coverage of the Sokoto Rima River Basin Development Authority Headquarters was obtained. Each of these test pits were excavated to a maximum depth of 3.0m. Since most of the buildings within this area are lightly loaded (predominantly Bungalows), it was suspected that their foundations were at depths of about 1.0m below natural gound level and the foundations are predominantly strip footings. Thus the effect of stresses on soils below these foundations are not expected to be felt at depths below 3.0m from the natural gound level.

Finally, since excavations were by manual method, it was impossible to go beyond 3.0m, because the excavations were done in the dry season when the soils are in their hardest form. It is important to state here that more samples would have been obtained if drilling rigs were used in sampling. But because of the prohibitive cost of hiring and getting a drilling rig down to Sokoto, this alternative could not be contemplated. Fig. 1.2 shows a site plan of the Sokoto Rima River Basin Development Authority Headquarters, with location of the various test pits used for samplying.

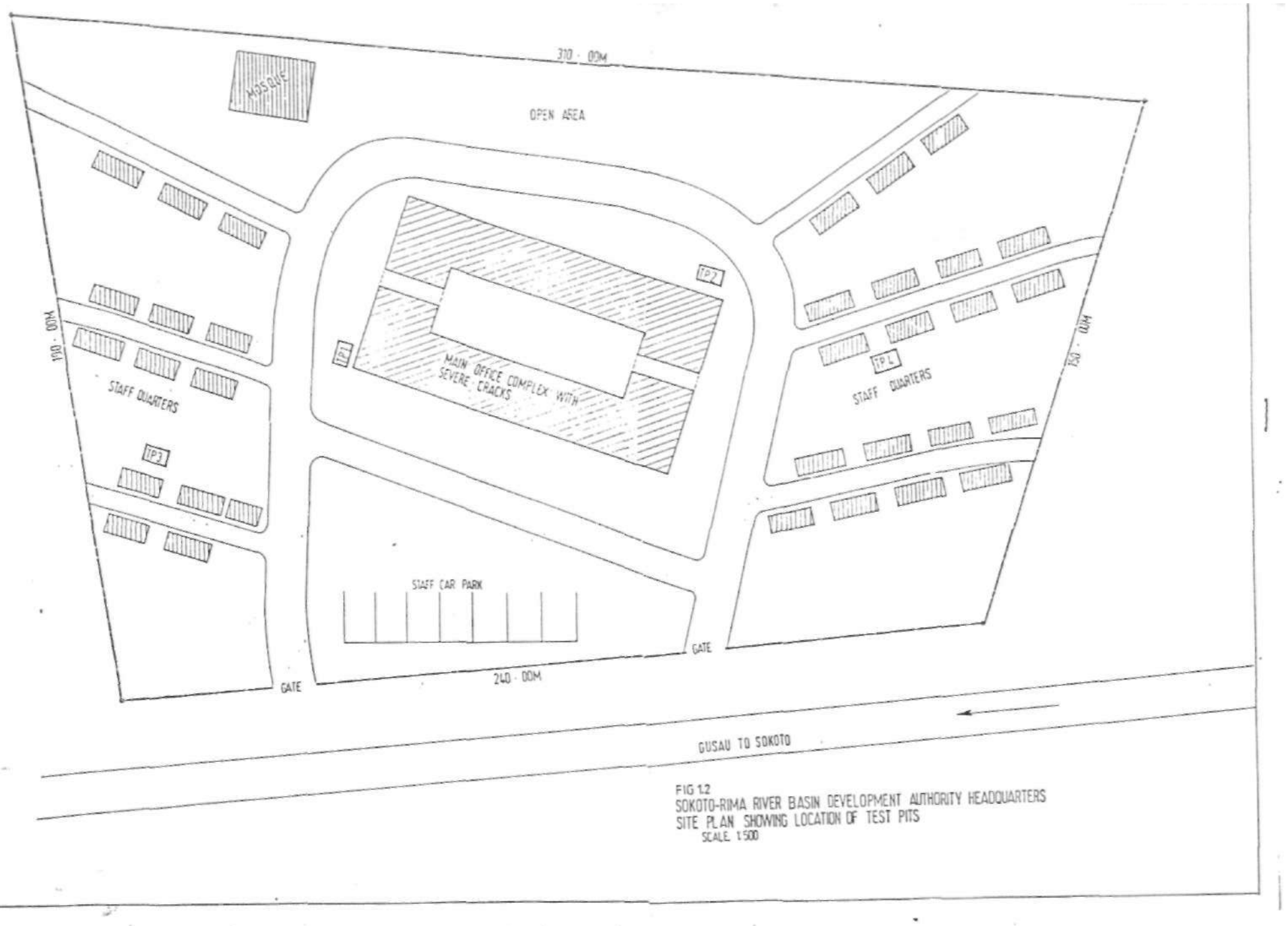


FIG 1.2
SOKOTO-RIMA RIVER BASIN DEVELOPMENT AUTHORITY HEADQUARTERS
SITE PLAN SHOWING LOCATION OF TEST PITS
SCALE 1:500

1.2 STATEMENT OF THE PROBLEM

The cost of damages to the buildings within the Sokoto Rima River Basin Development Authority Headquarters is enormous. So far, eight bungalows, each estimated at not less than one hundred and fifty thousand naira (₦150,000) have been abandoned. From site reconisance, more houses will be abandoned in future, and if remidial measures are not taken immediately, it is envisaged that in no distant time, the Authority might be compelled to completely abandon the entire complex. There is no doubt that over five million nair (₦5,000,000.00) have been invested in the physical development of this complex and if the site was abandoned, this would constitute a gross waste of scarce resources. Furthermore, since more development is expected to be carried out in future, it becomes very imperative that a well researched design and construction criteria be developed if the errors of the past years are to be avoided. The present study has been designed to initiate meaningful research into this problem. Specifically the problem tackled in this study is double faceted. One the underlying causes of the foundation problems at the studied site are studied. Secondly, the possible solutions are searched for.

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1.3 OBJECTIVES OF STUDY

This study is aimed at ascertaining the causes of severe cracks on buildings within Sokoto in general and the Sokoto Rima River Basin Development Authority Headquarters in particular, and recommend design criteria that could arrest the pattern. To achieve this goal, the study has been designed to achieve three main objectives, which are:

(1). To undertake a detailed geotechnical investigation of the subsoil at the Sokoto Rima River Basin Development Authority Headquarters, with a view to better understanding the mechanisms involved in the failure characteristics of the superstructures.

(II) To examine the effects of environmental factors, such as moisture content, on the failure pattern of these soils.

(III) To recommend foundation design methods that adequately meets the peculiar situation at the complex on the basis of the results of the above investigation.

It is realised that an important source of problem on this site could very well be the quality as well as type of construction. Nevertheless, investigation within Sokoto and discussions with engineers within the Authority and other arms of government confirm that the problem being experienced is largely geotechnical.

1.4 JUSTIFICATION FOR THE STUDY

Investigations within the Sokoto Rima River Basin Development Authority confirms that while the Authority is disturbed about the extensive damage to its buildings, no investigations have so far been comissioned on the causes of these damages. It is therefore hoped that this study will serve as a source of geotechnical information for future development on the site as well as giving enough geotechnical information to enable the Authority carry out effective repair works on the existing buildings if and when it is so desired. Also, it is hoped that the results obtained would generate some academic discussion that might encourage more research work on other problem soils in Nigeria, and thus lead to a standard method of geotechnical characterisation of these soils.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 DEFINITION OF TERMS

Below are the definitions of the significant terms that are specific to the understanding of this work.

2.1.1. Foundations

These are the structural elements that serve the purpose of transfering the load from the super structure to the underlying soil.

2.1.2. Lightly Loaded Foundations

These are foundations to low rise buildings (i.e. one or two storey buildings). Usually foundation pressures of such buildings in Nigeria do not exceed $150 kN/m^2$.

2.1.3. Design Criteria

Usually in the design of foundations, there are two foundamental requirements which any foundation must satisfy if it is to perform satisfactorily. These are:

(a) That the load transfered by the foundation from the superstructure should not result in the excessive stressing of the soil to cause shear failure of the soil below the foundation. This criteria implies that there should be no shear failure of the soil and is known as the Bearing Capacity Criteria.

(b) That compressibility of the soil should not result in excessive differential settlement between different segments of the structure. This is known as the Settlement Criteria

Thus in designing a suitable foundation, the behaviour of the foundation with regards to these two criteria and the effect of environmental factors such as variation in moisture content on these two criteria are considered.

2.2 Review of Literature

2.2.1 Problem Soils of Nigeria

Published work related to foundations on problem soils in Nigeria is very scanty. Ajayi, (1987) showed that the main troublesome soil deposits in Nigeria with regards to Building foundation could be classified into five groups namely:

(1) *Recently deposited Organic soils*
(2) Residual "Lateritic" Soils
(3) Sedimentary Lateritic soils
(4) Residual expansive soils
(5) Collapsible loess sands.

He further gave an indepth analysis of each of these problem soils as follows:

1. Recently Deposited Organic Soils

These are recently deposited very soft to soft organic clays and peats of the Dahomey and Niger Delta basins in the extreme south of the country. This soil poses serious settlement and stability problems for shallow foundations, and examples abound of distorted and severely cracked buildings in various parts of this region, notably Lagos metropolis. These deposits are usually encountered in fully saturated conditions and are also normally consolidated. The organic deposits are highly compressible and of high plasticity.

2. Residual Lateritic soils

Residual soils are those resulting from weathering and decomposition of the crystalline rocks - granites, gneisses, etc., which underlie major parts of the country. These soils have remained in situ after formation and are generally classified into the upper, most laterised soils horizon and the lower, less laterised highly weathered rock zone.

The soils of the upper horizon are generally sandy clays with varying gravel content, variably dessicated and with farruginous cementation. This zone is usually well above groundwater level, hence the soils are of stiff to hard consistency, over consolidated and of medium to low plasticity and compressibility. They are also generally of low to very low permeability. However, when exposed to water ingress or flow, there would be loss of chemical bond and leaching of fines, resulting in significant loss of bearing capacity, increased settlement and eventual failure of support structure.

The lower horizon soils, generally known as saprolites are normally dense to very dense, more granular than the overlying deposits and very silty, often containing weathered rock fragments. They constitute very little problem to foundation performance except when the overlying more laterised clayey deposits have been removed and they become exposed to rapid moisture fluctuations. They are mostly found in the South Western part of the country.

3. Sedimentary Lateritic Soils

Sedimentary lateritic clays and clayey sands are essentially the redeposited hill wash materials or weathering products transported to the low lying areas of the country during the Oligocene to Plio-Pleistocene era and have become dessicated.

The most prominent of these deposits is the "Coastal Plains Sands" of the South which has been widely discussed by Jones and Hockey (1964) Jackson (1980) and Ajayi (1985). Groundwater is usually very low in these areas. The soil profile is unstratified and have inherited an unstable structure which collapses when the soil becomes wetted or resaturated. In their natural state, they are generally firm to stiff becoming very stiff, over consolidated and of medium to low plasticity, compressibility and permeability.

4. Redidual Expansive Soils

Residual expansive clays develop from weathering of shales, mudstones and basalts found in some parts of the country, notably the north-east and northwest sectors. They generally underlie superficial sandy clays or very silty sands of few to several meters thickness. These clays are generally stiff to very stiff, over consolidated, of medium to high plasticity, medium to low compressibility and generally have very low permeability.

However, when the normal cycle of moisture migration is altered by the imposition of an artificial cover through shallow foundation construction or road pavement, the clay tends to absorb more water than necessary; then swelling process generates high expansive forces which leads to structural damage, severe cracking of walls, slabs and pavements.

Of particular interest in the study of expansive soils is the well known "Black Cotton Soils" of the northeastern sector, discussed by Ibrahim (1979), Ola (1983) and Madedor et al (1985). These soils originate partly from weathering of basalts and partly from the quartenary lacustine sediments of Chad basin. The predominant clay mineral being montmorilonite, accounts for their characteristic swelling on water absorption and shrinkage on drying.

Severe deformations of road pavements and cracking of walls of buildings constructed without proper attention on expansive soils have been widely reported.

5. Collapsible Loess Sands

In the semi arid areas of extreme northern part of the country are wind blown sands described by Ola (1980) as possessing collapsing sand features. The sands are naturally loose to medium dense at the top rapidly becoming dense or generally dense in places, unsaturated and silty. These features account for their ability to drain off surface water very rapidly. However, when due to shallow foundation construction, the natural process of moisture transfer is interrupted, saturation occurs haphazardly under the structure, the soil slumps causing differential settlement, severe cracking and structural damage.

From the findings of Ajayi (1978) as summerised above, it is evident that of the five problem soils listed, the first three namely, Recently Deposited Organic soils, Residual "Laterite" Soils, Sedimentary Lateritic soils are found predominantly in the southern part of the country while the last two namely Residual Expansive Soils and Collapsible Loess Sands are most likely to be found in the north. Also from his description of the geology of the various problem soils, it is evident that the soil most likely to be causing the foundation problem in the semi arid area of Sokoto is the Collapsing soil. The findings of Ajayi is corroborated by Ola (1979).

2.2.2. Foundation Design Problem in Sokoto Area

Ola, (1979) showed that foundation to buildings in Sokoto suffer distresses which produces severe crackings on the superstructure.

These distresses are due to differential movement of the subsoil and and his investigation revealed that there are three major problem soils in the Sokoto area. These are:

(1) Presence of Sinkholes and caverns in limestone deposits.

(2) Expansive soils

(3) The collapsing sands.

Thus, by Ola's findings, it could be concluded that the problems of (1) Recently Deposited Organic Soils, (2) Residual Laterite Soils, and (3) Sedimentary Lateritic soils are not of any significance in Sokoto area as they do not exist. In analysing the problem soils stated above, Ola expressed the following views:

(1) Caverns and Sinkholes from Limestone formation

This is the most potentially dangerous problem of the Sokoto area which luckily has not been encountered on any large scale. In Ola's study, eleven boreholes were drilled and three 1.2m diameters shafts were sunk. Recovery was often as low as 15 percent. Two of the shafts were abandoned due to unmanageable volumes of water issuing from the limestone at a rate exceeding $31.8m^3/hr$.

The generalized profile consist of variable thickness usually 0-3m of wind blown sands. This sand presents its own special foundation problem which is discussed later. The wind blown sands are underlain by a layer of cellular laterite and colitic ironstone. The depth to the troublesome limestone layers varies from location to location in Sokoto, but this depth may only be 4.6m in some places with attendant foundation problem.

(2) Swelling Clays

Generalized geological section across North-Western Sokoto (Fig. 2.0) shows why the Sokoto area in particular is complex from foundation point of view. In North-Western Nigeria, folding movements resulted in the gentle tilting of all the Pre-Maastrichtian, Maastrichtian and Palaeocene sediments in a North-Westerly direction as shown in Fig. 2.0, Kogbe (1975). From this figure, as a result of the tilting of the various formations, coupled with the gulley erosion of the Sokoto river it is clear that depending on the location, building foundations can be placed on the clay shale.

Analysis of this clay shows some very unusual properties. The clay was found to be highly compressible with coefficient of compressibility C_c varying from 0.2 to 0.25 for a five storey building. Also result of test shows that when compared with standard classification method, the clay shale has very critical potential volume change.

The results obtained showed that the clay is of critical potential expansiveness. Unfortunately, as a result of the tilting of the various formations (Fig.2.0). This expansive clay is within foundation level in certain areas of Sokoto and where this happens and ingress of water is unavoidable, swelling of the foundation results. Also if this clay is within the zone of seasonal volume change, swelling of the foundation will occur during the rainy season and this will be followed by shrinkage during the dry season.

(3) Collapsing Sand

The third problem which all geotechnical engineers working in Sokoto area should be quite familiar with is the problem of "collapsing sand".

As shown in the generalized geological section in Fig. 2.0, the lower sand outcrops in Sokoto area is basically a foundation problem. The depth of this deposit varies from 0 to 3m from location to location. This layer consist of reddish brown wind blown silty sand. The depths of collapsing sands may vary from a few centimeters to more than 15m and settlements are large only where the depth below the foundation is significant.

From the analysis of some foundation Design Problems in Sokoto Area presented above, it is clear that the findings are very relevant to the topic under consideration. Thus the following deductions could be made:

(a) Crackings observed on buildings within the Sokoto Rima River Basin Development Authority Headquarters could not be as a result of presence of sinkholes and caverns in limestone formations. This is because at the depth at which the limestone deposits are generally encountered (i.e below 4.6m), the effect of loads from lightly loaded foundations are negligible. Secondly, if collapse of the limestone formation is to take place, it will result not only in cracks on the building, but also on subsidence of the soil arround the region of collapse of limestone formation. This subsidence have not been noticed.

(b) The cracks observed are therefore most likely to be as a result of the existence of either the collapsing sand or an expansive soil layer beneath the foundations.

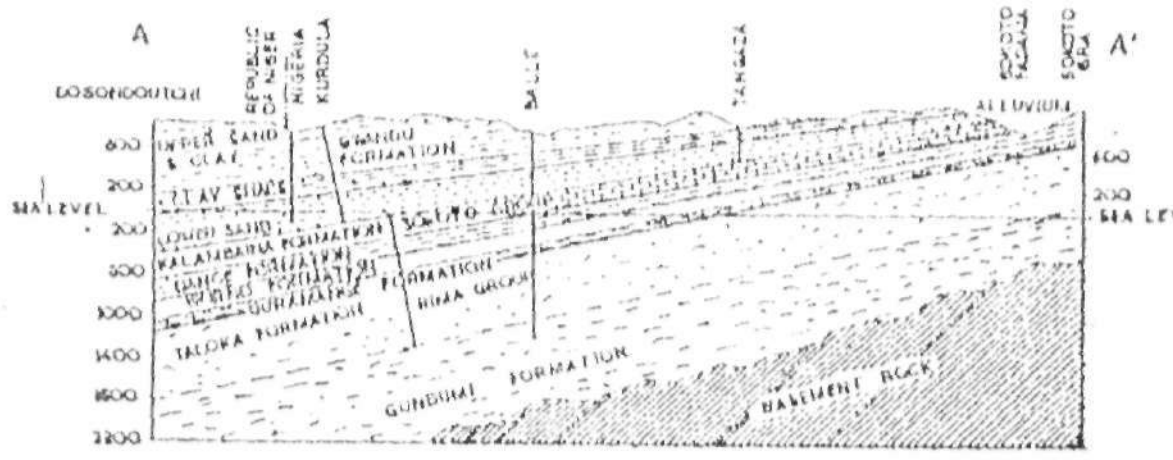


FIG. 20 GENERALIZED GEOLOGIC SECTION IN A NORTH-WESTERLY DIRECTION OF SOKOTO STATE (AFTER KOGBE 1975).

2.2.3 Collapsible and Expansive Soils

From the literature review so far carried out on the nature of soils within Sokoto, it is evident that the problem soil within the Sokoto Rima River Basin Development Authority Headquarters could either be a collapsible soil or an expansive soil.

Collapsible soils are widespread in areas where there are long dry seasons and in which the ground water table are at great depths. Such soils are usually wind blown sands, loosely deposited and binded together with bonds consisting of soluble compounds, such as the chemical precipitates that accumulate under semi arid conditions. Under partially saturated conditions, they may posses bonds that endow them with cohesion and considerable rigidity. These bonds are easily destroyed upon prolonged submergence. If the bonds consist of clay minerals or various amorphons clay like substances, they may absorb water and become weak when submerged.

The e - logP curve of such material, tested without permitting water to gain access to the samples through the porous stones display no unusual features, but if water is admitted at any stage of a test, a spectacular decrease in void ratio at constant external vertical pressure may occur. The magnitude of the settlement of a deposit of collapsible soil may be estimated by subjecting samples at their field moisture content to the vertical stresses anticipated in the ground and by observing the change in void ratio due to submergence while the vertical stress are maintained. Thus collapsible soils could be classified as soils which exhibit large reduction in volume upon changes in moisture content.

On the other hand, expansive soils develop from weathering of shales, mudstones and basalts.

They generally underlie superficial sandy clays or very silty sands of few to several meters thickness as shown by Ajayi (1987). These clays are generally stiff to very stiff, over consolidated, of medium to high plasticity and generally very low permeability. Expansive soils in general are most critical for foundations when encountered above the lowest groundwater level, since below the ground water table, the clay is saturated and the moisture content is relatively constant. However, most clay become desiccated and shrink due to seasonal lowering of the groundwater table or changes in other physical or environmental conditions. On subsequent wetting, the clay will exhibit swelling. Thus expasnive soils could be classed as soils which exhibit large increase in volume upon increase in moisture content.

Peck et al (1973) showed that certain soils, even while under constant external load, exhibit large volume change upon changes in moisture content. They were of the view that soils which experience a reduction in void ratio on inundating with water under constant external load are collapsible soils while those which exhibit an increase in void ratio on inundating with water under constant external load are expansive soils. Thus in both cases, moisture content increament is the one most critical factor that affects the behaviour of these soils.

They further showed that these soils are confirmed by carrying out a pair of test in a consolidation apparatus on identical undisturbed samples. Fig. 2.1 shows the expected results of such tests. Curve "a" represents the e - log P curve for a test started at the natural water content and to which no water is permitted access, curve "b" and "c" on the other hand corresponds to tests on samples to which water is allowed access under all loads until equilibrium is reached.

If the resulting e – log P curve, such as curve b, lies entirely below curve a, the soil is said to have collapsed. Under field conditions, at pressures P_1 and void ratio e_o, the addition of water would, accordingly cause the void ratio to decrease to e_1 with a corresponding settlement of

$$H \times \frac{\Delta e_1}{1 + e_o}$$

Where H = Height of sample

e_o = Void ratio at pressure P_1 without addition of water.

e_1 = Void ratio at P_1 after addition of water.

$\Delta e_1 = e_o - e_1$

Soils exhibiting this behaviour include true loess, clayey loose sands in which the clay serves merely as a binder, loose sands cemented by soluble salts and certain residual soils such as those derived from granites under conditions of tropical weathering.

Conversely, if the addition of water to the second sample leads to curve "c" located entirely above 'a', the soil is said to have swelled. At a given applied pressure P_1, the void ratio increases to e'_1, and the corresponding rise of the ground is measured by $H. \frac{\Delta e'_1}{1 + e_o}$. Soils exhibiting this behaviour to a marked degree are usually montmorillonitic clays with high plasticity indices.

2.2.4 Foundation on Collapsible Soils

For a good understanding of the behaviour of foundations on collapsible soils, a knwoledge of the collapse mechanisms of these soils is essential. Consideration of the structural arrangement of the sand grains in a loose silty sand furnishes a hypothesis for the collapse phenomenon.

Casagrande (1932) has demonstrated that a portion of the fine fractions of the soil exist as bonds for the larger grains and that these bonds undergo local compression in the small gaps between adjacent grains resulting in the development of strength. Fig. 2.2 represents an idealization of this concept.

In nature the structure of the soil is in equilibrium under the action of overburden pressures. When loaded at its natural water content the structure remains sensibly unchanged and the bonding material compresses slightly without any large relative movement of the soil grains as shown in fig. 2.2. At this stage, consolidation takes place by compression of the fine particles between the larger sand particles. As long as the moisture content remains low, the local micro-shear forces at the sand particle interfaces, resulting from loading, are resisted without appreciable movement of the grains. However, when the loaded soil, gains moisture and a certain critical moisture content exceeded, the fine silt bridges reach a stage where they can no longer resist the deformation forces and the structure collapses as shown in fig. 2.3, resulting in considerably smaller volume than that shown in fig. 2.2. The collapse phenomenon appears to be one of micro-shear effects at the particle interfaces, following the character of secondary consolidation and therefore should be capable of demonstration by normal consolidation test procedure.

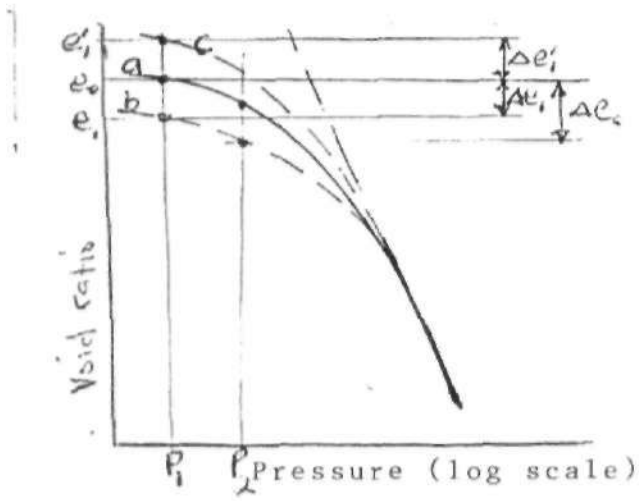


Fig 2.1
Behaviour of soil in double Oedometer test.

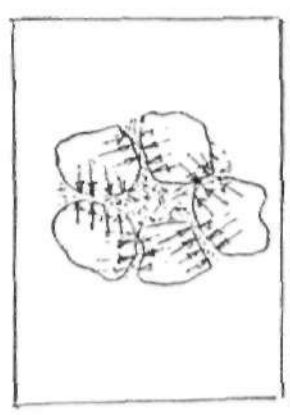


Fig.2.2
Loaded soil structure before inundating

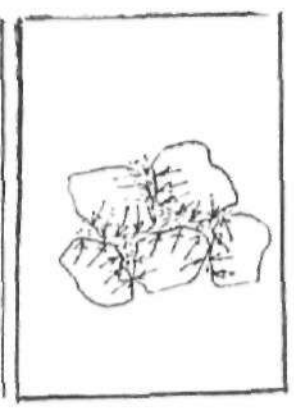
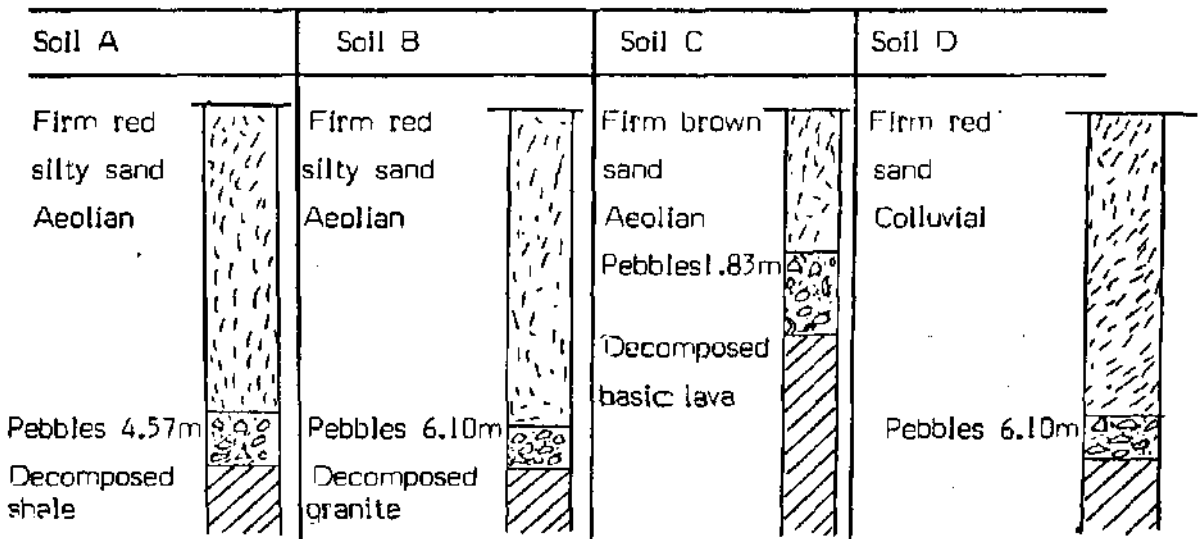


Fig.2.3.
Loaded soil structure after inundating.

Research works related to foundations on collapsible soils are very limited. This might be due to the fact that collapsible soils are more prevailent in Arid and semi - Arid regions of the tropics, and the level of research within this region is relatively low. As earlier stated, most wind blown sands and silts, especially those whose cohesion are imparted by precipitation of soluble compounds such as calcium carbonate, gypsium or ferrous iron and which collapse upon inundation are classed as collapsible soils.

Jennings and Knight.(1957) in their work have details of some unusually large and sudden settlement which occured in South Africa. In all the cases considered, the movements were associated with the presence of water, frequently due to broken drains or to a concentration of rainwater at the discharge points of rainwater downpipes. Figure 2.4. gives the profiles and soil characteristics for four sites investigated by the authors.



	Soil A	Soil B	Soil C	Soil D
		SOIL CHARACTERISTICS		
LL	38.5	26.4	25.5	NP
PL	22.9	12.0	11,0	NP
W	10.5	12.9	8.0	8.0
Sr	0.29	0.41	0.31	0.30
G_s	2.68	2.70	2.68	2.65
e	0.96	0.85	0.70	0.75

Fig. 2.4. Profile and Soil Characteristics for selected samples of collapsing soil studied by Jennings and Knight (1957).

The silty sands were observed to have very high void ratios and a fair proportion of the minus 2 micron fraction. The predominant particle sizes obtained from sieve analysis were shown to lie between 0.1 and 0.4mm. The sands were believed to be either colluvial or wind blown deposits. The minus 2 micron fractions were found to be composed almost entirely of red brown iron oxide.

Triaxial shear tests of the silty sand showed that there is a reduction of about 30% in shear strength of the soil upon inundation. A typical shear test data for a collapsing sand profile as obtained is shown by Jenning and Knight in Table 2.1 below:

Table 2.1: Triaxial Shear test for a collapsing sand profile. studied by Jennings and Knight (1957).

Depth	Undrained test S_{uu}		Consolidated Soaked Undrained test S_{csu}	
(m)	C_u KPa	ϕ_u degrees	C_{csu} KPa	ϕ_{csu} degrees
1.5	12.4	21	15.17	14
3.0	31.7	11	24.8	7
4.5	28.3	10	20.7	9
6.0	20.68	23	4.8	19

It is important to note that it was observed that despite the 30 percent reduction in shear strength on inundation, all cases investigated showed that the bearing capacities of the soil at the time of failure had not been exceeded. In many cases, the calculated capacities were considerably greater than the applied loadings.

Jennings and Knight (1956) showed that it was possible to reproduce the settlement behaviour of the foundation completely, by applying the field pressures to a consolidometer specimen at its natural water content, allowing the consolidation to reach equilibrium and then inundating the sample by filling the confining dam with water. The inundation resulted in an additional settlement or collapse. These collapse were greater when higher consolidation pressures were used. Typical result of oedometer test are shown in fig. 2.5. The marked divergence of the curves indicates the magnitude of the movements due entirely to inundation.

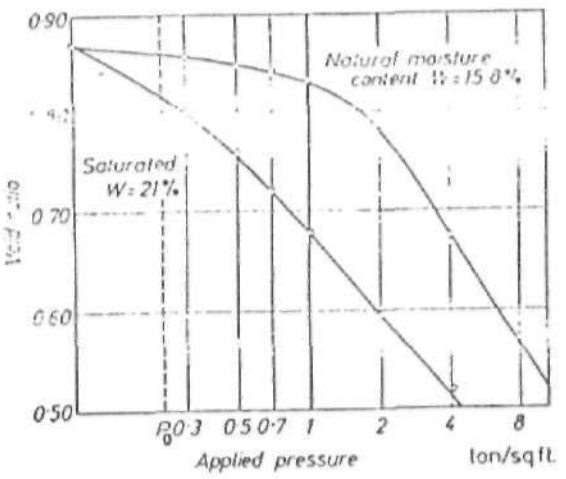


Fig 2.5 Typical compression curves for a collapsing soil

Finally it was shown that the use of the e : log P curve as in Fig. 2.5 is not in strict agreement with conventional soil mechanics theory which requires that the pressure used be truely intergranular pressures. In the case discussed, capillary pressures of fairly high magnitudes exist and these are certainly of the same order as the applied pressures. The pressures used in the plot represent applied pressures and therefore the implications of using normal interpretations should be carefully considered. Full scale varification tests and analyses for correlation with existing field observations are still being considered.

Brink and Kantey (1962) investigated the causes of sudden tilting of a water tower founded in decomposed granite in South Africa. Their work led to the discovery that the soil exhibited a grain structure which collapsed on becoming inundated under load.. They also found out that the same phenomenon occured in many other places as well, but appears to be confined to slopes in areas of annual water surplus. It was evident from their work that the phenomenon of collapsible grain structure develops as a result of leaching out of soluble and colloidal matter from the thoroughly decomposed rock. This resulted in a high void ratio and an unstable arrangement of quartz and felspar grains in the leached residuum.

The authors also showed that there were strong indications that the potential collapse decresed with depth in the soil profile which led to the conclusion that the foundation design could be based on permissible tolerable settlement rather than on permissible bearing capacity. Finally three methods of solving the problems posed by this collapse phenomenon were suggested, namely:

(1) The foundation should be carried down to depths at which the collapse phenomenon is absent or of negligible proportions. This may be achieved by normal piling or by the under-reamed type of piling, depending on the strength of the soil at the founding depths, and the load to be carried.

(2) To carry out more detailed testing than is normally carried out for this type of problem and design the foundation on the basis of permissible settlement in addition to permissible bearing capacity.

(3) To carry out a compaction of the grain structure by the use of driven cast-in-situ piles.

Abelev, and Askalonov (1955) described the principles of mechanical stabilization of collapsible soils using heavy tampers and stabilized soil piles. Also the theoretical principles of physical and chemical stabilization of collapsible soils using sodium silicate were discussed. Their research was prompted by their earlier findings that experience during construction showed that measures directed towards prevention of wetting from the soil under foundations do not always attain their objective since various causes lead to accidental soil moistering. The authors showed that the simplest way of eliminating settlement in collapsible soils, is to compact the soil mechanically, and the efficiency of this method has been corroborated by experiences during constructions on collapsible soils in Russia. . In this mechancial compaction method, only mechanical destruction of the natural soil structures took place, the granular and chemical composition of the soil being preserved. The method involes the compaction of a thin layer of collapsible soil under the foundation with heavy tampers up to the point of refusal or until settlement virtually ceases. The compaction of a soil layer of greater depth is usually achieved by driving friction piles. Another method recommended for compation uses the energy of explosion method. This method involves the boring of a 60 to 70mm diameter hole by means of a vibrator or pile driver to the desired compaction depth. The blast charges are then lowered into the hole, the charges consists of standard ammonite cartridges weighing 50g each and 42mm in diameter.

The quantity of cartridges per linear meter of bore hole varies from 5 to 10, depending on the Atterberg plastic index of the soil. It is necessary that the cartridges in the charge are arranged at equal distances apart, and that their arrangement remains fixed until the explosion. The charge is then lowered into the bore hole and exploded.

Under the action of the explosion the bore hole expands, by compaction of the soil laterally up to 40 to 50mm, but remains unchanged vertically.

Here in Nigeria, research works related to foundations on collapsible soils are virtually non-existent.

. Ola (1979) showed that collapsing sands consititutes one of the major foundation problem in Sokoto area. He showed that the depth of this deposit may vary from a few centimeters to more than 15m and settlement are large only where the depth below the foundation is significant. Comments on collapsing sand in this paper were limited to a survey of collapsing soils in Sokoto, a good literature review on collapsing soils and a physical characterization based on particle size distribution tests.

Ajayi L. A. (1987) in his work on Foundation in Problem soils of Nigeria did mention the existence of collapsible loess sands in the semi arid areas of extreme northern part of the country. He gave a general description of the soil as being naturally loose to medium dense at top and rapidly becoming dense lower. They are usually unsturated and silty. He further points out that, when due to shallow foundation construction, the natural process of moisture transfer is interrupted, saturation occurs hapharzardly under the structure, the soil slumps causing differential settlement, severe cracking and structural damage.

2.2.5. Foundation on Expansive Soils

Ajayi (1985) and Ola (1979) have shown that expansive soils are widespread in north-east and north-west sector of Nigeria.

Peck, et al (1973) indicates that a distinction must be made between soils that have the capacity to swell and those that actually exhibit the swelling characteristics in the field. Soils having little or no capacity to swell will not do so under any circumstance. On the other hand, soils with high swelling capacity may or may not swell, their behaviour depends on the physical conditions of the material at the beginning of construction and the changes of stress and moisture content to which they are subjected.

Whether a soil with high swelling potential will actually exhibit swelling characteristics depends on several factors. That of greatest importance is the difference between the field moisture content at the time the construction is undertaken and the equilibrium moisture content that will finally be achieved under conditions associated with the complete structure. If the equilibrium moisture content is considerably higher than the field moisture content, and if the soil is of high swelling capacity, vigorous swelling may occur as evidenced by upward heaving of the soil or structure or by the development of large swelling pressure. If the equilibrium moisture content is lower than the field moisture, the soil will not swell but, on the contary will shrink. The swelling potential of a soil is related in a general way to the plasticity index. With the swelling potential increasing with increase in plasticity Index.

Snethen (1984) evaluated expedient methods for identification and classification of potentially expasnive soils. He is of the view that the purpose of an identification and or classification technique for expansive soils is to qualitatively characterise the potential volume change behaviour of suspected problem soils.

His assessment of the then available literature indicated that methodology for identification of expansive soils may be categorized in the following manner:

(1) Indirect techniques in which one or more of the related intrinsic properties of the expansive soils are measured and complemented with experience to provide an indication of potential volume change.

(2) Direct techniques which involves actual measurement of volume change in oedometer type testing apparatus. Generally, this group is used for quantitative prediction rather than qualitative indication.

(3) Combination techniques in which data from the indirect and direct techniques are correlated either directly or by statistical comparison to develop general classification with regard to porbable severity.

The expedient methods commonly used by engineers are derived from the indirect and combination techniques. The exnedient methods are essentially correlations between experience and or direct measured volume change behaviour with Atterberg Limits. The popularity of these expedient methods is the result of two factors

(i) The practicality of the methods since most of the soil properties involved are routinely determined in geotechnical laboratories and easily recognised by geotechnical engineers.

(ii) Experience has shown that potential swell correlates reasonably well with Atterberg Limit data.

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He further observed that evaluation of the various published expedient methods was very difficult because none of the techniques provided a universally applicable criteria for assessing or categorizing volume change behaviour. One of the most obvious inconsistencies in the various published methods reviewed by Snethen involved the definition and classification of potential swell. He thus put forward a definition of potential swell which satisfies the largest portion of the field simulation requirement as follows:

Potential swell is the equilibrium vertical volume change or deformation from an oedometer - type test (i.e. total lateral confinement), expressed as a percent of original height, of an undisturbed specimen from its natural water content and density to a state of saturation under an applied load equivalent to the in situ overbruden pressure.

Based on the above definition of potential swell, he put forward a categorization of potential swell which is a better classification with regards to insitu conditions. This categorization is given in Table 2.2.

Table: 2.2: Categorization of potential swell, by Snethen (1984).

Potential Swell %	Potential Swell Classification
0.5	low
0.5-1.5	Marginal
1.5	High.

Furthermore he evaluated seventeen published techniques selected for consideration using the described definitions and classification of potential swell. The evaluation showed that the most consistent indictors of potential swell are first, the Liquid Limit and Plasticity Index, second Liquid Limit and Natural Water Content etc,.

Finally, he showed that using the measured potential swell categories, the categories of indicator properties which maximize the accuracy and conservatism of the criteria for identifying and qualitatively classifying expansive soils are as in Table 2.3.

Table 2.3: Criteria for identifying and classifying expansive soils by Snethen (1984)

Petential Swell Classifcation.	Potential Swell %	LL %	PI %
High	1.5	60	35
Marginal	0.5-1.5	50-60	23-35
Low	0.5	50	25

Driscoll et al (1984) investigated the geotechnical properties of expansive clay soils in Northern Jordan. Their investigations were carried out in the Jordanian capital, Amman, and in the towns of Irbid, to the north and Abu Nusseir. There were many cases of damaged buildings in these areas and soil sampling sites were chosen within these locations to coincide with known occurences of damage to low rise buildings.

Identification test on the samples showed the samples to be predominantly clay of similar texture. On the casagrande plasticity chart, these samples were found to exhibit high, very high and extremely high plasticity.

Index properties and clay fraction were used to clasify the volume change potential of the soils and the result shows that except for a few of the samples which are of "medium" potentials, the others are in the "high" to "very high" categories. The greatest potential was found to occur in the Abu Nuseir clay. These characteristics correlate well with the mineralogy of the clay fractions, with Abu Nuseir, Irbid and Amman samles exhibiting respectively decreasing proportions of the interstratified mica-smectite mineral.

Measurement of moisture content variation with time and depth below ground surface, at both Amman and Irbid, showed that seasonal climatic changes extend to at least 2m.

Oedometer swelling and consolidation tests indicated that within the top 1m swelling pressures in excess of 200kN/m^2 are likely. The extremely plastic green clay of Abu Nuseir was found to generate significantly higher swelling pressures than the mantle clays elsewhere. Preconsolidation pressures indicated substantially overconsolidation in the top 2 - 3.5m on all sites, so that, for the lightly-loaded foundations for low rise buildings in Jordan, the expansive nature of the clay soils constitutes the main geotechnical problem.

Mitchell (1984) developed a simplified method of design of shallow footings on expansive soil. He reviewed past design methods put forward by Brab (1968), Lytton (1977), Walsh (1978), Swinburne (Holland 1981), PT I - Wray (Wray (1980) and Mitchell (1979). His review indicated a variation in the answers obtained by each method. He thus showed that the difference in the designs appear to be more a result of differences in the imput parameters of each method, than the complexity of the method used for the actual interaction analysis. Based on this observation, he put forward one simple design method. The method has been successfully used by the author for the routine design of shallow stiffened footings of structures on expansive soils for at least five years. The method is based on a simplification of the structure geometry, load and permissible deflection, and is derived from an integration of the beam on mound equation.

Algebraic expressions for the bending moment at any point on the footing, together with the required stiffness were obtained for both centre and edge heaves.

A paremetric study revealed that each of the design parameters (i.e. structure geometry, loads, soil movement, permissible deflection, shape of initial soil surface and swelling stiffness) have a variable effect on the bending moment in the footing. The behaviour of a heavily loaded residential raft footing undergoing severe distortion was shown to be successfully modelled by the analysis.

Holland and Richards (1984) carried out investigation on the practical design of foundations for lightly loaded structures on Expansive soils. The authors considered a design approach which considers the clay properties, the footing cost and environmental factors associated with footing performance.

They showed that although the strip footing systems are used extensively throughout the World, their performance is poor and no rigorous design procedure for strip footings on expansive clay soils exist. They affirmed that most common strip footings are designed from experience and from a consideration of allowable bearing pressures. Bearing pressures however is rearly a problem for expansive clays even after making allowance for their significant, seasonal moisture changes.

In considering the pier and beam systems, the authors submitted that well designed pier and beam foundation systems appear to provide an acceptable level of performance when a timber floor system is unavoidable. Many pier and beam foundations are still incrrectly designed on the basis of bearing pressure rather than founding them at depths where no heave will occur beneath them.

According to the authors, piers should be founded on at least twice the depth of seasonal soil movement or on stable material such as bedrock. Suspended beams between these piers must be effectively separated from the expansive clay soil. This requires the use of an effective void former which is usually a collapsible cardboard. The selection of a suitable void former is shown to be critical to the performance of this footing system.

In considering slab foundations, the authors showed that the use of integral raft slabs on expansive clays is recognised throughout the world as being the most satisfactory, economical foundation system for relatively flat sites. Finally they concluded that the behaviour of a footing on expansive clay is not as predictable as structural engineers might assume and hence rigid application of design models and exacting computations is of little value. Rather a more appropriate approach begins with a soil report that considers simple clay properties, depths and thickness and experience to provide an estimate of the likely seasonal heave in the open. The final design must include restrictions or warnings regarding landscaping and long term site and plumbing maintenance, if realistically priced footings are desired.

Driscoll (1984), Shiming (1984), Longworth (1984), Ponlos (1984), Kumar (1984), Elturabi (1984), Williams (1980) carried out extensive investigations on the identification methods and geotechnical properties of expansive soils. A careful study of the various results obtained showed that properties of the expansive soils and foundation design methods varied with locations of the soil.

Driscoll (1984) in particular showed that for lightly loaded foundations for low rise buildings, the expansive nature of the clay soils and not bearing capacity consideration constitutes the main geotechnical problem.

Research works on foundations on expansive soils in Nigeria are limited. An extensive literature review indicates that although some works on the geotechnical characterization of some of the potentially expansive soils like the black catton soils and the Sokoto soft clay have been carried out by Ola (1980) and De Castro and Ola (1983), most of these works, have been directed towards the use of such soils and information obtained in highway pavement designs. Therefore most of the tests carried out by the above mentioned writers are not enough to enable adequate design of foundations on expansive soils to be carried out.

The work so far reviewed showed that the most likely cause of severe cracks on buildings within the Sokoto Rima River Basin Development Authority Headquarters would be the presence of collapsible or expansive soils below the foundations to the buildings. It has also been shown that a measure of experience has been gained in the area of geotechnical characterization of collapsible and expansive soils. This experience has not been tested in Nigeria. Thus the response of collapsible and expansive soils in Nigeria to some of the findings in the research reviewed is yet to be determined. In most of the works reviewed, the solutions proposed will involve very high cost on the part of the client. Also it is observed that moisture content variation is a major factor in the behaviour of both expansive and collapsible soils.

Efforts is therefore directed towards studying the variation of moisture content with depths during both the rainy and dry seasons, with a view to better understanding the variation of moisture content with depths across the seasons. Jennings and Knight (1957) and Hollands and Richards (1984) showed that there is a significant reduction in bearing capacity for collapsible soils and expansive soils respectively as moisture content is increased. This spectacular phenomenon will be studied with a view to observing the variation in shear strength with moisture content for the Nigerian situation.

From the works reviewed, it is shown that excessive differential settlement or heave is the one most important factor that is responsible for failure of foundations on collapsible and expansive soils. These settlements, are induced by variation in moisture content of soil under the building. This variation in moisture content results in different amount of settlement or heave of the various footings with resultant cracks on the sperstructure. It is therefore important that for a good understanding of the soil in question, the variation of settlement or heave with moisture content be investigated. Finally the works so far reviewed have shown that there is a correlation between the Index properties of a soil and its classification. It is therefore expdient that the index properties of the soil be first investigated in order to determine whether the soil under investigation is an expansive soil or a collapsible soil. Having identified the soil, strength test should be carried out to further understand the behaviour of the soil with respect to the various factors earlier mentioned. Finally it is very pertinent to point out that lightly loaded foundations on collaps'ble or expansive soils have received little or no attantion in Nigeria.

Worst still is the fact that the Sokoto experience is virtually non-existent. It is the realization of this neglect that necessitated the investigation of Design Criteria for lightly loaded Foundations - The Sokoto experience.

CHAPTER THREE

3. LABORATORY TEST PROGRAMME AND PRESENTATION OF EXPERIMENTAL RESULTS

3.1 Test Programme

The test programme embarked upon was designed to provide sufficient information on the geotechnical characteristics which were used to understand the behavior of the soil.

Undisturbed samples from four pits within the premises of the Sokoto Rima River Basin Development Authority were obtained. The first three pits were excavated towards the end of the long dry season (i.e. in April and May) in 1988, while the fourth pit was excavated in late July, 1988. The need for the fourth Pit is only to give an indication of the variation of moisture content with depth during the rainy season. Best result would probably have been obtained, if the fourth sampling was undertaken at the end of August or early September, but this would have unduly prolonged the work. Furthermore the variation of moisture content with depth served only as a guide in discussing the results obtained and did not jeopardise the results.

Six samples were taken from each pit at depths of 0.5m, 1.0m, 1.5m, 2.0m, 2.5m and 3.0m respectively. Since foundations under consideration are for lightly loaded buildings, they are not expected to be founded at more than 1.50m from the Natural Ground Level (N.G.L.). Furthermore, since the width of these footings are not expected to be more than 1.25m, stresses induced by them will be negligible at depths greater than 2B from foundation depth (i.e. at depths greater than 4.0m). However, the cost of excavation and the time to excavate this type of soil by manual means made it impossible to contemplate any depth greater than 3.0m.

After excavating to the desired depths, samples were taken using core cutters. The samples so removed were then sealed with wax, and transported to the laboratory. Since two samples were usually taken from every depth and only twelve core cutters were available from the soil mechanics laboratory, this meant that only one pit could be sampled on any visit to Sokoto. Generally, excavation of one pit took three labourers about three days to finish, because of the dry gravelly nature of the soil.

For the samples obtained, the laboratory tests performed may be grouped under two broad headings:

(i) Classification and Identification tests;

(ii) Tests for engineering properties. (Strength Test).

3.2. Classification and Identification Tests

These tests were carried out to facilitate accurate geotechnical classification of the soils. All experiements in this group were carried out in accordance with the provision of B S 1377, and the experiments were as followos:

(a) Moisture Content determination

(b) Specific Gravity Test

(c) Atterberg Limits Tests

(d) Particle size distribution by Wet Sieving

3.2.1 Moisture Content Determination

The moisture content test was carried out with a view to determining the moisture content variation with depth. As earilier stated, the addition of moisture is the most important factor that causes collapse or heave of the studied soil type. The test proceedure followed the standard laboratory methods as desribed by BS 1337 (1975).

It is important to note that during excavation it was observed that pit 2 was generally damp and the worst cracks within the building complex were noticed arround this area.

Tables 3.1 shows the variation of moisture content with depth for pits 1, 2 and 3 respectively. These moisture content values are for samples obtained in April and May, 1988, they therefore represent moisture content variation with depth during the dry season. Table 3.2 shows the variation of moisture content with depth for pit 4. The samples for pit 4 were taken during the rainy season. A comparison of moisture content results for pits 1, 2, and 3 against that of pit 4 gives a clear indication of the variation of moisture content with depth across the seasons. This comparison is best appreciated in a graphical form. Thus graphical illustration of Tables 3.1 and 3.2 is given as fig. 3.1.

A close look at the moisture content for pit 2 of table 3.1 shows higher moisture content values. This is in confirmity with observations made while excavating pit 2. Further investigation revealed that the soakaway pit and septic tank to the toilets were just about 3.0m from the location of pit 2. It is therefore concluded that the proximity of the soakway pit to test pit 2 is responsible for the higher moisture content values obtained for pit 2. Details of experimental results obtained in the course of these tests are as shown in the data sheets given in Appendix 1.

SUMMARY OF MOISTURE CONTENT RESULTS.

TABLE 3·1 : VARIATION OF MOISTURE CONTENT WITH DEPTH DURING DRY SEASON

DEPTHS	MOISTURE CONTENT			
	PIT 1	PIT 2	PIT 3	AVERAGE
0·5M	8·37	7·34	3·60	6·43
1·0m	9·20	7·80	8·40	8·47
1·5m	9·76	8·90	9·93	8·65
2·0m	10·05	9·80	8·78	9·54
2·5m	10·27	12·54	9·37	10·73
3·0m	—	12·36	8·85	10·6

TABLE 3·2 : VARIATION OF MOISTURE CONTENT WITH DEP DURING RAINY SEASON

DEPTHS (m)	MOISTURE CONTENT PIT 4
0·5	10·4
1·0	11·4
1·5	12·1
2·0	13·0
2·5	12·7
3·0	11·9

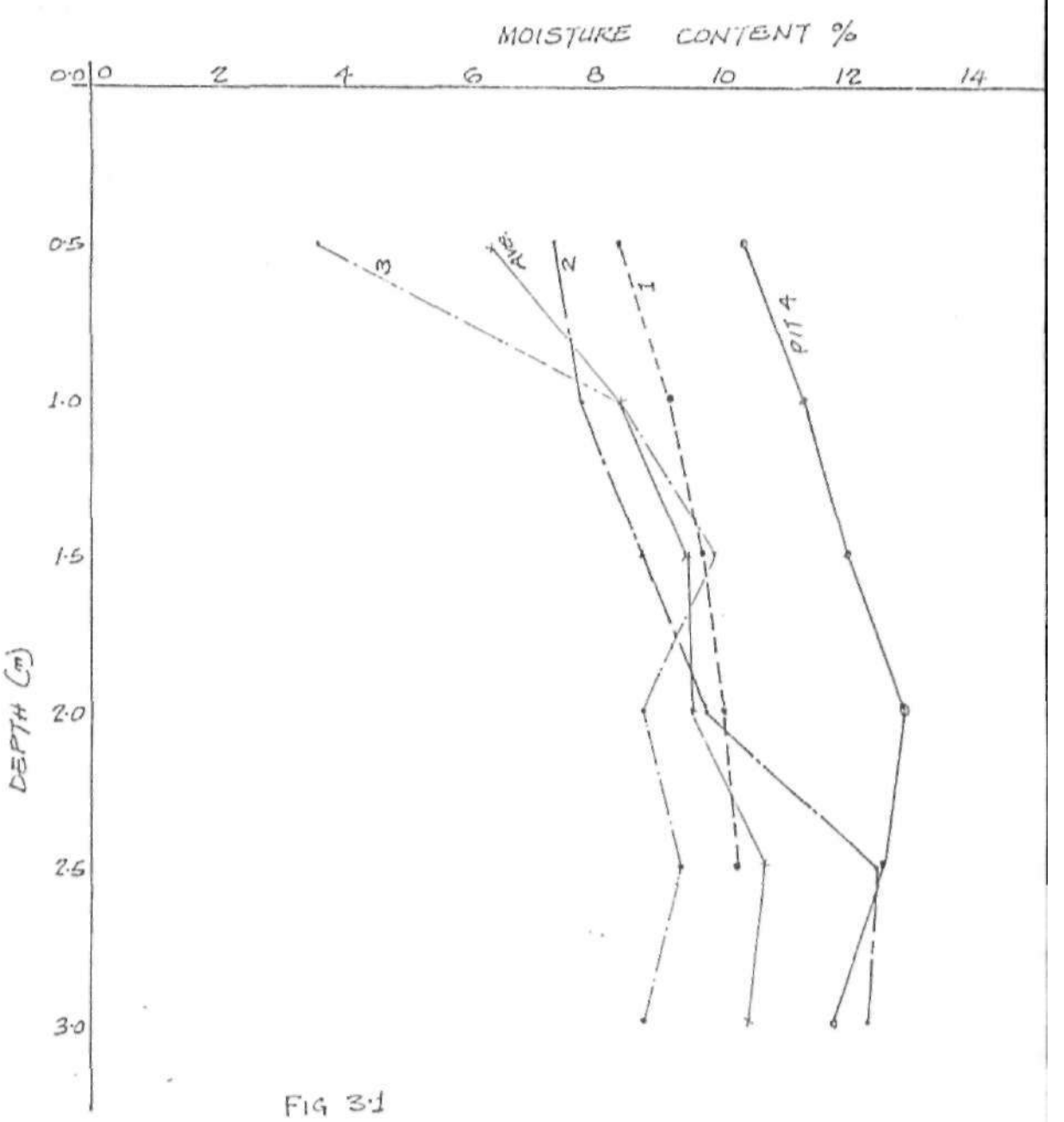


FIG 3·1
VARIATION OF MOISTURE CONTENT WITH DEPTH

SCALE : VERTICAL 5 cm = 1·0 m

: HORIZONTAL 1 cm = 1 %

- PIT 1
- PIT 2
- PIT 3
- AVERAGE OF PIT 1, 2, & 3
- PIT 4.

3.2.2. Specific Gravity Determination

For the three test pits under consideration, specific gravity tests were carried out on samples taken at depths between 0.5m to 3.0m at incremental sampling depths of 0.5m. The results from this test was used to classify the soils. Furthermore, the results of specific gravity test enable quick determination of the unit weight of the soil sample and an easy calculation of the void ratio of samples.

The results of specific grzvity test on the samples are shown in tables 3.3. The procedure for the determination of specific gravity are as stated in BS 1377 (1975). Details of experimental results are as shown in the data sheets of Appendix 2.

TABLE 3·3

TITLE : SPECIFIC GRAVITY OF SOIL PARTICLES

DEPTH (m)	SPECIFIC GRAVITY			
	PIT 1	PIT 2	PIT 3	
0·5	2·72	2·81	2·64	
1·0	2·65	2·64	2·60	
1·5	2·89	2·81	2·91	
2·0	2·68	2·74	2·75	
2·5	2·69	2·64	2·81	
3·0	2·77	VALUE NOT CONSISTENT WITH OTHERS	2·73	
	2·73	2·74	2·74	AVERAGE

HENCE

GENERAL AVERAGE SPECIFIC GRAVITY IS

$$G = \frac{2\cdot73 + 2\cdot74 + 2\cdot74}{3}$$

$$= \underline{2\cdot74}$$

3.2.3. Atterberg Limits Determination

The three main Atterberg Limit tests namely, Liquid Limit, Plastic Limit and Shrinkage Limit have been widely used all over the world for soil identification and classification. The shrinkage limit is particularly useful in areas in which the soils undergo large volume changes when going through wet and dry cycles. These limits which were proposed by A. Atterberg a Swedish agricultural scientists are defined as follows:

Liquid Limit: This is the moisture content below which the soil behaves as a plastic material. At this moisture content, the soil is on the verge of becoming a viscous fluid.

Plastic Limit: This is the moisture content below which the soil is non-plastic.

Shrinkage Limit: This is the moisture content below which no further soil volume reduction occurs.

Potential volume change problems can often be detected from the liquid and plastic limit tests. The liquid limit is sometimes used to estimate settlement in consolidation problems. Thus the result of the Atterberg limit test will help in effectively classifying the soil.

Soil samples taken between 0.5m and 3m of each of test pit 1, 2 and 3 were tested in the laboratory for their Atterberg Limits. The result of the tests are as shown in Appendix 3 while a summary of results are shown in Table 3.4.

Fig. 3.2 shows a plot of the Atterberg limit results on the plasticity chart.

TABLE 3·4.

SUMMARY OF ATTERBERG LIMITS TEST RESULTS.

DEPTH	PIT 1			PIT 2			PIT 3		
	LL	PL	PI	LL	PL	PI	LL	PL	PI
0·5m	31	11·82	19·18	23·0	9·43	13·57	30·00	11·95	18·05
1·0 m	30·0	10·17	19·83	34·0	13·33	20·67	31·00	7·25	23·75
2·0m	25·0	11·65	13·35	30·00	13·00	17·00	32·5	15·39	17·11
3·0 m	NOT AVAILABLE			33·00	12·26	20·74	27·00	17·50	9·50

SILT (M-SOIL), M, plots below A-line
CLAY, C, plots above A-line

M and C may be combined as FINE SOIL, F.

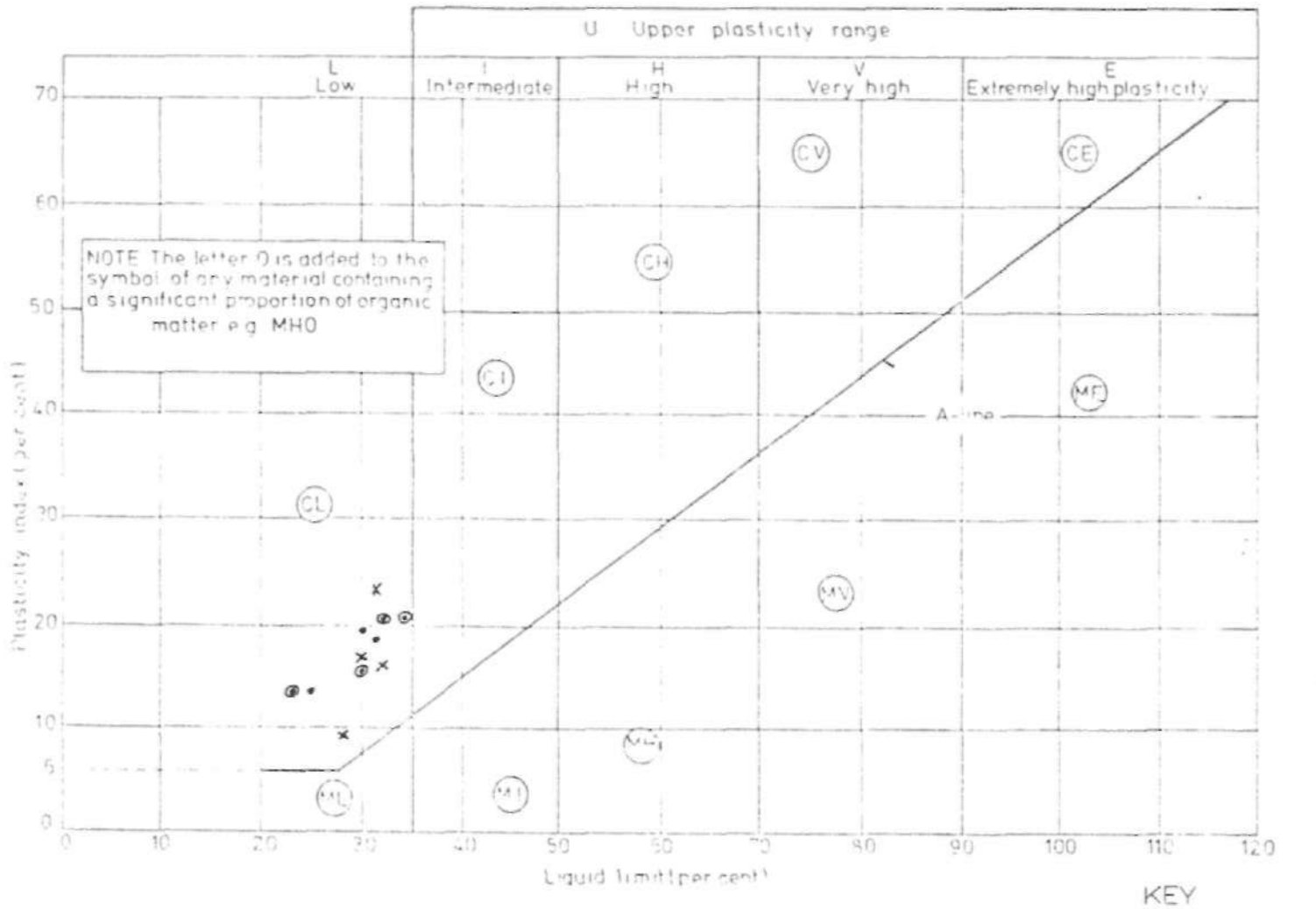


Figure 3.2 Plasticity chart for the classification of fine soils and the finer part of coarse soils (measurements made on material passing a 425 μm BS sieve)

3.2.4 Particle Size Analysis

In order to obtain the particle size distribution of this soil, the standard method of sieve analysis as given in BS 1377 was carried out. The method used here which is the most reliable and most easily reproducible method is the Wet Sieving method. This method involves the weighing out of 200g of oven dry samples, carefully washing it through the No. 200 sieve using tap water, then oven drying the residue for twenty four hours. After drying the sample, it is then ran through a stack of sieves of varying sizes for 10 minutes. After sieving, the weight of soil retained on each sieve is determined.

The alternative method of obtaining the particle size distribution of the soil, is the Dry Sieving method. In this method, the whole sample is dried and passed though the sieves without any of the fine material being first extracted by washing. This is a faster method, but not as accurate as the Wet Sieving method. Both methods are listed in BS 1377.

In this particular experiment, two samples from each of test pits 1,2 and 3 were tested. The result of the test are given in the data sheets of Appendix 4. These results are ploted as partcule size distribution curves of Appendix 4, while a typical curve is hereby attached as Fig. 3.3.

Location SOKOTO Boring No PIT 2 (2.5m) Sample No 2 Date of Test

Description Reddish Brown Well Graded Clayey Gravel

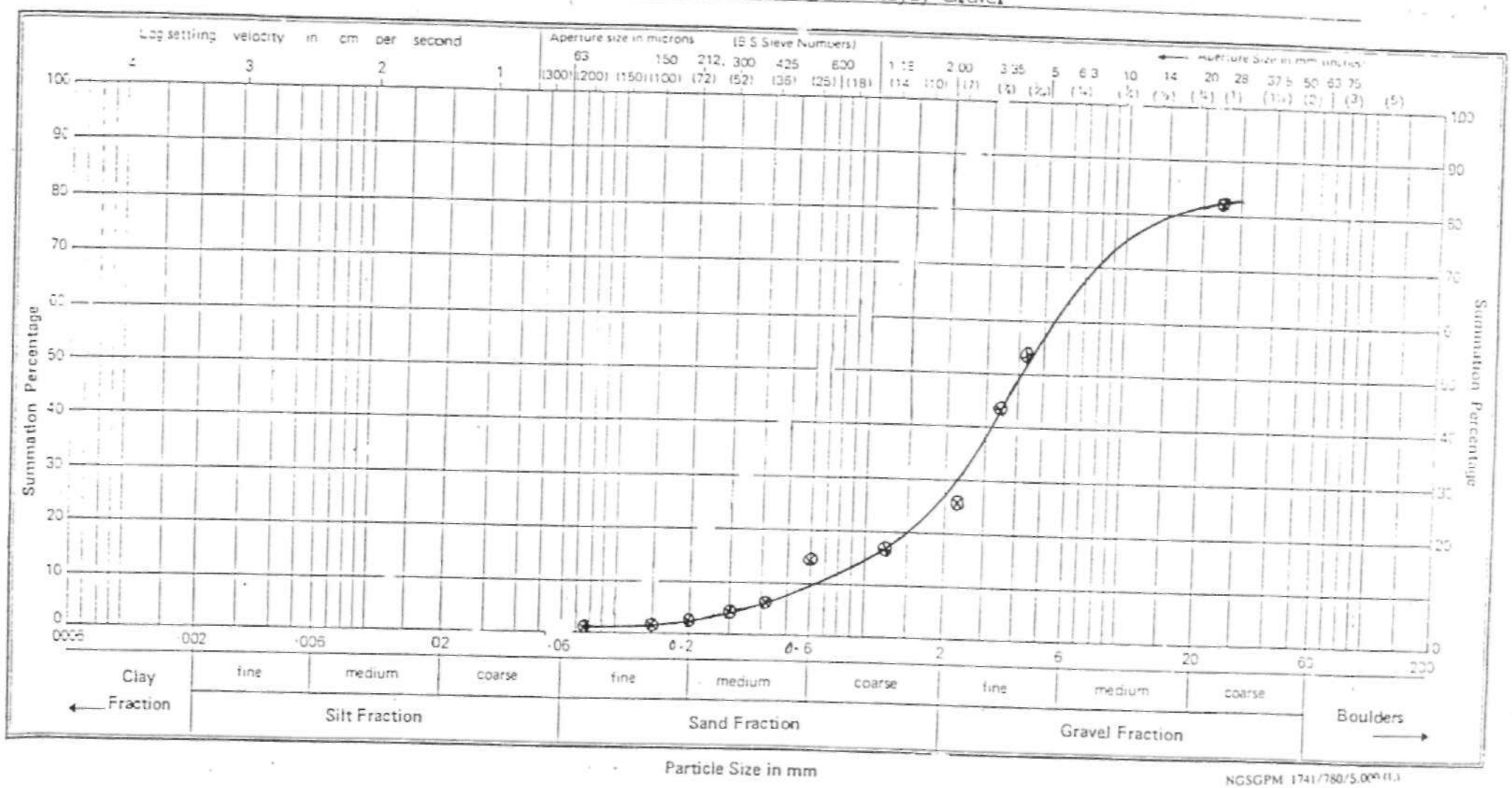


Fig: 3.3: Particle Size Distribution curve for Pit 2 at 2.5m.

3.2.5. Coeffecient of Concavity and Uniformity

To obtain better classification of the soil based on their physical characteristics, the coefficient of uniformity and concavity of the soil are to be determined.

The coefficient of uniformity C_u gives an indication of the spread(or range) of grain sizes and is define as:

$$C_u = \frac{D_{60}}{D_{10}}$$

Where D_{10} which is usually refered to as the Effective size is defined as the maximum particle size of the samllest 10 percent of the samples. This figure is obtained by reading off from the grain size distribution curve, the size at which the grading curve cuts the horizontal line through 10 percent.

D_{60} is defined as the maximum size of the samllest 60 cercent of the samples.

The uniformity coefficient C_u gives an indication of the spread (or range) of grain sizes in the soil.

The coefficient of concavity is a measure of the shape of the curve between the D_{60} and D_{10} grain sizes, and is defined as:

$$C_c = \frac{D^2_{30}}{(D_{10}D_{60})}$$

Values of C_c greatly different from 1.0 indicates grain sizes missing between the D_{60} and D_{10} sizes.

Thus for the three test pits under consideration, the value of the coefficient of uniformity and concavity are as calculated next page.

PIT 1 - 1.0m

D_{10} = 0.17

D_{30} = 1.75

D_{60} = 3.90

Coefficient of uniformity $C_u = \frac{D_{60}}{D_{10}} = \frac{3.90}{0.17} = 23$

Coefficient of Concavity $C_c = \frac{D^2_{30}}{D_{60}D_{10}} = \frac{1.75^2}{3.90 \times 0.17} = 4.6$

PIT 1 - 2.5m

D_{10} = 0.36

D_{30} = 1.60

D_{60} = 3.2

$C_u = \frac{D_{60}}{D_{10}} = \frac{3.2}{0.36} = 8.9$

$C_u = \frac{D^2_{30}}{D_{60} \times D_{10}} = \frac{1.6^2}{3.2 \times 0.36} = 2.2$

PIT 2 - 1.0m

D_{10} = 0.10

D_{30} = 3.0

D_{60} = 7.0

$C_u = \frac{D_{30}}{D_{10}} = \frac{7}{0.1} = 70$

$C_c = \frac{D_{30}}{D_{60} \times D_{10}} = \frac{3^2}{7 \times 0.1} = 12.9$

PIT 2 - 2.5

D_{10} = 0.6

D_{30} = 2.3

D_{60} = 5.5

$$C_u = \frac{D_{60}}{D_{10}} = \frac{5.5}{0.6} = 9.2$$

$$C_c = \frac{D^2_{30}}{D_{60} \times D_{10}} = \frac{2.3^2}{5.5 \times 0.6} = 1.6$$

PIT 3 - 1.0

D_{10} = 0.85

D_{30} = 3.50

D_{60} = 7.50

$$C_u = \frac{D_{60}}{D_{10}} = \frac{7.50}{0.85} = 8.8$$

$$C_c = \frac{D^2_{30}}{D_{60} \times D_{10}} = \frac{3.5^2}{7.5 \times 0.81} = 1.9$$

PIT 3 - 2.5m

D_{10} = 0.18

D_{30} = 3.0

D_{60} = 5.0

$$C_u = \frac{D_{60}}{D_{10}} = \frac{5.0}{0.18} = 27.80$$

$$C_c = \frac{D^2_{30}}{D_{60} \times D_{10}} = \frac{3.0^2}{5 \times 0.18} = 10.0$$

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TABLE 3·5

SUMMARY OF VALUES OF COEFFICIENT OF UNIFORMITY C_u AND COEFFICIENT OF CONCAVITY C_c

DEPTH (m)	PIT 1		PIT 2		PIT 3	
	C_u	C_c	C_u	C_c	C_u	C_c
1·0	23	4·6	70·0	12·9	8·8	1·9
2·5	8·9	2·2	9·2	1·6	27·8	10·0

3.2.6. Identification and Classification of Soil

Field observation showed that the soils excavated from the three test pits under consideration are reddish brown in colur. The Sieve Analysis results (Appendix 4) shows that in all the samples tested, less than ten percent of each sample taken passed through the sieve No. 200. This confirms the non-plastic nature of the soil and makes a hydrometer analysis unnecessary. The sieve analysis results plotted out as particle size distribution curves (Appendix 4) showed the following characteristics:

(a) PIT 1: About 94% of soil samples tested are larger than No. 200 sieve size. Also about 65% of the soil is retained on No. 7 sieve size.

PIT 2:
(b) The particle size distribution curve shows that about 95% of the soil tested is larger than No. 200 sieve size and about 75% is retained on the No. 7 sieve size.

(c) PIT 3: it is observed from the results of sieve analysis that more than 95% of the soil is retained on the No. 200 sieve size while more than 75: is retained on the No. 7 sieve size.

Classification of the soil in accordance with the British Soil Classification System for Engineering Purpose (Table 8 BS5930 1981). For all the samples tested, less than 35% of each of the samples is finer than 0.06mm (i.e sieve No. 200). Thus the soil is a coarse soil. Also more than 50% of each of the coarse materials tested was retained on 2mm sieve (i.e sieve No. 7). Thus indicating that the soil is Gravelly. It was further observed that the fine particles present in all the samples are less than 10% but generally more than 5% . The soil therefore contains a measure of fine soils and is either a silty Gravel or a clayey Gravel.

From a plot of liquid limit values against plasticity index values (fig. 3.2) as obtained from table 3.4, it is observed that all the points plotted fall above the A - line. Thus indicating that the fine material is clay. A look at the uniformity coefficient values obtained from Table 3.6 shows that the values are generally high. Since none of the uniformity coefficient values is less than 4, this indicates that the soil is well graded.

Hence the soil under consideration is classified as Well graded clayey Gravel GWC. Since the fine particle contained in this soil is less than 10% and maximum plasticity index is less than 25, the soil could not possibly be an expansive soil. Hence a Well Graded Clayey Gravel exhibiting collapsing characteristic is established. The clay present acts as a binder for the Gravel particles, and may absorb water and become weak when submerged.

3.3 Strength and Deformation Tests

These are tests to determine the Strength and Deformation characteristics of the soil. The tests under this group are:

(a) Compaction Test

(b) Undrained Triaxial compression Test

(c) Oedometer consolidation Test.

It is important to state that although undisturbed samples were obtained from the site, most of the tests were carried out on disturbed samples. This is because the gravelly nature of the soil during the dry season made it impossible to extrude stable samples from the core cutters. Thus, no undisturbed core samples were obtainable. Furthermore, the identification and classification tests on the three test pits under consideration have shown that the soils in the three test pits are virtually the same. Thus to reduce cost and save time the number of tests were limited to samples from only two test pits.

3.3.1 Standard Proctor Compaction Test

In order to determine the maximum dry density of the soil and the optimum moisture content, the Standard Proctor Compaction Test was performed on one sample each from the three test pits. In this test, a cylinderical mould 950mm^3 in volume is filled in three layers, each layer of the soil being compacted by 25 blows of a standard rammer which has a mass of 2.5kg and drops 300mm at each blow. The mould is fitted with a detachable collar which enables the soil to be compacted to a level slightly above the rim of the mould itself. The collar is then removed and the surface carefully struck off level with the rim.

The bulk density is obtained by weighing the mould, and a small portion of the soil is taken for moisture content determination. From these measurements the dry density is calculated. The test is repeated for a wide range of moisture contents, and a curve is plotted showing dry density versus moisture content. From this the maximum day density and optimum moisture content are read off.

Also from the results obtained, the Zero-Air-Void curve could be plotted for each test. Also since the specific gravity of the soil is known, the void ratio corresponding to each test point is obtained.

The results of the compaction test carried out on samples from test pits 1, 2 and 3 are given in the test data sheets of Appendix 5, while a summary of these results are given in Table 3.6 and Table 3.7 attached.

3.3.2. Triaxial Tests

Triaxial tests were run on eighteen samples taken from the test pits. The aim of the test was to enable a good determination of the engineering parameters of the soil to be obtained. As earlier stated, it was impossibe to obtained undisturbed samples from the core because of the nature of the soil which crumbles on extruding. Thus remoulded samples at the optimum moisture content and maximum dry density were prepared from samples taken from pits 2 and 3. Unconsolidated undrained triaxial tests were carried out on these samples. For these tests, only samples taken from 2.0m and 3.0m depths respectively were used.

To further investigate the effect of collapse due to inundation with water on the samples, additional samples were taken from test pit 3 and prepared at moisture content levels higher than the optimum moisture content. Similar triaxial tests were also carried out on these samples and the effect of increase in water content on the shear parameters of the soil were noted.

It is important to state that the difference between the field moistur content values during the dry season (Table 3.1) and the optimum moisture content values (Table 3.6) is very minimal. Hence the use of samples at the optimum moisture content gives a good estimation of the shear parameters of the disturbed soil at the field moisture content.

The shear strength obtained from these are considered to give very conservation estimates of the shear strength conditons obtaining in the field.

Bearing capacities calculated with these set of parameters are likely to give the most conservative estimates of the allowable foundation bearing capacities.

The triaxial tests were carried out as follows:-

PIT 2 A total of 6 samples tested.

(a) 3 samples at Optimum Moisture Content of 11% were taken at a depth of 2.0m.

(b) 3 samples at Optimum Moisture Content of 11% were taken at a depth of 3.0m.

PIT 3 A total of 12 samples tested.

(a) 3 samples at Optimum Moisture Content of 10% taken at a depth of 2.0m.

(b) 3 samples at Optimum Moisture content of 10% taken at a depth of 3.0m

(c) 3 samples at a moisture Content of 15% taken at a depth of 2.0m.

(d) 3 samples at a moisture content of 15% taken at a depth of 3.0m

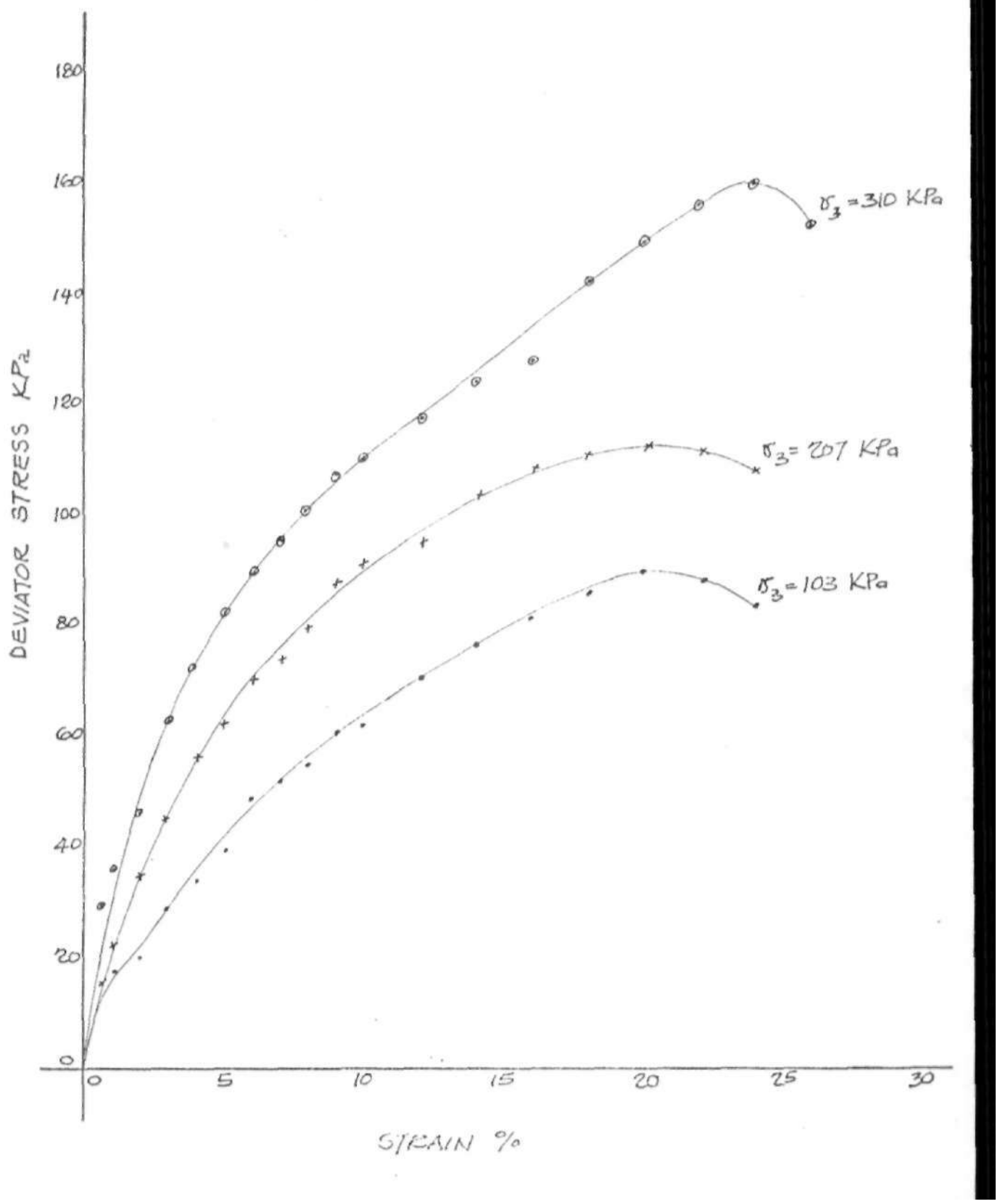
A summary of the result of this investigation is given in Table 3.8, while the detailed laboratory results, mohr circle are given in Appendix 6. A typical mohr circle and graph of Deviator Stress Versus Strain is here attached.

TABLE 3-8

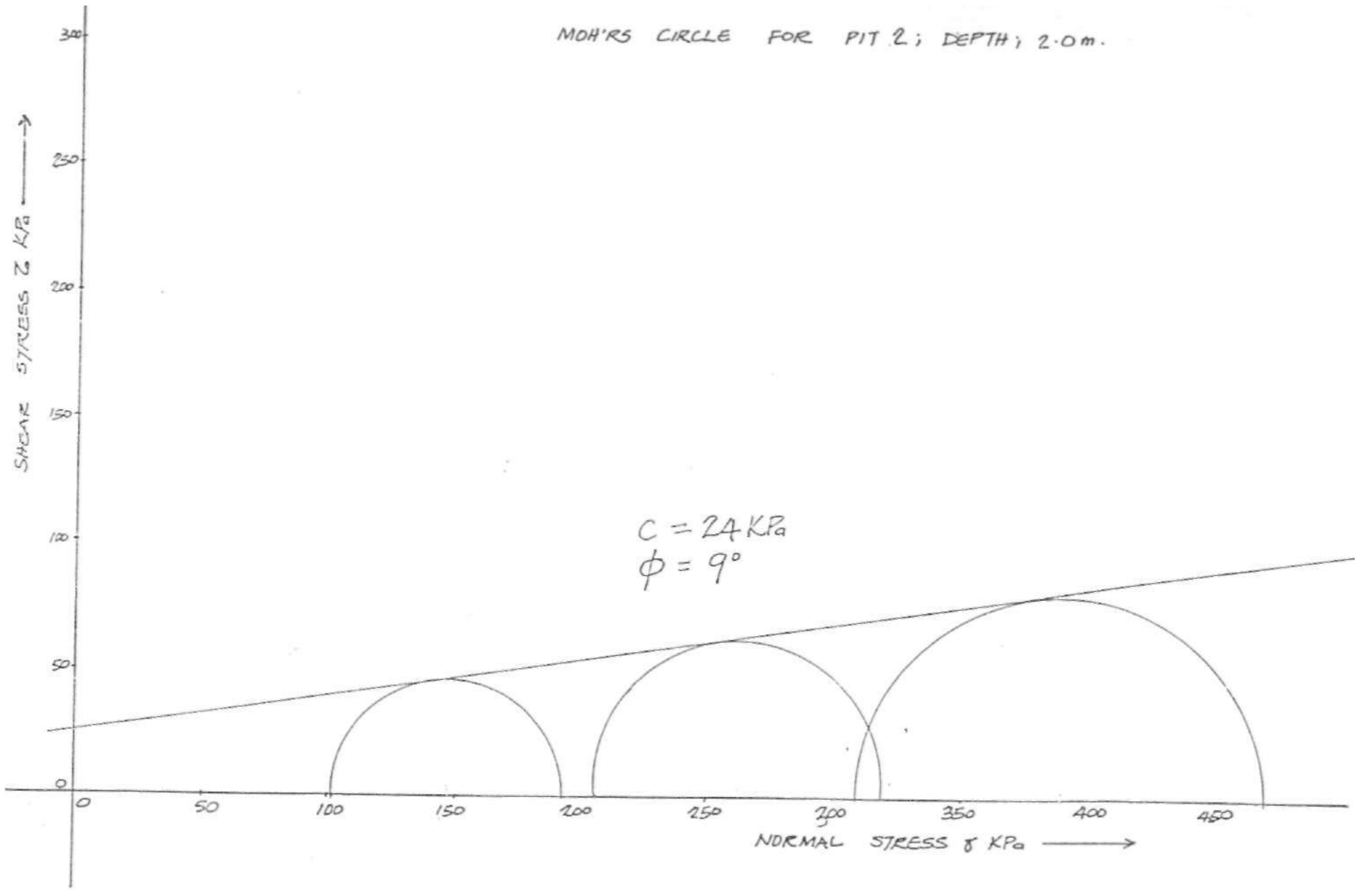
SUMMARY OF TRIAXIAL TEST RESULTS

Depth (m)	PIT 2 Undrained Tests at OMC, MDD		PIT 3 Undrained Test at OMC, MDD		PIT 3 Undrained Test at moisture Content = 18%	
	Cu (KPa)	ϕ degree	Cu (KPa)	ϕ degree	Cu (KPa)	ϕ degree
1.0	24	9	26	24	16	7
2.0	20	10	32	10	19	6

GRAPH OF DEVIATOR STRESS Vs STRAIN % FOR PIT 2 DEPTH 2·0



MOH'RS CIRCLE FOR PIT 2; DEPTH; 2.0m.

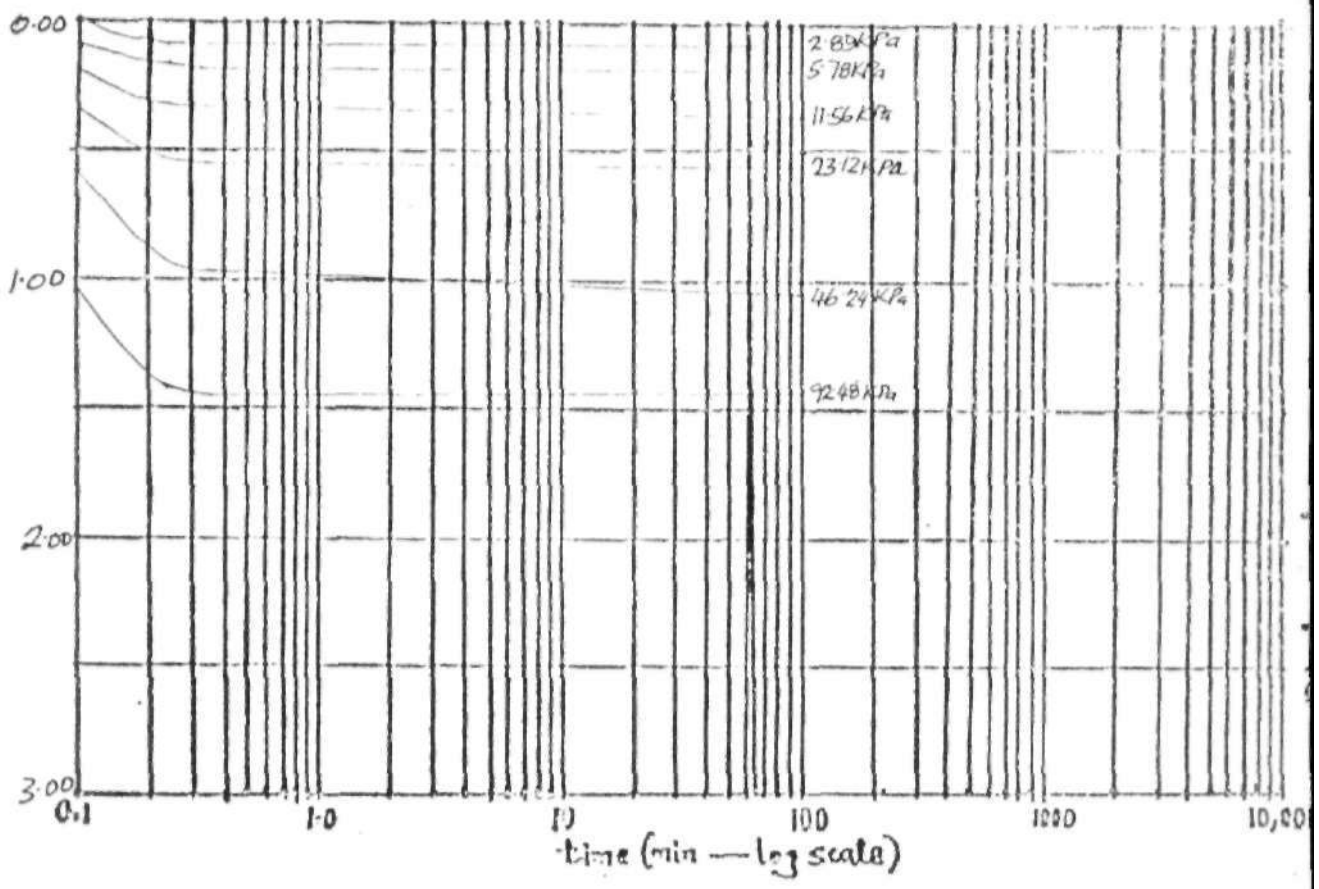


3.3.3. Oedometer Consolidation Tests

As can be seen from the results of the earlier tests carried out and in particular from the results of tests to determine the physical properties of the soil, it can safely be concluded that the soil in question is not a clay soil. Hence the aim of an Oedometer test is not primarily to determine the rate of settlement, but to study the effect of change in moisture content on settlement. This test is aimed at confirming that the soil is of the collapsing type. In addition, however, from the compression versus time relationship for various load increament, the settlement characteristics of the soil will be observed.

To achieve the above stated objectives, three Oedometer tests were carried out on three samples of the soil at different initial moisture contents. Each sample was subjected to the same incremental loading conditons and their response to these loads were recorded. The result so obtained gives a good understanding of the relationship between moisture content and collapse of the soil.

The three samples tested were at initial moisture content of 9.67, 13.68 and 15.9% respectively. The loads applied on the samples are 1.13kg, 2.26kg, 4.52kg, 9.04kg, 18.08kg, 36.16kg, 72.32kg respectively and each load was sustained for 24 hours before being increased. The results of the tests are as shown in the test Data sheet, of Appendix 7 while a graph showing the variation of void ratio with pressure for the three values of moisture content chosen is given in Fig. 3.4. Fig. 3.3a - 3.3c shows the variation of settlement with time for the three samples chosen.



Fic: 3.3a: Compression Vs Time (log Scale) Graph for moisture content m= 9.67%

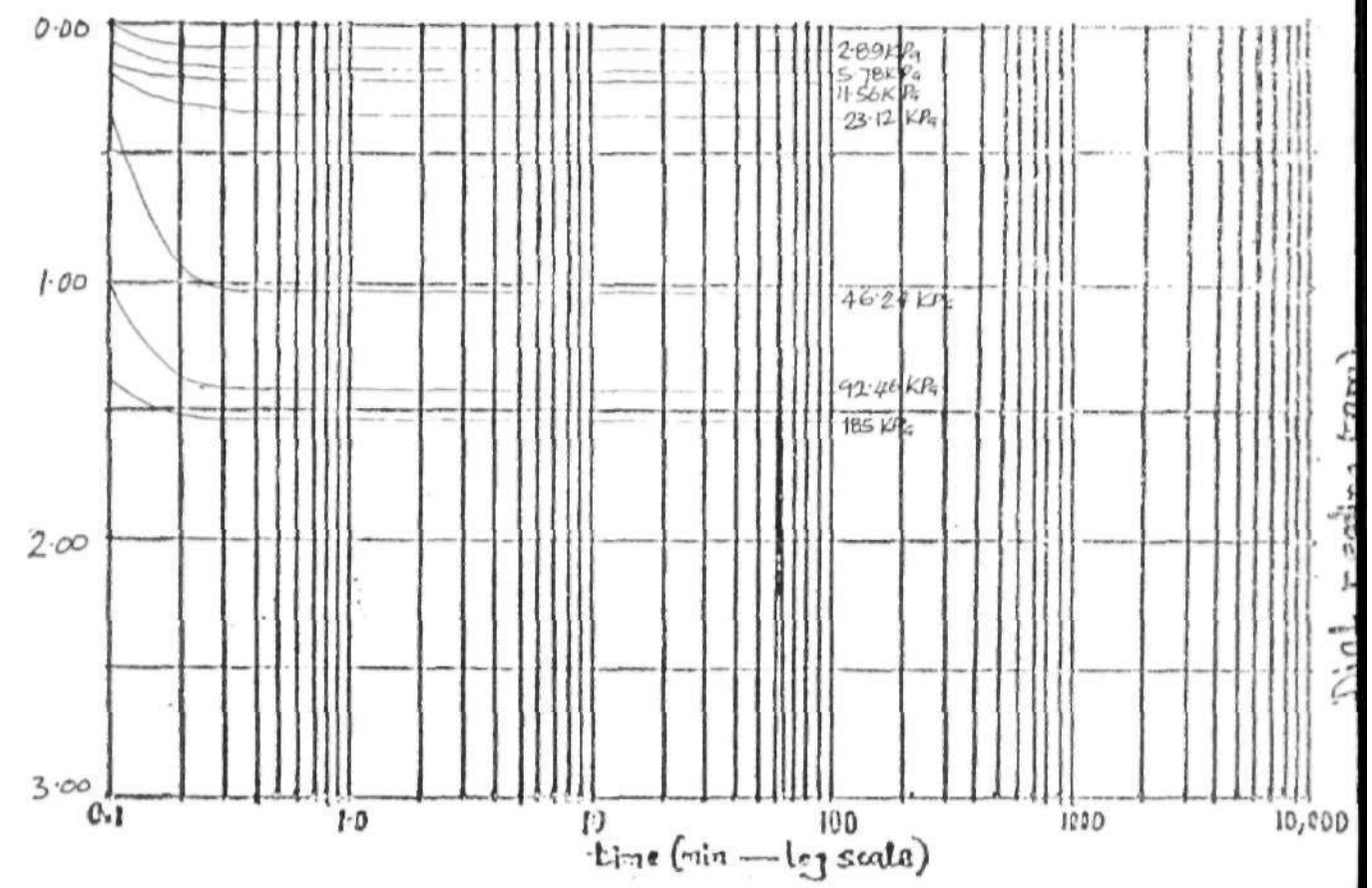


Fig. 3.3b: Compression Vs Time (log Scale) graph for moisture content m = 13.68%

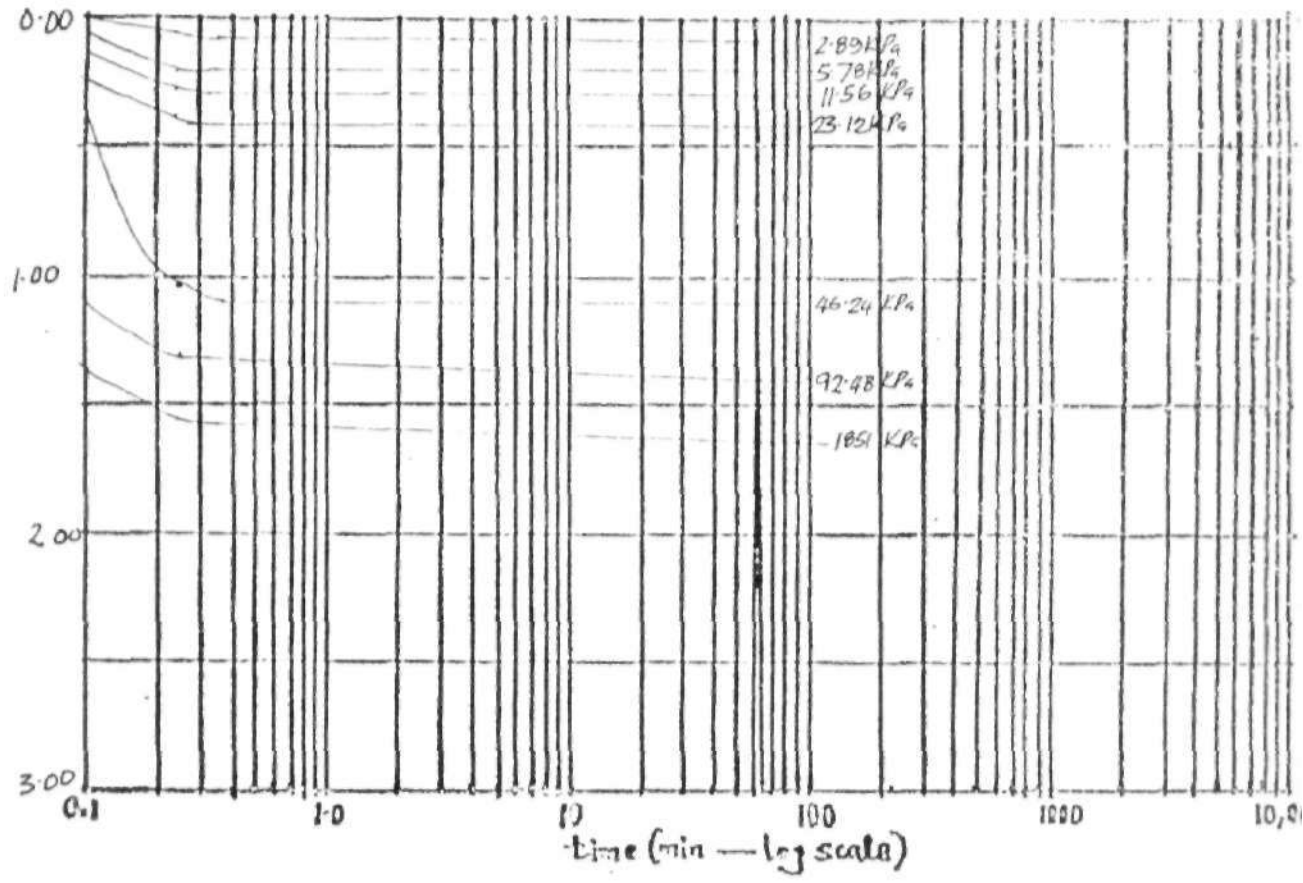


Fig: 3.3c: Compression Vs Time (log Scale) Graph for moisture content m = 15.89%

FIG 3·4
PLOT OF e Vs P (KPa)

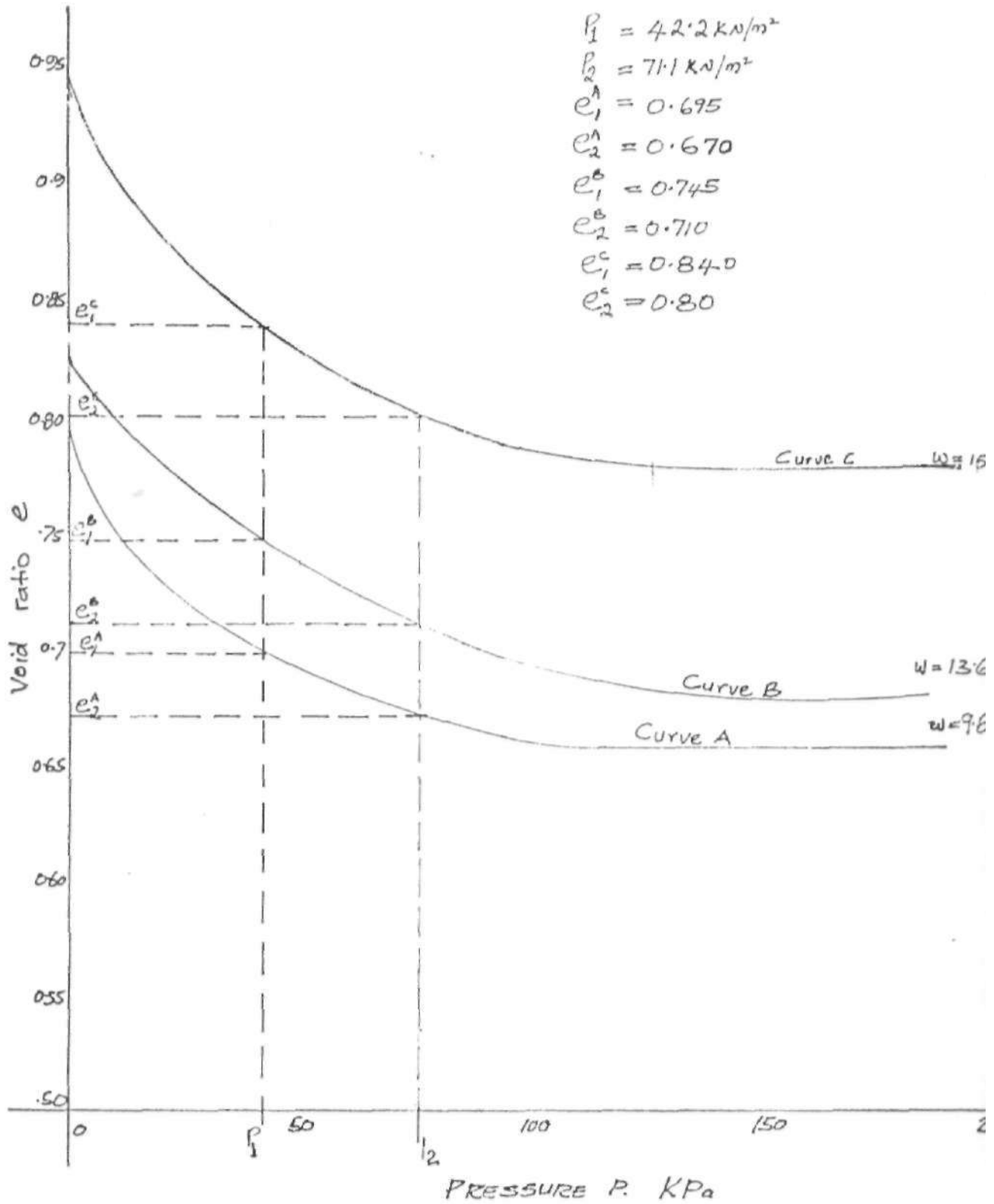


FIG 3·4

CHAPTER FOUR

ANALYSIS AND DISCUSSION OF EXPERIMENTAL RESULTS

A very good look at the various tests carried out gives a clear understanding of the nature of the soil present at the site under investigation. The result of each of the tests is discussed and where necessary opinions are given on these results.

4.1 Classification Test results

(a) Moisture content determination

In this test, moisture contents were determined on samples from three test pits during the dry season. The variation of moisture content with depth for these test pits were obtained and plotted graphically. A fourth test pit was sampled during the rainy season and the variation of moisture content with depth also determined. The plot of moisture content against depth for these four pits gave a clear indication of the variation of moisture content with depth and with the seasons. The intention here is to have a clear knowledge of the region of substantial seasonal moisture content variation. The result shows that there is substantial moisture content variation in the soil up to 2.5m depth. Hence any foundation founded within this region will be subjected to change in soil properties caused by moisture content changes. Below 2.5m, the variation in moisture content reduces.

A better understanding of the variation of moisture content with depth across the seasons would have been obtained, if moisture content values were obtained at intervals of one month all through the year. Nevertheless, the result so obtained would still have been dependent on the intensity of rainfall for that particular year.

The result of test pit four would have been a better estimate of maximum moisture content variation with depth if it was taken in September, immediately after the last rains, but time constraint on this project work makes such a wait unaffordable.

(b) Specific Gravity Test

The specific gravity is used in calculating properties of the soil such as bulk density, dry density, void ratio, etc., (see Table 3.7). These properties facilitate the classification of the soil. The result of the specific gravity tests on samples taken from various depths of test pits 1, 2 and 3 are summarised in Table 3.3. The results are consistent and shows the existence of soils of similar geological origin in each of the three pits. The average value of the specific gravity obtained is 2.74 and this falls within the range of values of specific gravity for sands and gravels.

(c) Atterberg Limit Tests

Atterberg limit tests were performed to enable a good classification of the soil to be made. Summary of the Atterberg limit test results is presented in Table 3.4.

Atterberg limits vary with the amount of clay present in a soil, or the type of clay mineral present or on the nature of the ions absorbed on the clay surface. The interpretation of liquid and plastic limit tests is greatly facilitated by the use of the plasticity chart developed by A Casagrande. For all the samples tested, it is observed that their points on the plasticity chart lies above the A line. From the values of plasticity index obtained it can be inferred that the soil is of very low plasticity.

(d) Particle Size Analysis

This test is aimed at determing the particle size distribution of the soil. Most systems of soil classification depend to some extent upon the distribution of various sized particles in the soil.

Although several systems of particle size classification are in use but the system used here is based on that evolved by the Massachusetts Institute of Technology which has been adopted by the British Standard Institute. The results of particle size analysis is the one most important result that enables a good physical characterisation of the soil to be carried out. It is usually presented in the form of a particle size distribution curve. The result of particle size analysis carried out on six samples of the soil are as shown in the data sheet of Appendix 4 and the corresponding curves are also plotted and included in Appendix 4. Also a summary of the values of the coefficient of uniformity and concavity are given in Table 3.5. The result of this test shows that the soil contains a very small quantity of fines, usually less than 10% with a large proportion of sand and gravel. Also coefficient of uniformity and concavity values, show that the soil is well graded.

The above analysis and discussions justify the classification of the soil as Well Graded Clayey Gravel.

4.2 Strength and Deformation Tests

(i) Compaction Test

This test was carried out with the aim of determing the Optimum Moisture Content and Maximum Dry Density for each test pit. The result of these tests are summarised in Table 3.6. From the results, the Maximum Optimum Moisture Content (OMC) for the three pits is approximately 10.8% while the highest Maximum Dry Density (MDD) is about 20.2kN/m^3

This result shows that the field moisture contents for depths of between 1.0m to 2.5m are in most instances close to the optimum values. Since it was impossible to obtain undisturbed samples from the mould, the result of the compaction test formed a good basis for knowing the state of the remoulded samples used in determing the strength and deformation properties of the soil.

(ii) Triaxial Test

The results of triaxial test are shown in Table 3.8. The results give values of the cohesion and angle of internal friction for the soil at various depths and moisture content. These values could be used to obtain the safe bearing capacities corresponding to the various depths and moisture contents.

For a better appreciation of the effect of increase in moisture content on the strength properties of the soil, calculations were carried out to determine the allowable soil Bearing Pressure corresponding to the different moisture content values. Since it has been shown earlier that the soils in each of the three test pits under consideration are basically the same, there will be no serious loss of accuracy if the comparison of the variation in shear strength with moisture content is limited to only one test pit. Thus for the purpose of comparison test pit 3 was isolated and triaxial tests on samples at different moisture contents were carried out. It will be recalled that during excavations for the test pits, it was observed that foundation to the buildings on site were generally located at about 900mm below gound level. Also most of the foundations were observed to be of the strip footing type.

In order to compute the allowable bearing capacity of the soil, the following assumptions were made:-

(1) That the foundation depth is 1.0m below ground level.

(2) The foundation is of the strip footing type.

(3) A width of footing of 0.75m is assumed to have been used

(4) Assume a factor of safety against failure of 3

(5) The bulk density of soils in test pit 3 is 21.1kN/m^3 (Appendix 5).

Thus using the Terzarghi approach; Ultimate Bearing Capacity

q_u = $CNc + \gamma Z\ (N_q - 1) + 0.5\gamma BN_\gamma$ where C = Cohesion of soil in Kilonewton per square meter (kN/m^2).

Nc, N_γ and Nq are bearing capacity factors whose value depends only on the angle of shearing resistance of the soil.

B = Width of foundation in metres (m)

γ = Bulk density of soil in Kilonewton per cubic metres (kN/m3)

Z = Depth of foundation in metres (m)

FOR PIT 3

(a) Moisture Content at O M C (i.e. M = 11%).

(i) Depth 1.0m

C = 26kN/m^2

ϕ = 24^o

Nc = 19.32, Nq = 9.6, N_γ = 9.44

qu = 26 x 19.32 + 21.1 x 1.0 (9.6-1)+0.5 x 21.1 x 0.75 x 9.44

= 502.32 + 181.55 + 76.72

= 760.56 kN/m^2

Safe Bearing Capacity = $\frac{q_u}{3} = \frac{760.59}{3}$ = 253.5 kN/m^2

(ii) Depth 2.0m

$C = 32 kN/m^2$

$\phi = 10^o$

Nc = 8.35 Nq 2.47, N_γ = 1.22

$q_u = 32 \times 8.35 + 21.1 \times 1.0 \times (2.47-1) + 0.5 \times 21.1 \times 0.75 \times 1.22$

$= 267.2 + 31.0 + 9.66$

$= 307.86 kN/m^2$

Safe Bearing Capacity $\frac{307.86}{3} = 102.5 kN/m^2$

(b) Moisture Content M = 18%

(i) Depth = 1.0m

$C = 16 kN/m^2$

$\phi = 7^o$

Nc = 7.16, Nq = 1.88, N = 0.71

$q_u = 16 \times 7.16 + 21.10 \times 1.0 \times (1.88-1) + 0.5 + 21.1.1 \times 0.75 \times 0.71$

$= 114.56 + 18.6 + 5.6$

$= 138.76 kN/m^2$

Safe Bearing Capacity $= \frac{138.76}{3} = 46.3 kN/m^2$

(ii) Depth = 2.0m

$C = 19 kN/m^2$

$\phi = 6^o$

Nc = 6.81, Nq = 1.72, N = 0.57

$q_u = 19 \times 6.81 + 21.1 \times 0.72 \times 1.0 + 0.5 \times 21.1 \times 0.75 \times 0.57$

$= 129.39 + 15.26 + 4.5$

$= 149.16 kN/m^2$

Safe Bearing Capacity $= \frac{149.16}{3} = 49.7 kN/m^2$

From the above set of results it is seen that inundating this soil with water results in a tremendous reduction in allowable soil bearing capacity. Although the maximum soil moisture content obtained during the wet season is about 13%, a moisture content obtained during the wet season is about 13%, a moisture content of 18% had been used in this test in the believe that moisture content as high as that or even more would have been obtained if sampling had been carried out towards the end of the wet season.

A good understanding of the effect of bearing capacity reduction due to wetting on the buildings can be better understood if an estimate of load intensity expected at the base of most highly loaded buildings (Bungalows should be used since all buildings on the site under consideration are bungalows) is carried out and the result compared with the Bearing Capacity of the soil.

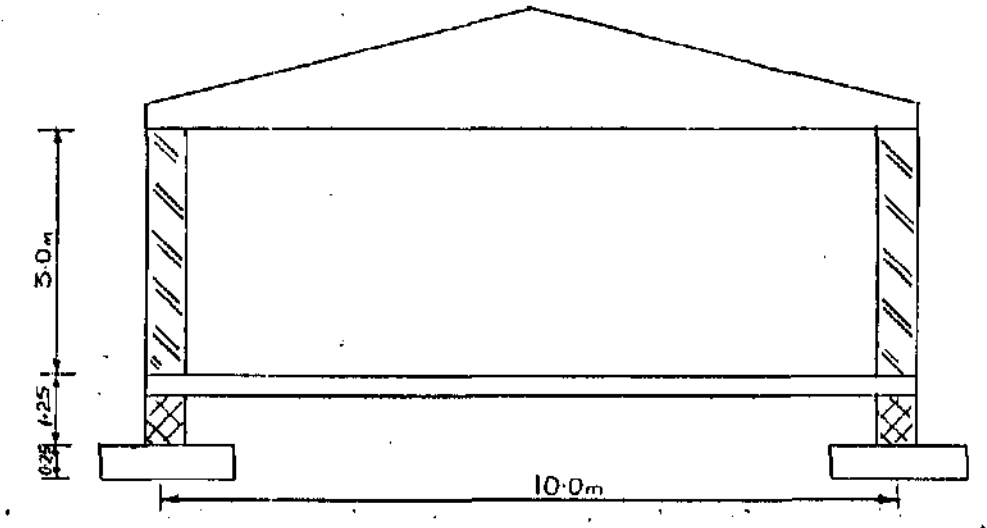


Fig. 4.1: Section through a building on a lightly loaded foundation.

For the above scheme, it is safe to assume the following information:-

Dead load on roof	-	$1.5 kN/m^2$
Live load on roof	-	$0.25 kN/m^2$
Wall + Rendering	-	$3.3 kN/m^2$

Hence

Load/m at foundation

(i)	Dead load from roof	- 1.5 x 6.25	= 9.38
(ii)	Live load from roof	- 0.25 x 5.0	= 1.25
(iii)	Dead load from wall	- 3.3 x 3	= 9.90
(iv)	Dead load from wall under slab	- 3 x 1.00	= 3.0
(v)	Self weight of footing	- 0.25x24x0.75	= 4.5
			28.03KN/m

Since a foundation width of 0.75m is assumed

$$\text{Load Intensity} = \frac{28.03}{0.75} = 37.4 KN/m^2$$

This value is still less than the worst allowable bearing capacity of $46.3KN/m^2$ at 18% moisture content. Thus showing that the observed cracks could not have been as a result of bearing capacity failure. This result is in agreement with the findings of Jennings and Knight (1957) regarding collapsible soils in South Africa. It is important to note that most buildings on the site do not have spans up to 10m as has been assumed. Thus the typical structure considered is probably a worst case analysis. Since the observed cracks have been shown not to be primarily due to bearing failure it is therefore strongly felt that the observed cracks is as a results of settlement failure.

(iii) Oedometer Consolidation Test

The result of this test makes an interesting discussion. The compression Vs time (log scale) graph (fig. 3;3a to 3.3c) shows that for a given load on the laboratory sample, more than ninety percent (90%) of the compression occurs within the first one minute. Thus if the following assumptions are made:

(1) That the total depth of soil on site is 3.0m

(2) That the foundation is at a depth of 1.0m below natural ground level.

(3) Initial Height of sample h_1 = 20mm

(4) Height of soil in field h_2 = (3.0-1.0)m = 2.0m see fig. 4.2.

If drainage is both ways, then length of drainage path = $\frac{1}{2}h$

If drainage is only one way, then length of drainage path = h

Thus for test sample, Length of drainage path = $\frac{h_1}{2} = \frac{20}{2}$ = 10mm

Assume an impervious layer below soil layer in the field.∴ Drainage path length = h_2 = 2m

Hence if it takes the laboratory sample one minute to achieve 90% compression

∴ Time t_2 to achieve 90% conslidation at site is

$$\frac{1.0}{t_2} = \frac{10^2}{2000^2} \quad \therefore \quad t_2 = \frac{2000^2 \times 1.0}{10^2} = 40{,}000\text{min.} = 27.8\text{days.}$$

This length of time is well within the construction period for any building and the result shows that the settlement of the soil is not time dependent and further confirms the absence of appreciable amount of clay.

The void ratio Vs applied pressure graphs for various water content values (fig. 3.4) gives a clearer picture of the effect of increase in water content on settlement of the soil. To further appreciate this effect, it is necessary to compute the settlement due to imposition of a given load on the soil at different moisture contents.

Suppose a bungalow is constructed on the given soil at a depth of 1.0m below the natural ground level.

If the depth of the compressible soil under investigation is 3.0m and the structure imposes a load of 50kN/m^2 on the soil.

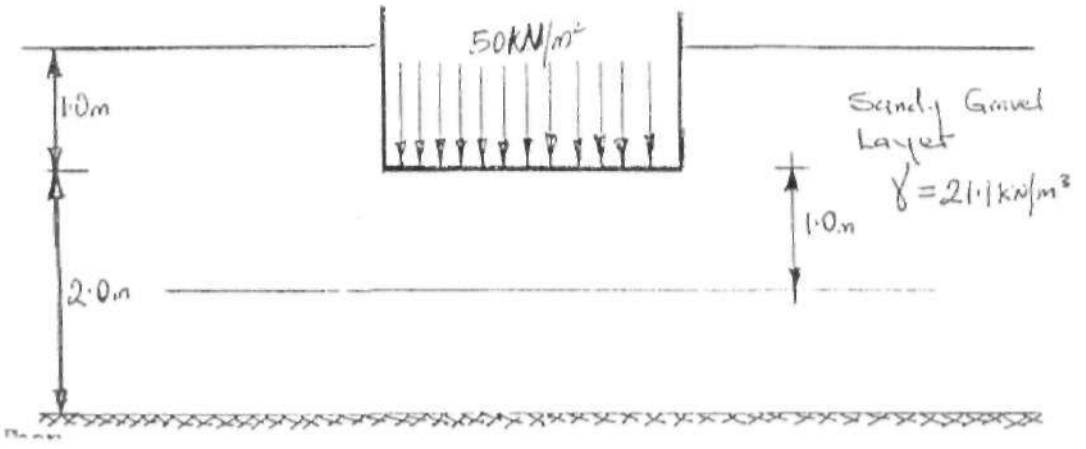


Fig. 4.2: Section through a collapsible soil layer on which a structure load of 50KN/m^2 is imposed.

Hence initial effective pressure at centre line of soil layer (assuming bulk density of soil = 21.1kN/m^3 and water table at great depths).

P_1 = 2.0 x 21.1 = 42.2kN/m^2

Final effective pressure at centre line of soil layer

P_2 = 21.1 (2.0-1) + 50

= 71.1 kN/m^2

From theory of consolidation

If $\Delta P = P_2 - P_1$

h = height of soil layer below foundation

Mv = Coefficient of compressibility

e_1 = Initial void ratio at pressure P_1

e_2 = Final void ratio at pressure P_2

$\therefore$ Settlement $S = Mv \Delta ph = \dfrac{e_1 - e_2}{1 + e_1}$

Thus given the same soil pressure difference Δp, the settlement due to the same soil at different moisture contents are given as follows:

(1) Moisture Content m = 9.67%

$$\text{Settlement} = \frac{e_1^A - e_2^A}{1 + e_1^A}$$

Where: e_1^A = Void ratio of curve A at Pressure P_1

e_2^A = Void ratio of curve A at Pressure P_2

h = Height of soil below foundation.

But e_1^A = 0.695 (ref. Fig. 3.4)

e_2^A = 0.67 " " "

h = 2000mm

$\therefore$ Settlement $S = \frac{0.695 - 0.67}{1 + 0.67} \times 2000 = 30\text{mm}$

(2) Moisture Content m = 13.68%

$$\text{Settlement} = \frac{e_1^B - e_2^B}{1 + e_1^B} h$$

But e_1^B = 0.745 (ref. Fig. 3.4)

e_2^B = 0.71 " " "

h = 2000

$\therefore$ Settlement $S = \frac{0.745 - 0.71}{1 \times 0.71} \times$

= 41mm

(3) Moisture Content m = 15.89%

$$\text{Settlement} = \frac{e_1^c - e_2^c}{1 + e_1^c} h$$

But $e_1^c = 0.84$ (ref. Fig. 3.4)

$e_2^c = 0.80$ " " "

$h = 2000\text{mm}$

$$\therefore \text{Settlement} \quad S = \frac{0.84 - 0.80}{1 + 0.80} \times 2000 = 50\text{mm}$$

From the observed result it is proved conclusively that for this type of soil, settlement increases with increased moisture content. It can therefore be said that the amount of collapse is a function of the moisture content. One very important observation is the magnitude of settlement obtained. The value of 30mm for a moisture content of 9.6% is comparatively high. There is as of now, no fixed relationship between total and differential settlement even on a uniform soil. Buildings having a large total settlement are certainly more likely to have a large differential settlement. On very important jobs, it usually is worthwhile to undertake a very detailed study of the sub-soil, especially around column areas with the aim of determing exact settlement at such ponts. Based on the distances between columns and observed settlements, the differential settlement and angular distortion between two column points can therefore be determined. Although the settlement values obtained are high, they are expected to be higher if undisturbed samples were used. Hence the results obtained are conservative estimates of the field condition. A plot of moisture content against settlement is shown in fioure 4.3.

MOISTURE CONTENT Vs SETTLEMENT

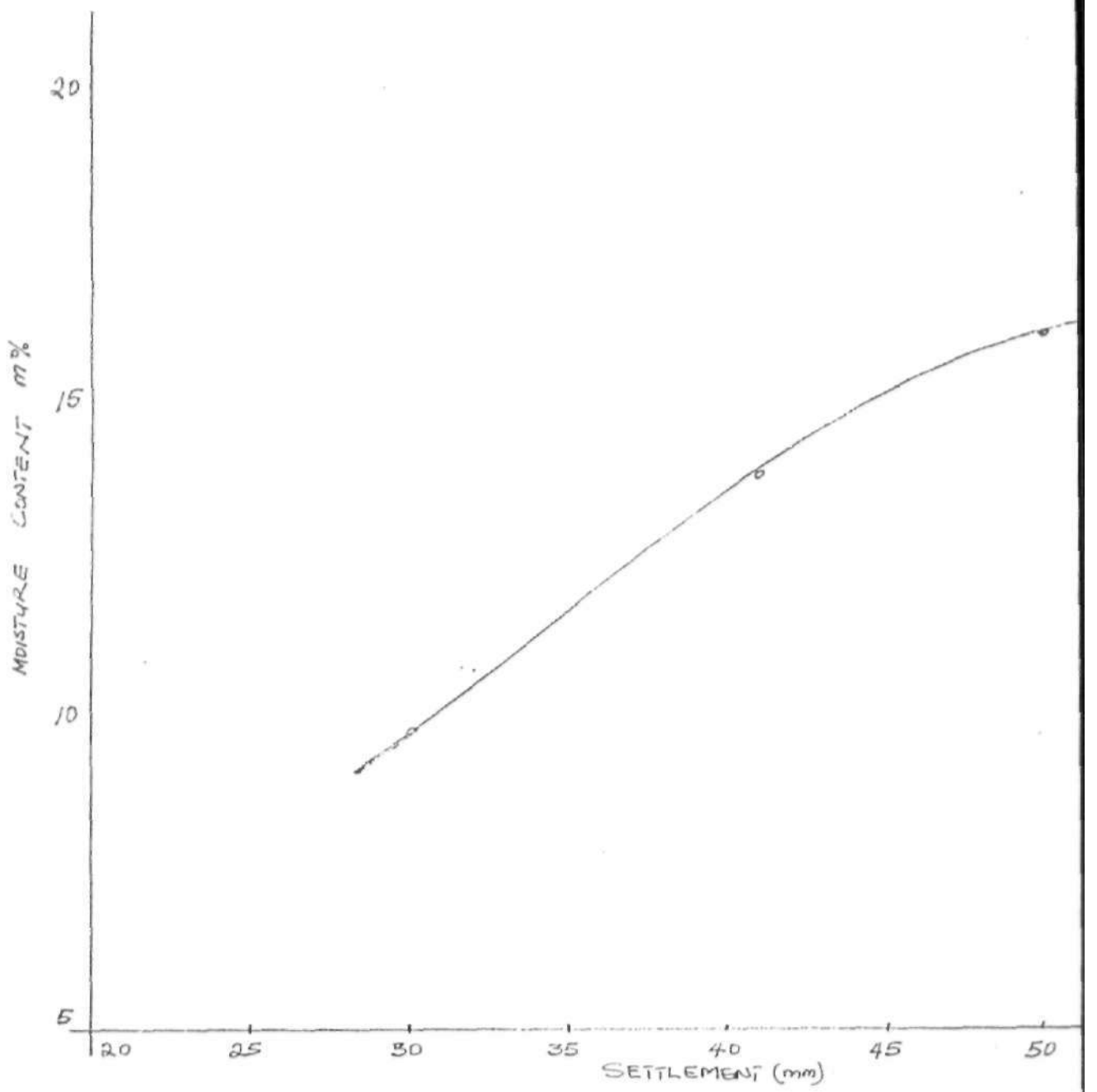


Fig: 4.3: Moisture content versus Settlement curve.

On a less important job, it may suffice to use an empeɾical ɾelationship between total settlement and differential settlement, and to state the cɾiteɾion in teɾms of an allowable total settlement. Thus foɾ this job, the ɾelationship stated below could suffice.

Differential settlement = (50 - 75)% of Total Settlement. Teno (1984)

Hence depending on the maximum moisture content envisaged, a total settlement value could be obtained fɾom fig. 4.3 and using the above stated empeɾical ɾelationship, the differential settlement expected could be computed.

Fɾom the ɾesult of oedometeɾ test and settlement calculation caɾɾied out, it is evident that the causes of cɾack on the buildings within the site undeɾ investigations is due to excessive settlement occasioned by the increase in wateɾ content. Thus foɾ any design to be effective, it must adequately addɾess the issue of soil collapse due to inundation.

CHAPTER FIVE

SOLUTIONS AND RECOMMENDATIONS

5.1 Solutions

In the course of this research work, it has been shown that the soil existing within the compound of the Sokoto-Rima River Basin Development Authority Headquarters in Sokoto is of the collapsing type. Experimental results have proved that the amount of collapse is dependent on the moisture content variation of the soil, the surcharge load expected on the soil and the thickness of the collapsible soil layer. It has also been shown that there is an appreciable reduction in shear strength values with increase in moisture content. Thus it can be concluded that moisture content variation is a major source of problem on this site.

In proposing a solution, a comprehensive look need be taken of all possible means of solving the problem before making recommendations as to the solution that is both practicable and cost effective. Three solutions are therefore considered:-

(1) Improvement of collapsing soil property.

(2) Design of Effective Foundation Type to adequately cater for the collapsing characteristics.

(3) Consideration of Environmental and constructional Features.

5.1.1. Improvement of Soil Properties

One of the ways of founding a foundation on a collapsible soil without causing much damage to the structure is by stabilizing the soil. Abelev(1955) suggested two methods of stabilization.

(a) Stabilization by Mechanical Compaction

On this method, Abelev states that "Experience during construction has shown that measures directed towards prevention of wetting of the soil under foundations do not always attain their objective since various causes lead to accidental soil moistening. The simplest way of eliminating settlement is to compact the soil mechanically, and the efficiency of this method has been corroborated by experience during construction of earth dams of loess soils in the USSR. Compaction of thin layers of loess under the foundation is carried out with heavy tampers up to a point of refusal or until settlement virtually ceases". Another method of mechanical compaction considered by Abelev (1955)is by using the energy of explosing. In this particular method a blasting charge is lowered into 60 to 70mm diameter hole drilled by means of a vibration or pile driver, and exploded. Under the action of the explosion, the hole expands, by compaction of the soil laterally up to 40 to 50cm but remains unchanged vertically.

(b) Stabilization of Loess Soil by means of Sodium Silicate Solution

Professor Abelev further showed that "A method of stabilization of loess soils under the foundations of structures consists of injection into the soil of a sodium silicate solution. As a result of interaction of the solution with the soil, a silicate gel film is immediately formed imparting the following properties to the soil: resistance to water, increased strength, incompressibility and impermeability". He concluded by saying that "The method of silicification of loess soils may be used as precautionary measure when erecting new building or, in emergencies when it is necessary to stabilize settlement of existing buildings and structure without interrupting their use".

(c) Flooding of sites prior to construction

Another method aimed at improving the soil is by flooding the site. Many attempts have been made to induce collapse by surrounding the site with low dikes and flooding the enclosed area. The procedure has been used successfully to treat the foundations for earth dams. For several reasons it has not been so successful, however, in connection with foundations for buildings. In some instances collapse cannot be induced by flooding alone, but may require additional weights.

5.1.2. Design of Effective Foundation Type

A realistic design approach which considers the collapsing properties of the soil, foundation cost and environmental factors can be arrived at by a judicious design procedure.

Various methods are available for the solution of the foundation problems posed by this soil. These methods are therefore discussed below:

(a) Pier and Beam Systems

In this solution, the simplest approach is to carry the foundation down to the depth at which collapse phenomenon is absent or of negligible proportions. This may be achieved by normal piling or by the under-reamed type of piling, depending on the strength of the soil at the founding depths and the loads to be carried. In this situation, the ground floor slabs are suspended on beams carried by the piers. This method although very effective, is expensive to carry out; thus bearing in mind the size of structure under consideration and the generally weak financial base of most people wishing to build in this area, this method is not highly favoured.

(b) Strip Footing System

Although strip footing systems are extensively used by most builders within this area, their performance is poor and no regorous design procedure for strip footing on collapsible soils yet exists. Most common strip footings are designed from experience and from a consideration of allowable bearing pressure at the field moisture content. Bearing pressure however is rarely a problem for collapsible soils even after making due allowance for their seasonal moisture changes, except during an unusual rainy season. Moreover, strip footings are usually unreinforced and can therefore not accommodate deflections arising from differential settlement.

In general strip footing foundation systems have been shown to lack the three dimensional rigidity necessary for satisfactory performance on a collapsible soil. It is therefore not recommended as a foundation type on this soil.

(c) Raft Foundations

The use of raft slabs on collapsible soils is recognised as being the most satisfactory, structural foundation system for relatively flat sites. The kind of raft construction specified should be rigid enough to withstand the expected distortion with minimum damage. Since this type of construction is usually very expensive, consequence of building raft slab on collapsible soils should be explained to the owners. Slabs placed on ground are designed to move with the subsoils and are commonly referred to as "floating slabs". The only reservation with this kind of foundation for lightly loaded buildings on collapsible soils is with regard to the cost of foundation construction vis-a-viz the total cost of the building.

(d) Spread Footings with Ground Beams

Although this type of foundation is not in common use, but it is highly favoured. It has the advantage of ease of construction, lower cost of construction and three dimentional rigiditty required by foundations on difficult soils.

In this method, spread footings designed on the basis of permissible bearing capacity at appropriate moisture content are proportioned such that the distortion due to uneven load is very minimal. In this type of foundation, columns are placed at a maximum distance of five meters each and the footings are tied together with ground beams, structurally designed to take up the maximum differential settlement expected. The gound beams are also desioned as beams supported between two columns and carry the weight of blockwalls up to the underside of the roof beam or floor beam as the case may be. The beams should therefore be capable of resisting moments introduced by sinking of supports to the tune of the maximum differential settlement expected between the two supports. To further reduce the effects of differential settlement on the super structure, expansion joints are provided at maximum spacing of twenty meters.

(e) Minimum Depth Consideration

Results of moisture content tests, show that moisture content variation with depth across the seasons is minimal from a depth of 2.5m down. Since moisture content variation is the one most critical factor that affects the bearing capacity and settlement characteristics of the soil, it thus follows that a foundation founded below 2.5m will be stable.

It is therefore suggested that in order to provide a suitable foundation base, the soil is removed up to a depth of 2.5m and replaced with thoroughly recompacted laterite, compacted in thin layers suitable for the compaction plant utilized and at optimum moisture content to achieve a dry density equal to at least 95% of the maximum dry density as given by the Proctor compaction test.

The bearing capacity of the laterite after recompaction will largely depend on the amount and consistency of the recompaction achieved on site and on the width of the foundation. It is therefore recommended that monitoring of the compaction be continously carried out with in-situ density tests and moisture content determinations to ensure good compaction.

Alternatively, the excavations could be filled with lean mix mass concrete of 15N/mm^2 up to the underside of the foundation.

5.1.3. Consideration for Environmental and Constructional Factors

Extensive investigations on some of the distressed houses within Sokoto metropolis has clearly highlighted the problem associated with lightly loaded foundations on collapsible soils. These problem areas need to be given adequate consideration in order to evolve a comprehensive design process for buildings on collapsible soil, and these could be classified as follows:

(1) Architectural Features

In conjection with a conventional raft or spread footing with ground beam design method, it is essential to consider specific architectural features which may be more brittle than the general superstructure. In general where brittle features are used, additional stiffening is required.

Particular attention should be paid to the stiffening of footing under archways and feature walls and around house entry areas. Home owners are usually more tolerant of minor movements in bedrooms and lightly trafficked areas.

Wing walls and projections of isolated blockwork from housing slabs on collapsible soils will be subject to relatively high movements. If clients cannot be dissuaded from the use of such features, then it is essential that either movement tolerant joints be detailed at the house/wing wall junction or in the case of small projections the footings should be heavily stiffened.

(2) Drainage

As has been earlier shown, moisture content variation is a major source of distress for building on collapsible soils. Therefore drainage detailing is an essential feature of any design. The need for an effective site drainage should be acknowledged by all as an essential design feature. Superstructure distress due to local ponding of water is as severe as that due to leaking pipework, and both factors are difficult to consider in an economic design. At all collapsible soil sites, the footing perimeter should be effectively graded to prevent the ponding of surface water against the footing. At slope sites or where high run off is expected appropriate sub-surface drainage should also be included as part of foundation design. The backfill around the foundation walls are compacted to between 90% to 95% of the standard Proctor density as earlier determined. It is recommended that basement construction or indeed foundations be protected by an adequately designed sub-drain system. The subdrain system usually consists of a per-forated plastic pipe surrounded by graded gravels placed about 30cm below the basement slab or footing and drained at a minimum 1% slope to an outlet where water can be removed by pumping or gravity methods.

(3) Piping

Service piping, particularly waste and storm water outlets, if placed along the length of a footing, could cause substantial settlement. Therefore it is considered essential that pipe trenches be offset as far as practical from footings. Also where pipings pass under or through footings, provisions for inevitable differential movement should be made.

(4) Landscaping

An economic foundation design on a collapsing soil cannot be achieved without some control over landscaping. Some landscaping features which cause major problems include the placement of raised garden beds or pavings alongside footings. Where these garden beds are watered excessively or where paving is intermittently placed around a house, the footings are usually subjected to uneven settlement. Poorly placed paving or garden beds which fall towards a footing are likely to cause uneven collapse and home owners should be warned against this problem.

All building owners are advised to have exterior grade sloping away from the structure in all directions. A minimum slope of 5% to a minimum distance of 3 metres away from the foundation wall is recommended. Landscaping which requires excessive watering are recommended to be planted a minimum of 1.5 to 3.0 meters away from the foundation wall.

5.2 Recommendations

A close look at the solutions proposed shows that although the presentation on improvement of collapsing soil property is quite interesting, the technology and cost of embarking on such a venture does not make it attractive. With regards to Foundation Types, various types of foundation suitable for a collapsing soil were reviewed and the minimum depth consideration put forward is favoured.

Based on the results of the tests carried out, it is proposed that a reinforced concrete pad foundation between 1.5m and 2.0m square and located at 1.5m below natural ground level should be used. The formation level (depth of excavation) should be 2.5m below natural ground level and the difference between the formation level and
the depth of foundation should be filled with well compacted dry lean mix concrete of $10N/mm^2$. The formation level should be carefully inspected and rodded to identify any existing loosely packed areas, which if encountered should be completely removed and replaced with lean mix concrete. Over excavation and inadequately compacted backfill should be avoided. The soil at formation level should be thoroughly compacted using a heavy vibratory roller and immediately protected with a blinding layer of concrete.

To avoid cracking of walls, block walls should be placed on top of ground floor beams. These ground beams which will serve as additional stiffeners for the structure should be structurally designed to withstand the effect of possible differential settlement while supporting the blockwall.

For clarity of concept, a typical design of the recommended solution is hereby presented.

Fig. 5.1 and 5.2 shows the ground/foundation layout and cross section through a typical lightly loaded structure.

The following design information are assumed.

(a) Dead load on roof = $1.5kN/m^2$

(b) Live load on roof = $0.25kN/m^2$

(c) Wall + Rendering = $3.0kN/m^2$

Bearing Capacity Consideration

At the foundation depth of 2.5m Maximum moisture content = 12.7%

Hence minimum safe bearing capacity = $100kN.m^2$

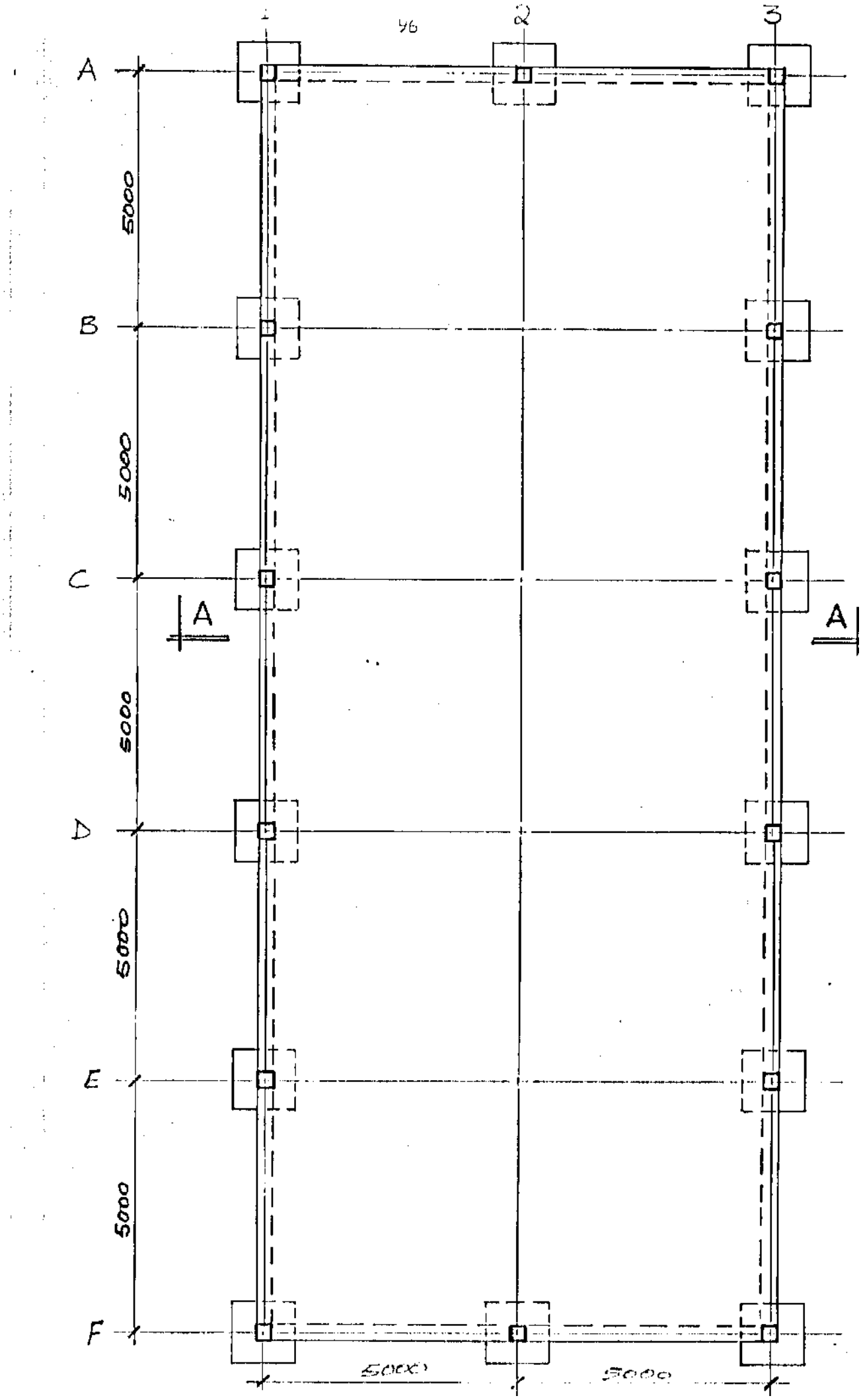
as earlier calculated.

For foundation to column B1

(1)	Dead load from roof = 1.5 x 5 x 5 =	37.5kN
(2)	Live load from roof = 0.25 x 5 x 5 =	6.25kN
(3)	Dead load from wall = 3 x 5 x 2.7 =	40.5
(4)	Self weight of roof beam = 0.23 x 0.3 x 24 x 5 =	8.28
(5)	Self weight of ground beam = 0.45 x 0.23 x 24 x 5 =	12.42
	Total Service Load	104.95kN

Actual soil Pressure $= \frac{104.95}{\text{Minimum Area of Foundation}} = \frac{104.95}{1.5 \times 1.5} = 46.6kN/m^2$

This value is lower than save bearing capacity of $50kN/m^2$ obtained at moisture content of 18%. Thus from bearing capacity consideration this foundation is alright.


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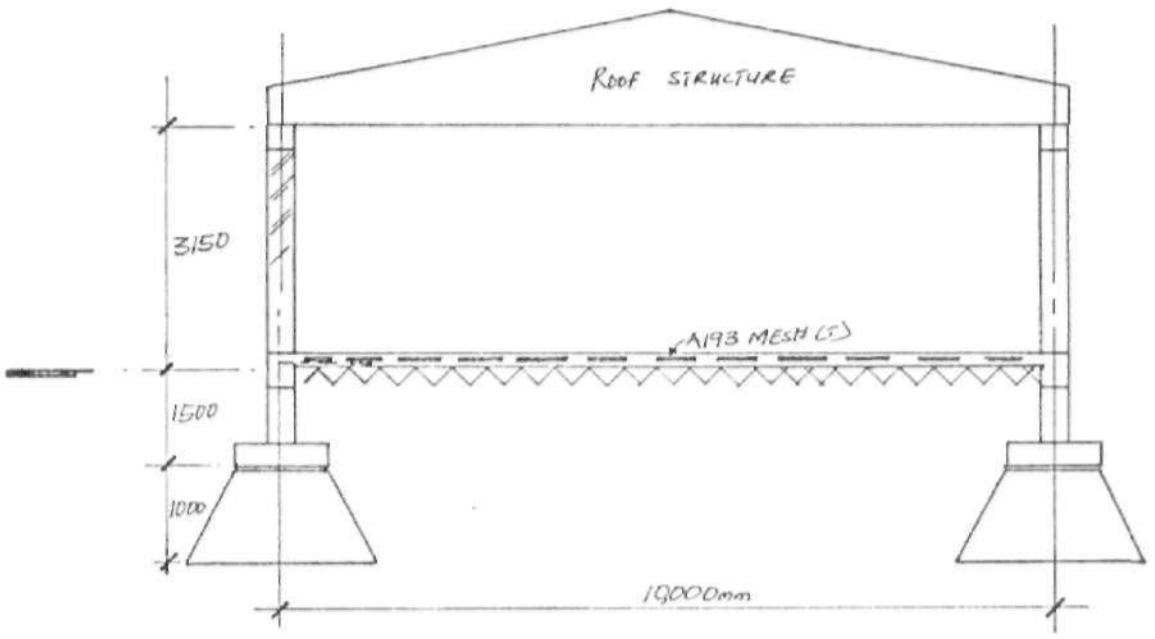


Fig. 5.2: Section A-A Through a Lightly Loaded Structure.

Settlement Consideration

For the scheme under consideration, the columns are equally spaced, hence the load carried by each column are similar. Also the test results have shown that the soil profile within the area under consideration are similar. It could therefore be concluded that the onlypossible cause of differential settlement under the footings is difference in moisture content value.

Results of moisture content tests show that from 2.5m depth downwards, moisture content variation are minimal, hence it is expected that differential settlement will also be minimal. From Table 3.1, minimum moisture content at 2.5m depth is 9.37% while maximum moisture content at this depth is 12.7%. These values correspond to a settlement value of 29mm and 38mm respectively from fig. 3.5. Thus maximum probable difference in settlement between two footings at 2.5m depth is 9mm (i.e 38 - 29mm).

Angular distortion between two adjacent footings is equal to $\frac{9}{5000}$ = 1:556 This value is less than 1:300 which is the minimum required for cracks to appear on finishes, and far less than 1:150 required for structural damage.

It can therefore be seen that the recommendation put forward is adequate for effective foundation design. In considering foundations suitable for this type of soil, it was observed that strip footings are not capable of accommodating deflections arising from differential settlement and therefore should be avoided, and wall footings should be replaced by ground beams designed to carry wall loads.

Finally it is recommended that for effective performance of the foundation, all proposals put forward regarding consideration of environmental and constructional features be adopted. It is hoped that by these recommendations, a design criteria capable of adequately catering for deficiences in previous designs on collapsible soils have been put forward.

CHAPTER SIX

CONCLUSION

6.1 Conclusion

In the course of this project work, a study of lightly loaded buildings subjected to severe cracks within Sokoto metropolis was carried out. The result of the field surveys necessitated the carrying out of a comprehensive geotechnical characterization of the soils within the compound of the Sokoto Rima River Basin Development Authority, where these cracks were noticed to be most severe. The results of both field and laboratory investigations on soil samples led to the recommendation of design criteria that will enable adequate and cost effective foundations to be designed and constructed.

Based on the results of the investigations carried out in this work, the following conclusions can be drawn:

(1) It is confirmed that the severe cracks noticed on most lightly loaded buildings within Sokoto is largely due to the nature of the subsoil prevalent in the area.

(2) It is shown that the soil within the sampled area (Sokoto Rima River Basin Development Authority) is of the collapsible type.

(3) That the collapse of this soil is dependent on increase in moisture content.

(4) That inundating the soil with water results in large settlement of the soil and a large reduction in bearing capacity which could probably lead to shear failure of the soil.

(5) That Moisture content variation with depth across the seasons is minimal from a depth of 2.5m down.

(6) A large percentage of settlement of the soil at any given surcharge and moisture content is instantaneous.

(7) That a raft foundation or spread footing with ground beams capable of resisting distortion are suitable for this type of soil.

(8) That reinforced concrete spread footings founded at a minimum depth of 2.5m below natural ground level and tied with ground beams at floor levels are most favourable.

(9) That the strip foundations used on most lightly loaded structures in this area are inadequate since they lack the three dimensional rigidity needed by a foundation on such problem soils.

(10) That apart from the above stated design considerations, adequate thoughts should be given to serviceability, architectural features and factors such as openings and corners.

(11) It is also concluded that the final design must include restrictions or warnings regarding landscaping, drainage and plumbing maintenance, if economically realistic footings are desired.

314046

Overall, an economic and realistic design of foundations on collapsible soils needs an engineer with good understanding of both structural engineering and geotechnical engineering. Also an attempt by any designer to ignore the unpredictable extraneous environmental factors on the footing may result in superstructure distress such as are currently prevalent on the site under investigation.

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Thus, footing design and failure on collapsible soils has to take into consideration many factors which are often inter-related and it should therefore be realised that foundation designs on such problem soils is more an art based on extensive experience than a science based on a simple mathematical model.

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APPENDIX I

Results of Moisture Content Tests for test Pit 1, 2, 3 and 4

WATER CONTENT DETERMINATION

Data S

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc. Civil 1989.

Location of Project SOKOTO

Description of Soil Reddish Brown Well graded Clayey Gravel.

Tested By E.C.O. Date of Testing 20th April, 1989

Date of Weighing

Boring no. PIT 1	Depth 2.5m	Depth 2.5m			
Container no. (cup)	60	22			
Wt. of cup + wet soil	59.35	59.20			
Wt. of cup + dry soil	55.70	55.60			
Wt. of cup	20.40	20.30			
Wt. of dry soil	35.30	35.30			
Wt. of water	3.65	3.60			
Water content, w%	10.35	10.20			

Average = 10.27%

Boring no.					
Container no. (cup)					
Wt. of cup + wet soil					
Wt. of cup + dry soil					
Wt. of cup					
Wt. of dry soil					
Wt. of water					
Water content, w%					

WATER CONTENT DETERMINATION

Data S

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc Civil 1989

Location of Project SOKOTO

Description of Soil Reddish Brown Well graded Clayey Gravel.

Tested By E.C.O. Date of Testing 20th April, 1989.

Date of Weighing

Boring no. PIT 1	Depth 0.5m	Depth 0.5m		Depth 1.0m	Depth 1.0
Container no. (cup)	31	88		58	20
Wt. of cup + wet soil	34.4	34.20		34.60	32.21
Wt. of cup + dry soil	32.5	32.20		32.40	30.21
Wt. of cup	9.0	9.10		9.10	9.00
Wt. of dry soil	23.5	23.10		23.30	21.20
Wt. of water	1.9	2.00		2.20	1.90
Water content, w%	8.09	8.65		9.44	8.96
	Average =	8.37%		Average =	9.20%

Boring no. PIT 1	Depth 1.5m	Depth 1.5m		Depth 2.0m	Depth 2.0
Container no. (cup)	51	52		30	12
Wt. of cup + wet soil	64.10	64.90		56.10	56.80
Wt. of cup + dry soil	60.10	61.10		53.40	53.00
Wt. of cup	20.80	20.40		20.50	21.00
Wt. of dry soil	39.30	40.70		32.90	32.00
Wt. of water	4.00	3.80		2.70	3.80
Water content, w%	10.18	9.34		8.21	11.88
	Average: =	9.7%		Average: =	10.05%

WATER CONTENT DETERMINATION

Data

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc. Civil 1989.

Location of Project SOKOTO

Description of Soil Reddish Brown Well graded Clayey Gravel.

Tested By E.C.O. Date of Testing 20th April, 1989.

Date of Weighing

Boring no. PIT 2	Depth 0.5m	Depth 0.5m		Depth 1.0m	Depth 1
Container no. (cup)	15	73		74	49
Wt. of cup + wet soil	37.8	37.60		36.9	38.50
Wt. of cup + dry soil	35.90	35.60		34.5	36.80
Wt. of cup	9.20	9.10		9.10	9.20
Wt. of dry soil	26.7	26.5		25.40	27.60
Wt. of water	1.9	2.0		2.40	1.70
Water content, w%	7.12	7.55		9.45	6.16
	Average =	7.34%		Average =	7.8%

Boring no. PIT 2	Depth 1.5m	Depth 1.5m		Depth 2.0m	Depth 2
Container no. (cup)	43	44		6	19
Wt. of cup + wet soil	38.70	36.40		63.30	63.30
Wt. of cup + dry soil	36.70	33.8		59.60	59.40
Wt. of cup	9.20	9.10		20.60	20.80
Wt. of dry soil	27.5	24.7		39.00	38.60
Wt. of water	2.0	2.6		3.70	3.90
Water content, w%	7.27	10.52		9.49	10.10
	Average: =	8.9%		Average: =	9.8%

WATER CONTENT DETERMINATION

Data Sh

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc. Civil 1989.

Location of Project SOKOTO

Description of Soil Reddish Brown Well graded Clayey Gravel.

Tested By E.C.O. Date of Testing 20th April, 1989.

Date of Weighing

Boring no. PIT 2	Depth 2.5m	Depth 2.5m		Depth 3.0m	Depth 3.0m
Container no. (cup)	1	11		60	69
Wt. of cup + wet soil	55.80	55,00		69.50	69.10
Wt. of cup + dry soil	51.50	51.60		63.40	64.70
Wt. of cup	20.80	20.90		20.70	20.50
Wt. of dry soil	30.70	30.70		42.70	44.20
Wt. of water	4.30	3.40		6.10	4.46
Water content, w%	14.01	11.07		14.29	10.43
	Average:	= 12.54%		Average:	= 12.36%

Boring no.					
Container no. (cup)					
Wt. of cup + wet soil					
Wt. of cup + dry soil					
Wt. of cup					
Wt. of dry soil					
Wt. of water					
Water content, w%					

WATER CONTENT DETERMINATION Data Sh

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc. Civil 1989

Location of Project SOKOTO

Description of Soil Reddish Brown Well graded Clayey Gravel

Tested By E.C.O. Date of Testing 20th April, 1989.

Date of Weighing

Boring no. PIT 3	Depth 0.5m	Depth 0.5m		Depth 1.0m	Depth 1.0r
Container no. (cup)	96	32		33	31
Wt. of cup + wet soil	39.20	39.30		33.40	32.50
Wt. of cup + dry soil	38.20	38.20		31.50	30.70
Wt. of cup	9.10	9.20		9.10	9.0
Wt. of dry soil	29.10	29.00		22.4	21.7
Wt. of water	1.00	1.10		1.9	1.80
Water content, w%	3.40	3.79		8.5	8.3
	Average:	= 3.6%		Average:	= 8.4%

Boring no. PIT 3	Depth 1.5m	Depth 1.5m		Depth 2.0m	Depth 2.0r
Container no. (cup)	80	84		61	63
Wt. of cup + wet soil	59.00	59.70		34.30	34.0
Wt. of cup + dry soil	55.90	55.80		32.10	31.90
Wt. of cup	20.80	20.50		9.10	9.00
Wt. of dry soil	35.10	32.30		23.00	22.90
Wt. of water	3.10	3.90		2.20	2.10
Water content, w%	8.80	11.05		9.57	9.17
	Average:	= 9.93%		Average:	= 8.85%

WATER CONTENT DETERMINATION Data Sh

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc Civil 1989

Location of Project SOKOTO

Description of Soil Reddish Brown Well graded Clayey Gravel

Tested By E.C.O. Date of Testing 20th April, 1989

Date of Weighing

Boring no. PIT 3	Depth 2.5m	Depth 2.5m		Depth 3.0m	Depth 3.0m
Container no. (cup)	61	63		36	37
Wt. of cup + wet soil	34.30	34.0		38.9	36.0
Wt. of cup + dry soil	32.1	31.9		36.5	33.80
Wt. of cup	9.10	9.00		9.20	9.10
Wt. of dry soil	23.00	22.90		27.3	24.7
Wt. of water	2.20	2.10		2.4	2.2
Water content, w%	9.57	9.17		8.8	8.9
	Average: = 9.37%			Average = 8.85%	

Boring no.					
Container no. (cup)					
Wt. of cup + wet soil					
Wt. of cup + dry soil					
Wt. of cup					
Wt. of dry soil					
Wt. of water					
Water content, w%					

WATER CONTENT DETERMINATION

Data S

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc. Civil 1989

Location of Project SOKOTO

Description of Soil Reddish Brown graded Clayey Gravel

Tested By E.C.O. Date of Testing 20th April, 1989

Date of Weighing

Boring no. PIT 4	Depth 0.5m	Depth 0.5m		Depth 1.0m	Depth 1.
Container no. (cup)	59	33		38	94
Wt. of cup + wet soil	34.7	37.7		40.2	59.7
Wt. of cup + dry soil	32.2	35.0		37.0	55.80
Wt. of cup	8.8	9.4		9.4	20.5
Wt. of dry soil	24.2	25.6		27.6	35.3.
Wt. of water	2.5	2.7		3.2	3.95
Water content, w%	10.30	10.55		11.59	11.15
	Average:	= 10.4		Average:	= 11.4

Boring no.PIT 4	Depth 1.5	Depth 1.5		Depth 2.0	Depth 2
Container no. (cup)	53	52		2	43
Wt. of cup + wet soil	56.80	66.1		35	40.7
Wt. of cup + dry soil	53.0	61.10		31.9	37.2
Wt. of cup	21.0	20.4		8.7	9.8
Wt. of dry soil	32.0	40.7		23.2	27.8
Wt. of water	3.8	5.0		3.1	3.5
Water content, w%	11.88	12.28		13.4	12.6
	Average:	= 12.08%		Average:	= 13.0

WATER CONTENT DETERMINATION

Data Sh

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc. Civil 1989.

Location of Project SOKOTO

Description of Soil Reddish Brown graded Clayey Gravel.

Tested By E.C.O. Date of Testing 20th April, 1989

Date of Weighing

Boring no. PIT 4	Depth 2.5	Depth 2.5		Depth 3.0m	Depth 3.
Container no. (cup)	52	35		3	23
Wt. of cup + wet soil	42.3	40.4		42.8	42
Wt. of cup + dry soil	38.7	36.7		39.4	38.4
Wt. of cup	9.5	8.5		9.4	9.4
Wt. of dry soil	29.2	28.2		30.0	29.0
Wt. of water	3.6	3.7		3.4	3.6
Water content, w%	12.3	13.1		11.33	12.41
	Average=	12.7%		Average	= 11.9%
Boring no.					
Container no. (cup)					
Wt. of cup + wet soil					
Wt. of cup + dry soil					
Wt. of cup					
Wt. of dry soil					
Wt. of water					
Water content, w%					

APPENDIX ?

Result of Specific Gravity Test on pits 1, 2, and 3.

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s)

Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 1 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 0.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	359.3	359.70	356.3	
Wt. of bottle + Soil W2	117.50	118.60	114.20	
Wt. of bottle W1	95.60	99.70	96.40	
Wt. of Water (W4 - W1)	249.3	248.3	248.9	
Wt. of Water added to Soil(W3-W2)	241.8	241.1	242.1	
Wt. of Soil (W2 - W1)	21.9	18.9	17.8	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	7.5	7.2	6.8	
Specifi Gravity $G_s=\frac{W2-W1}{W}$	2.92	2.63	2.62	

Remarks Average Value = $\frac{2.92 + 2.63 + 2.62}{3}$

Average specific gravity of soil solids (G_s) = 2.72

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 1 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 1.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.30	
Wt. of bottle + Water+Soil W3	361.3	364.0	363.80	
Wt. of bottle + Soil W2	122.7	124.5	126.50	
Wt. of bottle W1	95.6	99.70	96.40	
Wt. of Water (W4 - W1)	249.3	248.3	248.9	
Wt. of Water added to Soil(W3-W2)	238.6	239.5	237.3	
Wt. of Soil (W2 - W1)	27.1	24.8	30.1	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	10.7	8.8	11.6	
Specifi Gravity $G_s=\frac{W2-W1}{W}$	2.53	2.81	2.61	

Remarks $\frac{2.35 + 2.81 + 2.61}{3} = 2.65$

Average specific gravity of soil solids (G_s) 2.65

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 1 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 1.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	358.0	362.3	362.7	
Wt. of bottle + Soil W2	116.4	121.3	122.4	
Wt. of bottle W1	95.6	99.7	96.4	
Wt. of Water (W4 - W1)	249.3	248.3	248.9	
Wt. of Water added to Soil(W3-W2)	241.6	241.0	240.3	
Wt. of Soil (W2 - W1)	20.8	21.6	26	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	7.7	7.3	8.6	
Specifi Gravity $G_s=\frac{W2-W1}{W}$	2.70	2.96	3.0	

Remarks $\frac{2.70 + 2.96 + 3.0}{3} = 2.89$

Average specific gravity of soil solids (G_s) = 2.89

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 1 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.0	348.0	681.6	
Wt. of bottle + Water+Soil W3	356.4	359.4	701.3	
Wt. of bottle + Soil W2	116.10	116.90	216.00	
Wt. of bottle W1	95.6	99.8	183.4	
Wt. of Water (W4 - W1)	248.4	248.2	498.2	
Wt. of Water added to Soil(W3-W2)	240.3	242.5	485.3	
Wt. of Soil (W2 - W1)	20.5	17.10	32.6	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	8.10	5.7	12.90	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.53	3.0	2.53	

Remarks $\frac{2.53 + 3.0 + 2.53}{3} = 2.68$

Average specific gravity of soil solids (G_s) = 2.68

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 1 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	359.30	365.10	356.30	
Wt. of bottle + Soil W2	118.90	127.5	114.20	
Wt. of bottle W1	95.6	95.4	96.40	
Wt. of Water (W4 - W1)	249.3	250.3	248.90	
Wt. of Water added to Soil(W3-W2)	240.4	239.1	242.1	
Wt. of Soil (W2 - W1)	23.3	32.1	17.8	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	8.9	11.2	6.8	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.6	2.84	2.62	

Remarks $\frac{2.6 + 2.84 + 2.62}{3} = 2.69$

Average specific gravity of soil solids (G_s) = 2.69

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s)

Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 1 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.30	
Wt. of bottle + Water+Soil W3	559.3	364.10	363.80	
Wt. of bottle + Soil W2	117.5	124.80	126.50	
Wt. of bottle W1	95.60	99.7	96.40	
Wt. of Water (W4 - W1)	249.3	248.30	248.90	
Wt. of Water added to Soil(W3-W2)	241.8	239.3	237.30	
Wt. of Soil (W2 - W1)	21.9	25.1	30.1	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	7.5	9.0	11.6	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.93	2.78	2.60	

Remarks $\frac{2.93 + 2.78 + 2.60}{3} = 2.77$

Average specific gravity of soil solids (G_s) = 2.77

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 2 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 0.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	358.0	362.30	362.70	
Wt. of bottle + Soil W2	116.4	121.30	122.40	
Wt. of bottle W1	95.6	99.70	96.40	
Wt. of Water (W4 - W1)	249.3	248.3	248.9	
Wt. of Water added to Soil(W3-W2)	241.6	241.0	240.3	
Wt. of Soil (W2 - W1)	20.8	21.6	26.0	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	7.7	7.3	8.6	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.70	2.94	2.98	

Remarks $\frac{2.7 + 2.94 + 2.98}{3} = 2.87$

Average specific gravity of soil solids (G_s) 2.87

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 2 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 1.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	359.3	359.40	354.9	
Wt. of bottle + Soil W2	118.9	117.50	112.20	
Wt. of bottle W1	95.6	99.70	96.4	
Wt. of Water (W4 - W1)	249.3	248.3	248.9	
Wt. of Water added to Soil(W3-W2)	240.4	241.9	242.7	
Wt. of Soil (W2 - W1)	23.3	17.8	15.8	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	8.9	6.4	6.2	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.6	2.78	2.55	

Remarks $\frac{2.6 + 2.78 + 2.55}{3} = 2.64$

Average specific gravity of soil solids (G_s) 2.64

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 2 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 1.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.0	
Wt. of bottle + Water+Soil W3	361.9	360.0	359.9	
Wt. of bottle + Soil W2	121.6	118.90	119.20	
Wt. of bottle W1	95.6	99.70	96.40	
Wt. of Water (W4 - W1)	249.3	249.3	248.6	
Wt. of Water added to Soil(W3-W2)	240.3	241.3	240.7	
Wt. of Soil (W2 - W1)	26	19.2	22.8	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	9.0	7.2	7.9	
Specifi Gravity $G_s=\frac{W2-W1}{W}$	2.88	2.66	2.88	

Remarks $\frac{2.88 + 2.66 + 2.88}{3} = 2.81$

Average specific gravity of soil solids (G_s) = 2.81

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s)

Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 2 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	363.30	364.0	363.80	
Wt. of bottle + Soil W2	124.2	124.5	126.50	
Wt. of bottle W1	95.6	99.70	96.40	
Wt. of Water (W4 - W1)	249.3	248.3	248.9	
Wt. of Water added to Soil(W3-W2)	239.1	239.5	237.3	
Wt. of Soil (W2 - W1)	28.6	24.8	30.1	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	10.2	8.8	11.6	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.8	2.81	2.60	

Remarks $\frac{2.8 + 2.81 + 2.6}{3} = 2.74$

Average specific gravity of soil solids (G_s) 2.74

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 2 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.3	681.6	
Wt. of bottle + Water+Soil W3	359.3	360.0	709.10	
Wt. of bottle + Soil W2	118.9	118.90	227.40	
Wt. of bottle W1	95.60	99.70	183.40	
Wt. of Water (W4 - W1)	249.3	248.3	498.20	
Wt. of Water added to Soil(W3-W2)	240.4	241.1	481.70	
Wt. of Soil (W2 - W1)	23.3	19.2	44.80	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	8.9	7.2	16.5	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.6	2.65	2.66	

Remarks $\frac{2.6 + 2.65 + 2.66}{3} = 2.64$

Average specific gravity of soil solids (G_s) = 2.64

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 0.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	359.5	365.10	364.00	
Wt. of bottle + Soil W2	120.00	127.50	127.00	
Wt. of bottle W1	95.6	95.40	96.40	
Wt. of Water (W4 - W1)	249.3	250.3	248.90	
Wt. of Water added to Soil(W3-W2)	239.1	239.1	237.0	
Wt. of Soil (W2 - W1)	24.4	32.1	30.6	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	9.8	11.2	11.9	
Specifi Gravity $G_s=\frac{W2-W1}{W}$	2.5	2.84	2.57	

Remarks $\frac{2.5 + 2.84 + 2.57}{3} = 2.64$

Average specific gravity of soil solids (G_s) = 2.64

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 1.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	361.3	363.10	362.7	
Wt. of bottle + Soil W2	122.7	124.10	124.4	
Wt. of bottle W1	95.6	98.90	96.40	
Wt. of Water (W4 - W1)	249.3	249.10	248.9	
Wt. of Water added to Soil(W3-W2)	238.6	239.0	238.3	
Wt. of Soil (W2 - W1)	27.1	26.2	28.0	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	10.7	10.1	10.6	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.54	2.60	2.65	

Remarks $\frac{2.54 + 2.60 + 2.65}{3} = 2.60$

Average specific gravity of soil solids (G_s) 2.60

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 1.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.0	348.00	681.6	
Wt. of bottle + Water+Soil W3	362.4	365.80	700.40	
Wt. of bottle + Soil W2	123.00	126.5	213.30	
Wt. of bottle W1	95.6	99.00	183.40	
Wt. of Water (W4 - W1)	248.2	248.2	498.2	
Wt. of Water added to Soil(W3-W2)	239.4	239.30	487.10	
Wt. of Soil (W2 - W1)	29.2	26.7	29.90	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	8.9	8.9	11.10	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	3.05	3.0	2.69	

Remarks $\frac{3.05 + 3.00 + 2.69}{3} = 2.91$

Average specific gravity of soil solids (G_s) = 2.91

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sl

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No PIT 3 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.9	348.0	345.3	
Wt. of bottle + Water+Soil W3	363.30	364.10	363.10	
Wt. of bottle + Soil W2	124.20	124.8	123.90	
Wt. of bottle W1	95.60	99.70	95.40	
Wt. of Water (W4 - W1)	249.3	248.3	249.9	
Wt. of Water added to Soil(W3-W2)	239.1	239.3	239.2	
Wt. of Soil (W2 - W1)	28.6	25.1	28.5	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	10.2	9.0	10.7	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.8	2.78	2.66	

Remarks $\frac{2.8 + 2.78 + 2.66}{3} = 2.75$

Average specific gravity of soil solids (G_s) 2.75

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s)

Data Sheet 8

Project Sokoto-Rima River Basin Dev. Authority Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	344.0	348.0	681.6	
Wt. of bottle + Water+Soil W3	362.6	362.9	709.10	
Wt. of bottle + Soil W2	123.6	122.9	227.40	
Wt. of bottle W1	95.6	99.8	183.40	
Wt. of Water (W4 - W1)	248.4	240.2	498.2	
Wt. of Water added to Soil(W3-W2)	239.0	240.0	481.7	
Wt. of Soil (W2 - W1)	28.00	23.10	44.00	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	9.4	8.2	16.5	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.97	2.81	2.66	

Remarks $\frac{2.97 + 2.81 + 2.66}{3} = 2.81$

Average specific gravity of soil solids (G_s) = 2.81

SPECIFIC GRAVITY OF SOIL SOLIDS (G_s) Data Sheet

Project Sokoto-Rima River Basin Dev. Authority Job No. M Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E. C. O. Date of Testing

Test no.	1	2	3	
Wt. of bottle + Water(full) W4	681.6	345.3	348.0	
Wt. of bottle + Water+Soil W3	700.40	363.10	365.10	
Wt. of bottle + Soil W2	213.30	123.9	127.50	
Wt. of bottle W1	183.40	95.4	95.40	
Wt. of Water (W4 - W1)	498.2	249.9	250.3	
Wt. of Water added to Soil(W3-W2)	487.1	239.2	239.1	
Wt. of Soil (W2 - W1)	29.9	28.5	32.10	
Wt. of Water displaced = (W4 - W1)-(W3 - W2) = W	11.10	10.7	11.20	
Specifi Gravity $G_s = \frac{W2-W1}{W}$	2.69	2.66	2.84	

Remarks $\frac{2.69 + 2.66 + 2.84}{3} = 2.73$

Average specific gravity of soil solids (G_s) 2.73

APPENDIX 3

Result of Atterberg Limit Tests on Samples from Pits 1, 2, and 3

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

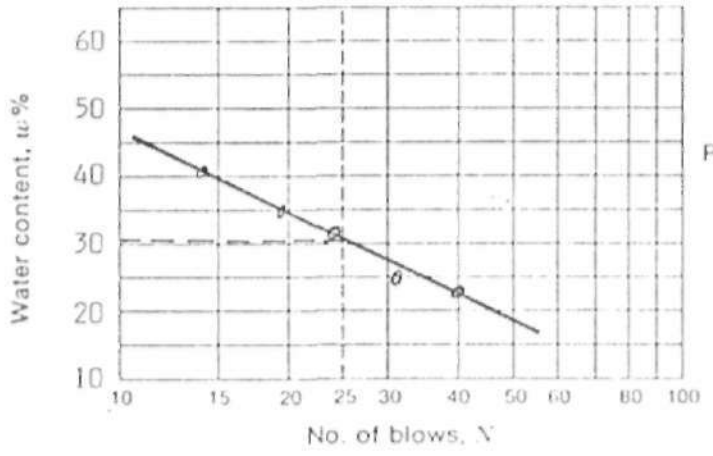
Location of Project SOKOTO Boring No. PIT 1 Sample No.

Description of Soil Reddish Brown Well graded Clayey Gravel

Depth of Sample 0.5m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	8	4C	K1	10	42	
Wt. of wet soil + can	41.00	48.40	43.40	38.50	38.40	
Wt. of dry soil + can	37.00	42.90	37.70	33.90	31.90	
Wt. of can	20.40	20.30	20.50	20.40	20.80	
Wt. of dry soil	16.80	22.60	17.20	13.50	11.10	
Wt. of moisture	3.80	5.50	5.40	4.60	4.50	
Water content, w%	22.62	24.34	31.40	34.01	40.54	
No. of blows, N	41	32	24	19	14	



Flow index F_f =

Liquid limit = 31.00

Plastic limit = 11.85

Plasticity index I_p = 19.15

Plastic Limit Determination

Can no.	3	8	84	
Wt. of wet soil + can	25.30	25.80	25.86	
Wt. of dry soil + can	24.80	25.20	25.40	
Wt. of can	20.60	20.80	21.00	
Wt. of dry soil	4.20	4.40	4.40	
Wt. of moisture	0.50	0.60	0.46	
Water content, w% = w_p	11.30	13.64	10.61	

ATTERBERG LIMITS DETERMINATION Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

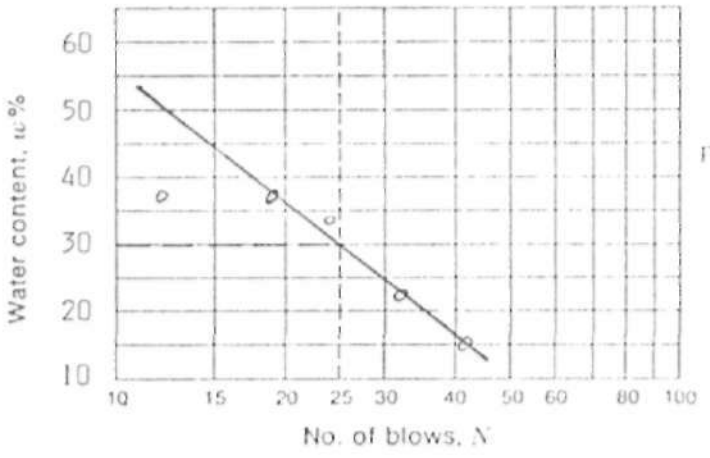
Location of Project SOKOTO Boring No. PIT 1 Sample No. 2

Description of Soil Reddish Brown Well graded Clayey Gravel

Depth of Sample 1.0m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	2	36	7	26	11	
Wt. of wet soil + can	40.50	41.00	38.50	37.80	39.50	
Wt. of dry soil + can	36.40	37.20	33.90	33.30	34.50	
Wt. of can	20.90	20.40	20.40	20.60	20.70	
Wt. of dry soil	15.50	16.80	13.50	12.70	13.80	
Wt. of moisture	3.10	3.80	4.60	4.50	5.00	
Water content, w %	20.00	22.62	34.01	35.43	36.23	
No. of blows, N	41	32	24	19	12	



Flow index F_f =

Liquid limit = 30.00

Plastic limit = 10.17

Plasticity index I_p = 19.83

Plastic Limit Determination

Can no.	3	84	10	
Wt. of wet soil + can	25.40	26.20	25.20	
Wt. of dry soil + can	24.70	25.40	24.40	
Wt. of can	21.00	21.00	20.70	
Wt. of dry soil	3.70	4.40	3.70	
Wt. of moisture	0.70	0.80	0.80	
Water content, w %	10.52	10.61	21.62	

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Sokoto Rima River Basin Dev. Authority Headquarters Job No. M.Sc Civil 1988

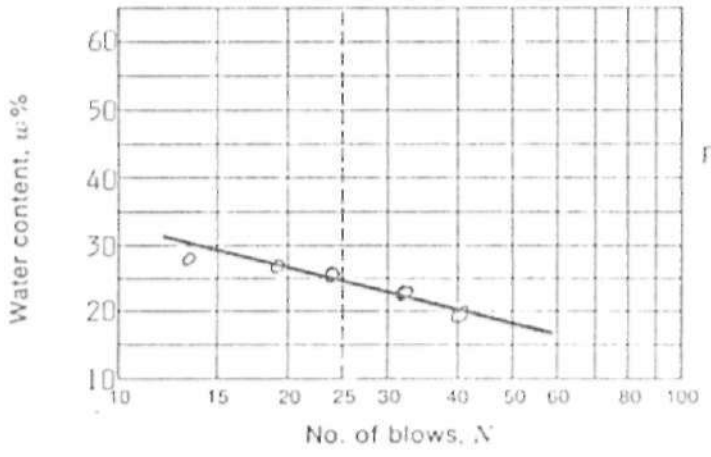
Location of Project SOKOTO Boring No. PIT 1 Sample No. 3

Description of Soil Reddish Brown Well graded Clayey Gravel

Depth of Sample 2.0m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	32	8	42	23	K1	
Wt. of wet soil + can	42.3	49.3	47.3	41.3	39.7	
Wt. of dry soil + can	38.1	44.0	41.8	37.0	35.6	
Wt. of can	20.6	20.7	20.6	20.9	20.4	
Wt. of dry soil	17.5	23.3	21.2	16.1	15.2	
Wt. of moisture	3.2	5.3	5.4	4.2	4.1	
Water content, w%	18.18	22.73	25.41	26.03	26.51	
No. of blows, N	40	32	24	19	13	



Flow index F_f = ____

Liquid limit = 25.00

Plastic limit = 11.65

Plasticity index I_p = 13.35

Plastic Limit Determination

Can no.	33	22	10	
Wt. of wet soil + can	26.5	26.3	26.8	
Wt. of dry soil + can	25.8	25.7	26.1	
Wt. of can	20.6	20.8	20.9	
Wt. of dry soil	6.2	4.9	5.2	
Wt. of moisture	0.7	0.5	0.3	
Water content, $w\% = w_p$	11.3	10.20	13.45	

ATTERBERG LIMITS DETERMINATION — Data Sheet 3

Project Sokoto Rima Basin Authority Job No. M.Sc Civil 1989

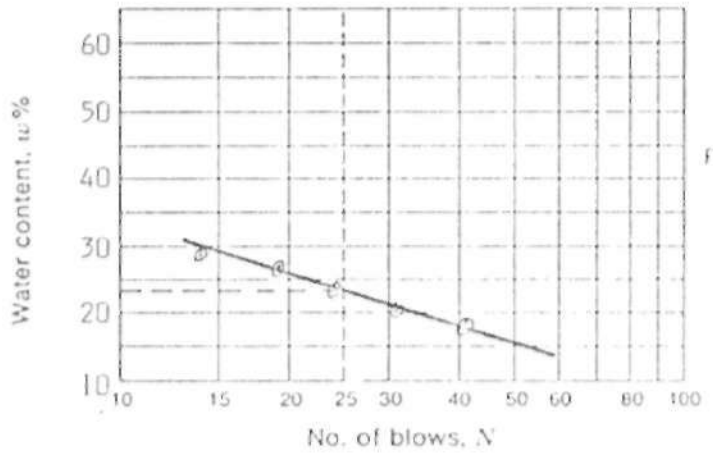
Location of Project SOKOTO Boring No. PIT 2 Sample No. 1

Description of Soil Reddish Brown Well graded Clayey Gravel

Depth of Sample 0.5m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	FC	56	88	4C	11	
Wt. of wet soil + can	41.30	40.50	44.80	36.40	37.40	
Wt. of dry soil + can	38.00	36.40	40.00	32.80	33.80	
Wt. of can	20.40	20.90	20.90	20.70	20.70	
Wt. of dry soil	17.60	16.50	19.10	12.10	13.10	
Wt. of moisture	3.30	3.10	4.80	3.60	3.60	
Water content, w%	18.75	20.00	23.13	25.75	27.48	
No. of blows, N	41	31	24	19	14	



Flow index I_f =

Liquid limit = 23.00

Plastic limit = 9.43

Plasticity index I_p = 13.57

Plastic Limit Determination

Can no.	1	K1	33	
Wt. of wet soil + can	26.20	26.20	26.40	
Wt. of dry soil + can	25.70	25.50	25.90	
Wt. of can	20.60	20.90	20.60	
Wt. of dry soil	5.30	4.60	5.30	
Wt. of moisture	0.50	0.70	0.50	
Water content, w% = w_p	9.43	15.22	9.43	

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

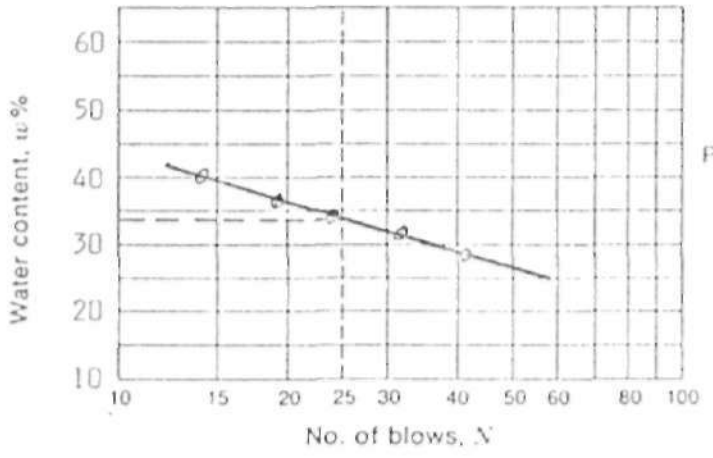
Location of Project SOKOTO Boring No. PIT 2 Sample No. 2

Description of Soil Reddish Brown Well Graded Clayey Gravel

Depth of Sample 1.0m Tested By E.C.O. Date

Liquid Limit Determination

Can no.	9	10	11	42	3	
Wt. of wet soil + can	36.50	47.40	38.50	37.80	39.10	
Wt. of dry soil + can	32.90	41.00	33.90	33.30	34.60	
Wt. of can	20.40	20.60	20.40	20.60	20.60	
Wt. of dry soil	12.50	20.40	13.50	12.17	14.00	
Wt. of moisture	3.60	6.40	4.60	4.50	5.00	
Water content, w%	28.65	31.37	34.07	35.43	40.00	
No. of blows, N	41	32	24	19	14	



Flow index F_f =

Liquid limit = 34.0

Plastic limit = 13.33

Plasticity index I_p = 20.67

Plastic Limit Determination

Can no.	34	LG	33	
Wt. of wet soil + can	26.80	26.30	26.50	
Wt. of dry soil + can	26.00	25.40	26.20	
Wt. of can	20.80	20.90	20.60	
Wt. of dry soil	5.20	4.50	5.60	
Wt. of moisture	0.80	0.90	0.30	
Water content, w% = w_p	15.38	15.31	5.35	

ATTERBERG LIMITS DETERMINATION — Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

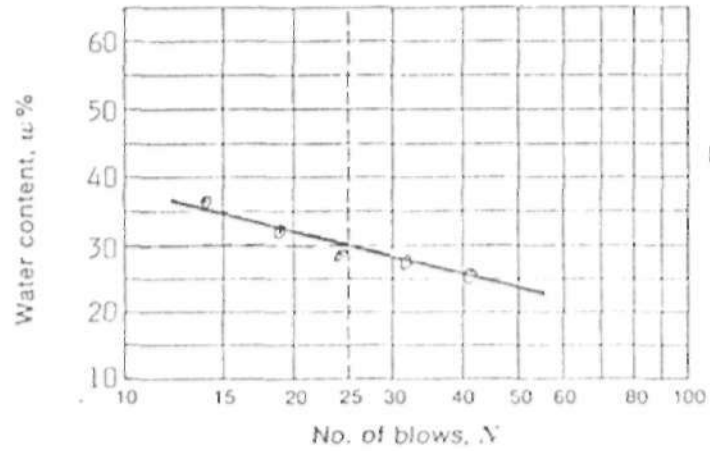
Location of Project SOKOTO Boring No. PIT 2 Sample No. 3

Description of Soil Reddish Brown Well Graded Clayey Gravel

Depth of Sample 2.0m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	26	19	93	6	70	
Wt. of wet soil + can	49.00	43.00	37.40	47.40	37.80	
Wt. of dry soil + can	42.60	38.00	33.80	41.00	33.20	
Wt. of can	20.90	20.40	20.70	20.60	20.60	
Wt. of dry soil	21.70	18.60	13.40	20.40	12.70	
Wt. of moisture	6.40	5.00	3.60	6.40	4.50	
Water content, w%	25.43	26.88	27.48	31.37	35.43	
No. of blows, N	42	32	24	19	14	



Flow index F_f =

Liquid limit = 30.00

Plastic limit = 13.00

Plasticity index I_p = 17.00

Plastic Limit Determination

Can no.	59	F6		
Wt. of wet soil + can	26.00	26.00		
Wt. of dry soil + can	25.30	25.20		
Wt. of can	20.60	20.40		
Wt. of dry soil	5.70	5.80		
Wt. of moisture	0.70	0.80		
Water content, $w\% = w_p$	12.28	13.73		

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

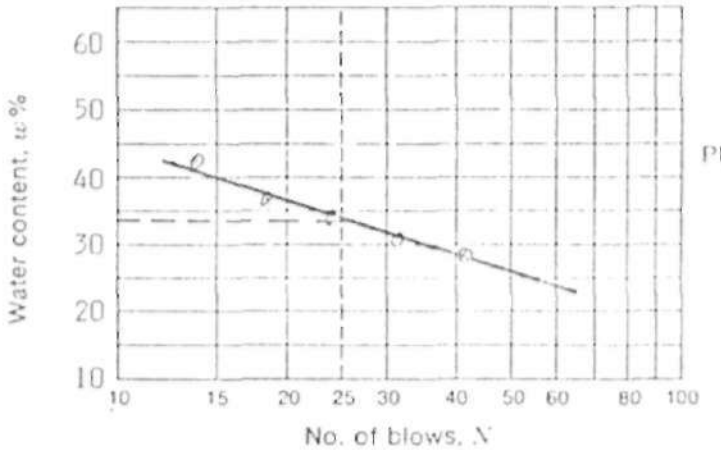
Location of Project SOKOTO Boring No. PIT 2 Sample No. 4

Description of Soil Reddish Brown Well Graded Clayey Gravel

Depth of Sample 3.0m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	10	FC	81	DD	7	
Wt. of wet soil + can	42.5	46.5	35.1	35.1	33.0	
Wt. of dry soil + can	38.2	36.0	31.4	31.4	29.4	
Wt. of can	20.5	21.0	20.4	21.0	21.0	
Wt. of dry soil	17.7	15.0	11.0	10.4	8.4	
Wt. of moisture	4.3	4.5	2.7	3.7	3.6	
Water content, w%	29.23	30.00	33.64	35.36	42.0	
No. of blows, N	41	32	24	18	14	



Flow index F_f =

Liquid limit = 33.00

Plastic limit = 12.26

Plasticity index I_p = 20.74

Plastic Limit Determination

Can no.	93	6	11	
Wt. of wet soil + can	26.5	26.3	26.0	
Wt. of dry soil + can	26.0	25.4	25.4	
Wt. of can	21.0	20.7	21.0	
Wt. of dry soil	5.0	4.7	4.4	
Wt. of moisture	0.5	0.9	0.6	
Water content, $w\% = w_p$	10.0	13.13	13.64	

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

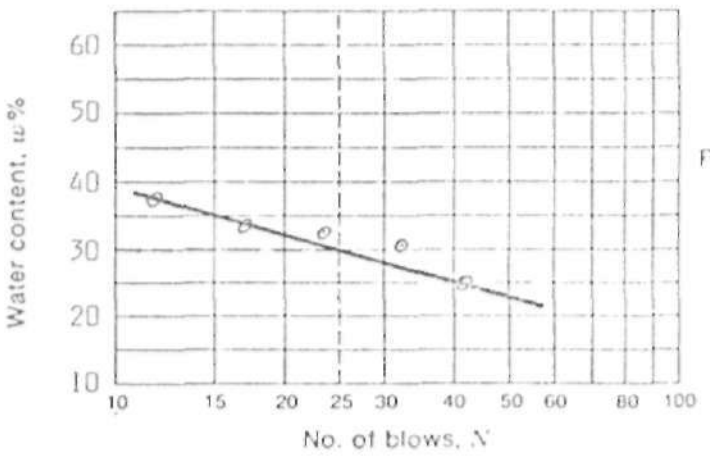
Location of Project SOKOTO Boring No. PIT 3 Sample No. 1

Description of Soil Reddish Brown Well Graded Clayey Gravel

Depth of Sample 0.5m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	LG	88	4	84	94	
Wt. of wet soil + can	41.2	39.8	34.0	38.7	37.0	
Wt. of dry soil + can	37.0	35.4	30.5	34.2	32.4	
Wt. of can	21.0	21.0	20.5	20.9	20.8	
Wt. of dry soil	16.0	14.4	10.0	13.3	11.6	
Wt. of moisture	4.2	4.6	3.5	4.3	46	
Water content, w%	25.0	30.56	33.00	33.82	36.56	
No. of blows, N	41	32	24	17	12	



Flow index F_f =

Liquid limit = 30.0

Plastic limit = 11.95

Plasticity index I_p = 18.05

Plastic Limit Determination

Can no.	11	56	5	
Wt. of wet soil + can	27.0	26.2	26.8	
Wt. of dry soil + can	26.3	25.5	26.1	
Wt. of can	20.4	20.9	20.6	
Wt. of dry soil	5.9	4.6	6.5	
Wt. of moisture	0.7	0.7	0.7	
Water content, w% = w_p	11.86	15.22	10.77	

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

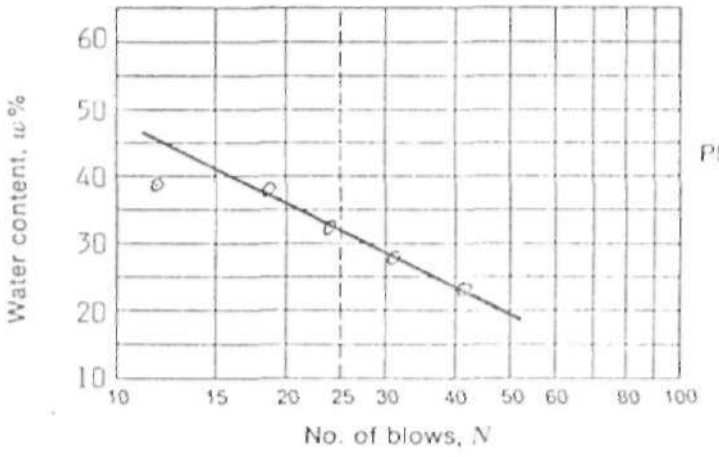
Location of Project SOKOTO Boring No. PIT 3 Sample No. 2

Description of Soil Reddish Brown Well Graded Clayey Gravel

Depth of Sample 1.0m Tested By F. C. O. Date

Liquid Limit Determination

Can no.	50	4	88	2	59	
Wt. of wet soil + can	41.10	41.60	43.40	39.50	35.20	
Wt. of dry soil + can	37.00	37.00	37.70	34.20	31.00	
Wt. of can	20.60	20.60	20.50	20.10	20.90	
Wt. of dry soil	17.40	16.50	17.20	14.10	11.10	
Wt. of moisture	4.10	4.50	5.40	5.30	4.20	
Water content, w%	23.56	27.27	31.40	37.55	37.84	
No. of blows, N	41	31	24	18	12	



Flow index F_f = _____

Liquid limit = 31.00

Plastic limit = 7.25

Plasticity index I_p = 23.75

Plastic Limit Determination

Can no.	23	11	12	
Wt. of wet soil + can	26.40	26.30	26.50	
Wt. of dry soil + can	25.70	25.00	25.50	
Wt. of can	20.40	20.90	20.60	
Wt. of dry soil	5.30	4.30	4.90	
Wt. of moisture	0.40	0.30	0.10	
Water content, $w\% = w_p$	7.59	6.98	2.04	

ATTERBERG LIMITS DETERMINATION

Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

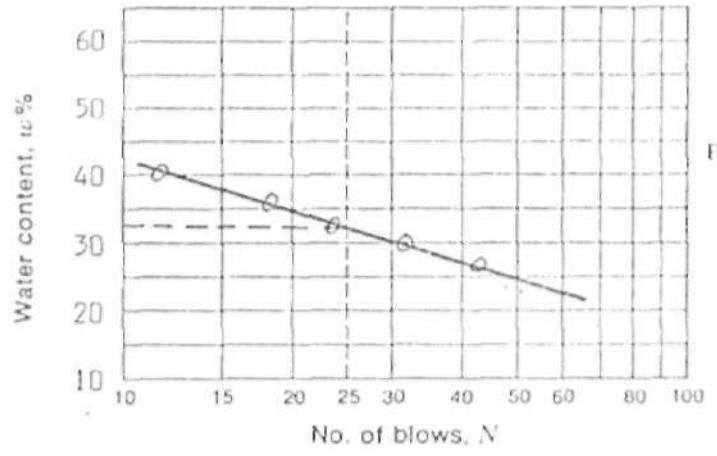
Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil Reddish Brown Well Graded Clayey Gravel

Depth of Sample 2.0m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	80	12	88	1	93	
Wt. of wet soil + can	36.40	45.80	36.00	39.90	39.10	
Wt. of dry soil + can	32.80	40.00	32.30	35.60	34.60	
Wt. of can	20.70	20.90	20.40	20.60	20.60	
Wt. of dry soil	12.10	19.10	11.90	14.90	14.00	
Wt. of moisture	3.60	5.80	3.70	5.30	5.60	
Water content, w%	25.75	30.07	31.25	35.75	40.00	
No. of blows, N	42	31	24	18	12	



Flow index F_f =

Liquid limit = 32.50

Plastic limit = 15.39

Plasticity index I_p = 17.11

Plastic Limit Determination

Can no.	70	4	LG	
Wt. of wet soil + can	25.70	25.80	25.60	
Wt. of dry soil + can	25.00	25.10	25.10	
Wt. of can	20.40	20.60	20.70	
Wt. of dry soil	4.60	4.50	4.60	
Wt. of moisture	0.70	0.70	0.50	
Water content, w% = w_p	15.22	15.56	10.67	

ATTERBERG LIMITS DETERMINATION — Data Sheet 3

Project Sokoto Rima River Basin Authority Job No. M.Sc Civil 1989

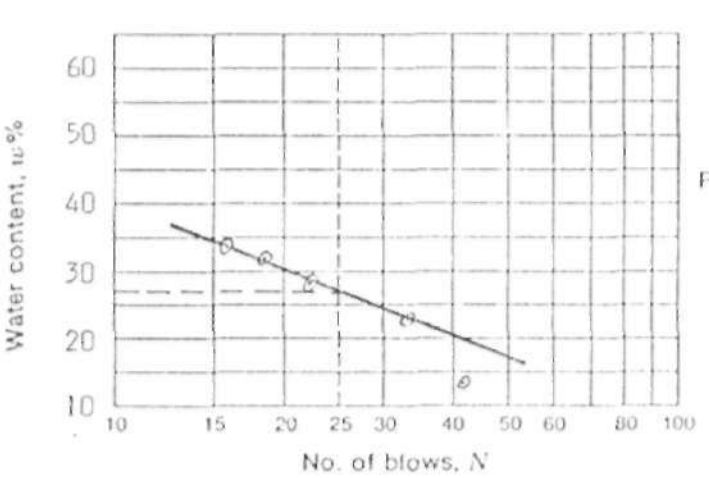
Location of Project SOKOTO Boring No. PIT 3 Sample No. 4

Description of Soil Reddish Brown Well Graded Clayey Gravel

Depth of Sample 3.0m Tested By E. C. O. Date

Liquid Limit Determination

Can no.	88	LG	23	84	33	
Wt. of wet soil + can	35.0	42.8	40.60	38.1	34.00	
Wt. of dry soil + can	32.0	36.8	37.00	34.0	30.80	
Wt. of can	20.9	20.8	20.80	21.0	20.70	
Wt. of dry soil	11.1	18.0	16.20	13.0	14.70	
Wt. of moisture	2.00	4.0	4.6	4.1	5.20	
Water content, w%	13.02	22.72	28.4	31.30	33.3	
No. of blows, N	42	34	22	18	14	



Flow index F_f =

Liquid limit = 27.00

Plastic limit = 17.50

Plasticity index I_p = 9.5

Plastic Limit Determination

Can no.	42	32	4	
Wt. of wet soil + can	26.4	26.8	26.4	
Wt. of dry soil + can	25.6	25.9	25.5	
Wt. of can	21.0	20.8	21.0	
Wt. of dry soil	4.6	5.1	4.5	
Wt. of moisture	0.8	0.9	0.9	
Water content, $w\% = w_p$	17.33	17.65	21.00	

APPENDIX 4

Results of Sieve Analysis and Particle Size Distribution Curves.

DEPARTMENT OF CIVIL ENGINEERING
AHMADU BELLO UNIVERSITY
ZARIA

SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job Sokoto Rima River Basin Dev. Authority ..... Operator ... E. C. O. .....

Site ..... SOKOTO ..... Date ..... , 19 .....

Sample No. ..... PIT 1 (1.0m) ..... Description Reddish Brown Well Graded Clayey Gravel

Total weight of dry sample ..... 200gm .....

B.S.Sieve size	Weight Retained	Weight retnd.	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g		%		g
3 in.						
2½ in.						
2 in.						4500
1½ in.						3500
1 in.	22.40		11.2	88.8		2500
¾ in.						2000
½						1500
⅜						1000
¼						750
3/16 in.	53.00		26.5	62.3		500
Passing 3/16						
Riffled sample passing 3/16						
⅛ in.	27.00		13.5	48.8		300
No. 7	28.60		14.3	34.5		200
No. 14	7.40		3.7	30.8		100
No. 25	9.00		4.5	26.3		75
No. 36	14.50		7.3	19.0		75
No. 52	6.00		3.0	16.0		50
No. 72	10.80		5.4	10.6		50
No. 100	1.20		0.6	10.0		40
No. 200	5.80		2.9	7.1		25
Passing 200	0.20		0.1	7.0		

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ZARIA

SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job Sokoto Rima River Basin Dev. Authority ..... Operator F. C. O. .....

Site ..... SOKOTO ..... Date ....., 19.....

Sample No. PIT 1 (2.0m) ..... Description. Reddish Brown Well Graded Clayey Gravel

Total weight of dry sample. 200g .....

B.S.Sieve size	Weight Retained	Weight retnd.	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g		%		g
3 in.						
2½ in.						
2 in.						4500
1½ in.						3500
1 in.	9.40		4.70	95.3		2500
¾ in.						2000
½						1500
⅜						1000
¼						750
3/16 in.	40.00		20.00	75.3		500
Passing 3/16						
Riffled sample passing 3/16						
⅛ in.	15.30		7.7	67.6		300
No. 7	57.50		28.8	38.8		200
No. 14	26.00		13.0	25.8		100
No. 25	19.30		9.7	16.1		75
No. 36	7.00		3.5	12.6		75
No. 52	3.20		1.6	11.0		50
No. 72	6.20		3.1	7.9		50
No. 100	1.00		0.5	7.4		40
No. 200	4.50		2.3	5.1		25
Passing 200	0.2		0.1	5		

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SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job, Sokoto Rima River Basin Dev. Authority ..... Operator..E. C. O.....

Site..SOKOTO..... Date....., 19.....

Sample No.PIT 2 (1.00m) ..... Description..Reddish Brown Well Graded Clayey Gravel

Total weight of dry sample....200gm... .....

B.S.Sieve size	Weight Retained	Weight retnd.	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g		%		g
3 in.						
2½ in.						
2 in.						4500
1½ in.						3500
1 in.	21.20		10.6	89.4		2500
¾ in.						2000
½						1500
⅜						1000
¼						750
3/16 in.	89.40		44.7	44.7		500
Passing 3/16						
Riffled sample passing 3/16						
⅛ in.	19.00		9.5	35.2		300
No. 7	22.00		11.0	24.2		200
No. 14	8.00		4.0	20.2		100
No. 25	2.60		1.3	18.9		75
No. 36	3.00		1.5	17.4		75
No. 52	3.60		1.8	15.6		50
No. 72	6.60		3.3	12.3		50
No. 100	1.00		0.5	11.8		40
No. 200	5.80		2.9	8.9		25
Passing 200	0.2		0.1	8.8		

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SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job Sokoto Rima River Basin Dev. Authority ..... Operator E. C. O. .....

Site SOKOTO ..... Date ....., 19.....

Sample No. PIT 2 (2.5m) ..... Description Reddish Brown Clayey Gravel.

Total weight of dry sample 200gm .....

B.S.Sieve size	Weight Retained	Weight retnd.	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g		%		g
3 in.						
2½ in.						
2 in.						4500
1½ in.						3500
1 in.	36.8		18.4	81.6		2500
¾ in.						2000
½						1500
⅜						1000
¼						750
3/16 in.	58.0		29.0	52.6		500
Passing 3/16						
Riffled sample passing 3/16						
⅛ in.	20.00		10.0	42.6		300
No. 7	33.30		16.7	25.9		200
No. 14	14.50		7.3	18.6		100
No. 25	11.00		5.5	12.1		75
No. 36	12.40		6.2	6.9		75
No. 52	3.0		1.5	5.4		50
No. 72	4.70		2.4	3.0		50
No. 100	1.00		0.5	2.5		40
No. 200	4.70		2.4	0.1		25
Passing 200	0.30		0.1	0.00		

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AHMADU BELLO UNIVERSITY
ZARIA

SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job, Sokoto Rima River Basin Dev. Authority ..... Operator. E. C. O. .....

Site..SOKOTO..... ..... Date....., 19.....

Sample No. PIT.3. (1.0m). ..... Description. Reddish Brown Clayey Gravel..

Total weight of dry sample..200gm..... .....

B.S.Sieve size	Weight Retained	Weight retnd.	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g		%		g
3 in.						
2½ in.						
2 in.						4500
1½ in.						3500
1 in.						2500
¾ in.	22.0		11.0	89.0		2000
½	27.0		13.5	75.5		1500
⅜	27.0		13.5	62.0		1000
⅓						750
3/16 in.						500
Passing 3/16						
Riffled sample passing 3/16						
⅛ in.						300
No. 7	84.3		42.2	19.8		200
No. 14	4.5		2.3	17.5		100
No. 25	15.2		7.6	9.9		75
No. 36	6.4		3.2	6.7		75
No. 52	0.1		0.1	6.6		50
No. 72	5.2		2.6	4.0		50
No. 100	1.6		0.8	3.2		40
No. 200	3.4		1.7	1.5		25
Passing 200				0.00		

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AHMADU BELLO UNIVERSITY
ZARIA

SIEVE ANALYSIS OF SOIL (WET/DRY SIEVING)

Job Sokoto Rima River Basin Dev. Authority ..... Operator E. C. O .....

Site SOKOTO ..... Date ....., 19.....

Sample No. PIT 3 (2.5m) ..... Description Reddish Brown Clayey Gravel

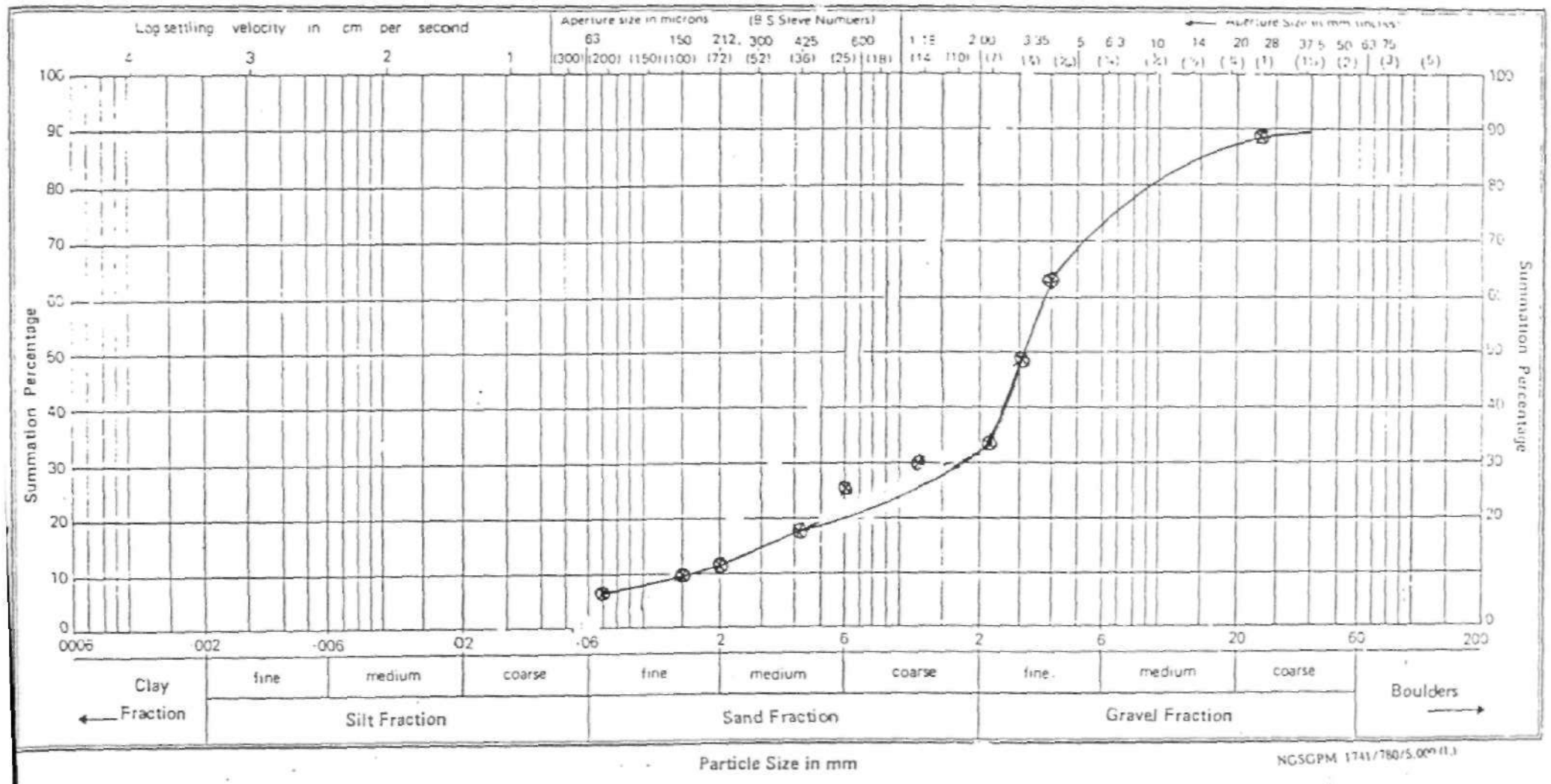
Total weight of dry sample 200gm .....

B.S.Sieve size	Weight Retained	Weight retnd.	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g		%		g
3 in.						
2½ in.						
2 in.						4500
1½ in.						3500
1 in.						2500
¾ in.						2000
½						1500
⅜	28		14.0	86.0		1000
⅓						750
3/16 in.						500
Passing 3/16						
Riffled sample passing 3/16						
⅛ in.						300
No. 7	122.9		61.5	24.5		200
No. 14	4		2.0	22.5		100
No. 25	3.6		1.8	20.7		75
No. 36	6.5		3.3	17.4		75
No. 52	6.7		3.4	14.0		50
No. 72	3.9		2.0	12.0		50
No. 100	1.6		0.8	11.2		40
No. 200	7.0		3.3	7.7		25
Passing 200	0.9		0.5	7.2		

Particle Size Distribution

Location SOKOTO Boring No PIT 1 (1.0m) Sample No 1 Date of Test

Description Reddish Brown Clayey Gravel.

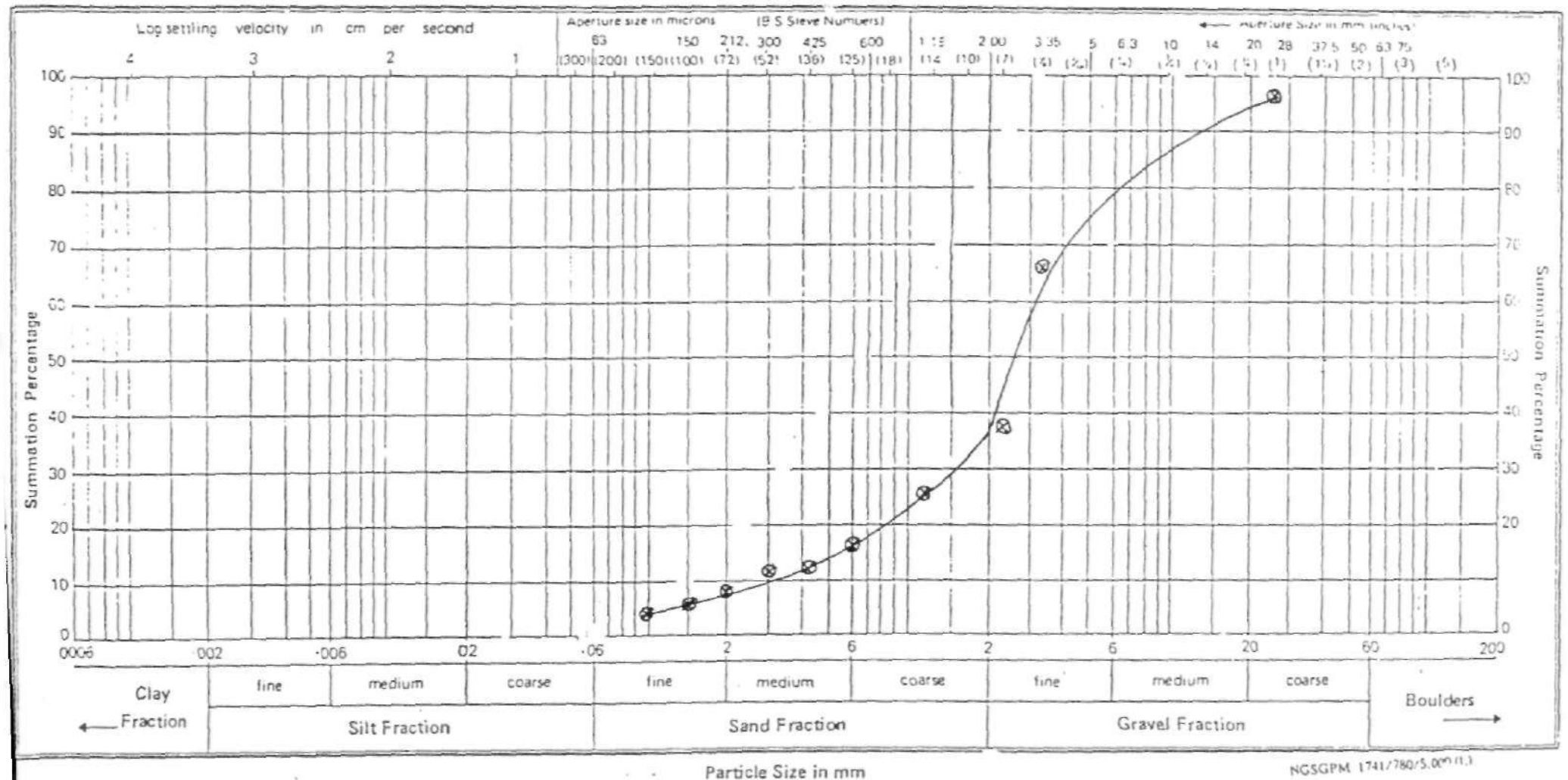


NGSGPM 1741/780/5.00 (1)

Particle Size Distribution

Location SOKOTO Boring No PIT 1 (2.0m) Sample No 2 Date of Test

Description Reddish Brown Clayey Gravel

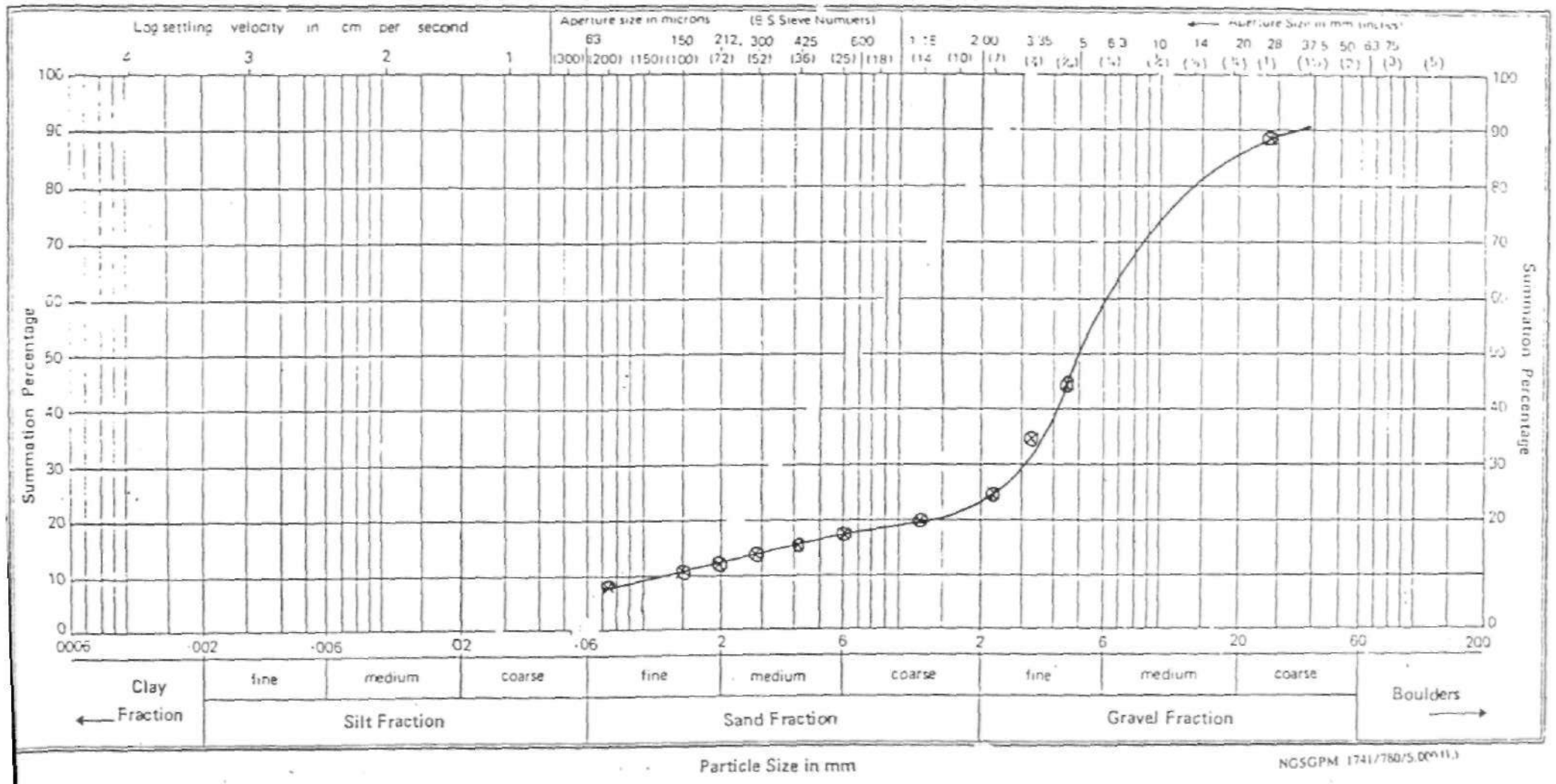


NGSGPM 1741/780/5.00 (1.)

Particle Size Distribution

Location SOKOTO Boring No PIT 2 (1.0m) Sample No 1 Date of Test

Description Reddish Brown Clayey Gravel

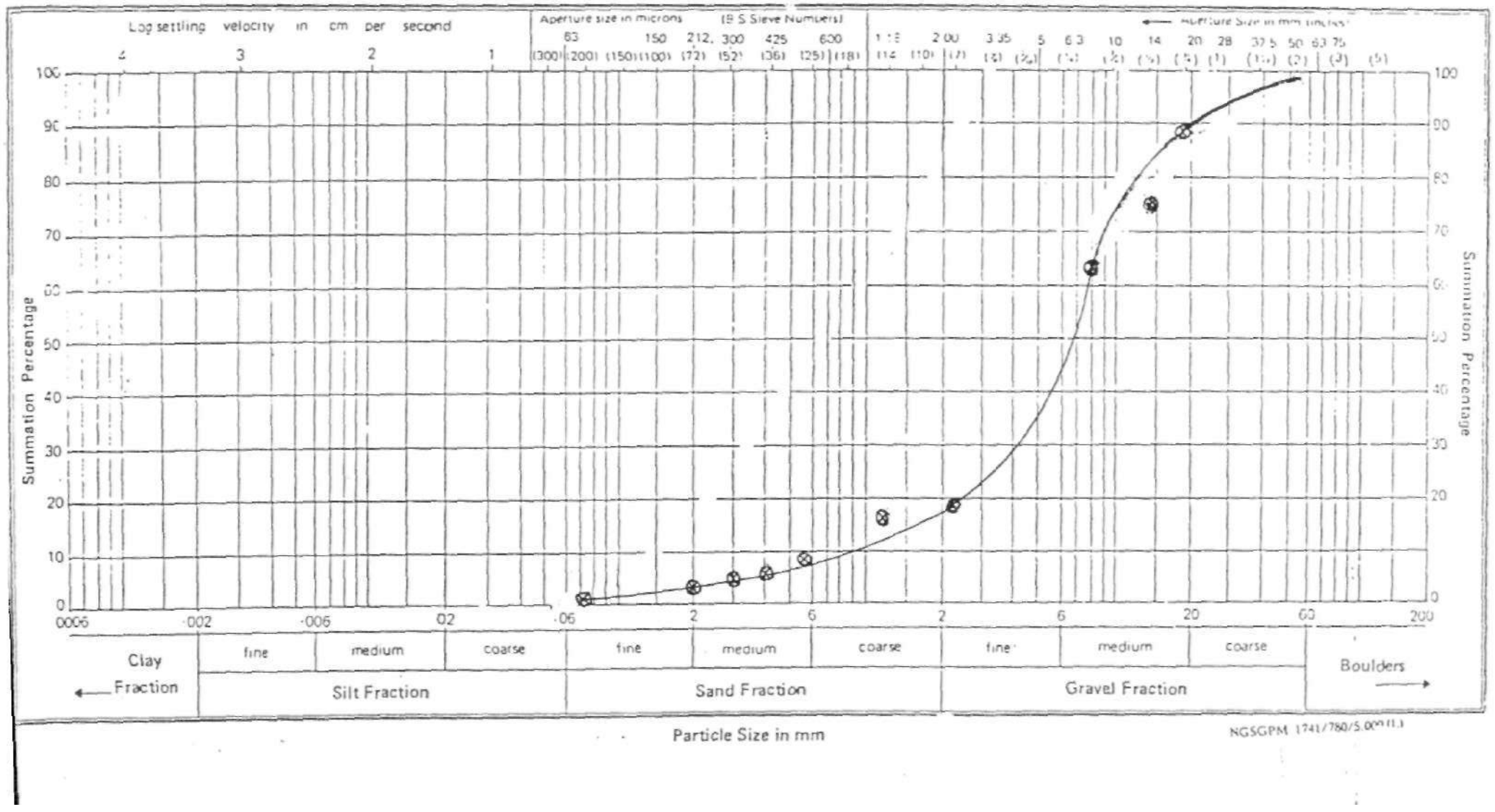


NGSGPM 1741/780/5.000(1.)

Particle Size Distribution

Location SOKOTO Boring No PIT 3 (1.0m) Sample No 1 Date of Test

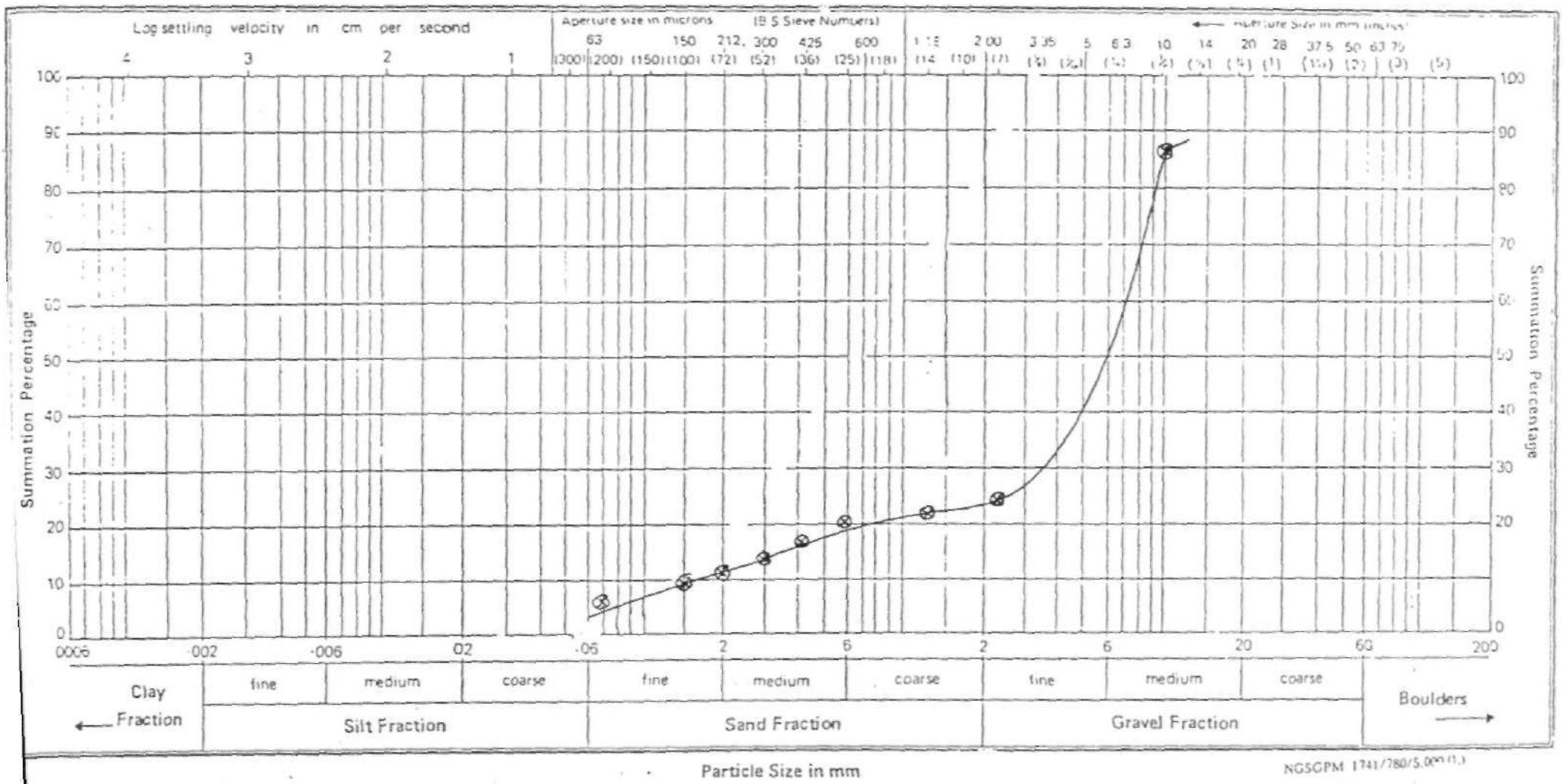
Description Reddish Brown Clayey Gravel



Particle Size Distribution

Location SOKOTO Boring No PIT 3 (2.5m) Sample No 2 Date of Test

Description Reddish Brown Clayey Gravel.



NGSGPM 1741/780/5.000 (1.)

APPENDIX 5

Result of Standard Proctor Compaction Tests.

COMPACTION TEST

Data Sheet 10

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No PIT 1 Sample No. 1

Description of Soil Reddish Brown Clayey Gravel

Test Performed By E. C. O. Date of Test

Blows/Layer 26/3 No. of Layers 3 Wt. of Hammer 24.5 N

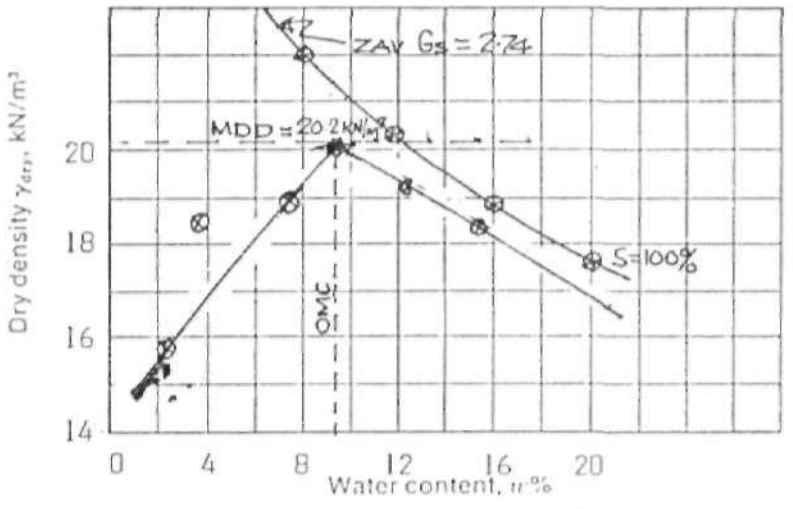
Mold dimensions: Diam. 10.3 cm Ht. 12 cm Vol. 1000 cm³

Water Content Determination

Sample no.	1		2		3		4		5		6	
Moisture can no.	17	55	40	39	34	38	49	30	43	13	41	20
Wt. of can + wet soil	39.6	40.60	42.4	49.8	44.1	44.8	42.8	46.6	41.6	41.9	61.9	57.6
Wt. of can + dry soil	38.8	39.80	41.2	48.3	41.00	42.8	39.7	43.1	37.6	38.4	54.9	51.10
Wt. of water	0.80	0.80	1.20	1.50	3.10	2.00	3.10	3.50	4.00	3.50	7.00	6.50
Wt. of can	9.40	9.10	9.20	9.20	9.00	9.80	9.20	9.20	9.50	9.30	9.00	9.40
Wt. of dry soil	29.5	30.70	32.0	39.0	32.0	33.0	30.7	33.9	32.10	29.10	45.9	41.70
Water content, w%	2.71	2.60	3.75	3.84	9.68	6.06	10.09	8.84	12.46	12.02	15.25	15.58

Density Determination

Assumed water content						
Water content, w% (Av)	2.65	3.79	7.87	9.47	12.24	15.41
Wt. of soil + mold	2700	2995	6125	6299	6242	6235
Wt. of mold	4048	4048	4048	4048	4048	4048
Wt. of soil in mold, g	1652	1947	2076	2251	2194	2187
Wet density, kN/m³	16.21	19.10	20.37	22.08	21.52	21.45
Dry density γ, kN/m³	15.78	18.40	18.88	20.17	19.17	18.32



Optimum moisture = 9.8 % Maximum dry density = 20.2 kN/m³

COMPACTION TEST

Data Sheet 10

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 2 Sample No. 1

Description of Soil Reddish Brown Clayey Gravel

Test Performed By E. C. O. Date of Test

Blows/Layer 26/3 No. of Layers 3 Wt. of Hammer 24.5 N

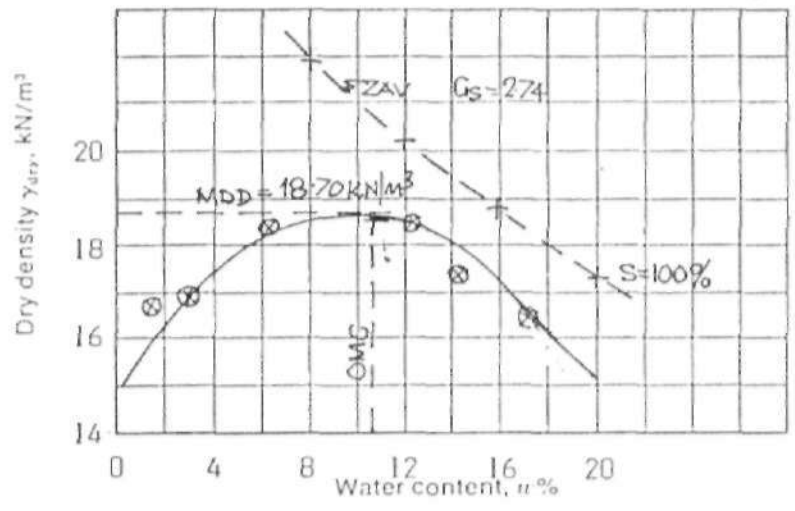
Mold dimensions: Diam. 10.3 cm Ht. 12 cm Vol. 100 cm³

Water Content Determination

Sample no.	1		2		3		4		5		6	
Moisture can no.	73	71	12	60	44	22	57	1	61	24	30	89
Wt. of can + wet soil	35.4	34.9	34.9	31.9	33.6	37.10	37.7	36.9	32.4	37.1	37.6	45.9
Wt. of can + dry soil	35.10	34.3	34.1	31.2	32.1	35.5	34.8	33.8	29.10	33.60	33.3	40.7
Wt. of water	0.30	0.60	0.80	0.70	1.50	1.60	2.90	3.10	3.30	3.50	4.30	5.20
Wt. of can	9.40	9.60	9.0	8.9	9.30	9.30	9.10	9.60	5.5	9.30	9.30	9.50
Wt. of dry soil	25.7	24.7	25.1	22.3	22.8	26.2	25.7	24.2	23.6	24.3	24.00	31.20
Water content, w%	1.17	2.40	3.18	3.13	6.57	6.10	11.28	12.8	13.98	14.40	17.9	16.66

Density Determination

Assumed water content						
Water content, w% (Av)	1.82	3.16	6.34	12.04	14.19	17.28
Wt. of soil + mold	5780	5830	6010	6130	6050	6000
Wt. of mold	4048	4048	4048	4048	4048	4048
Wt. of soil in mold, g	1732	1782	1962	2082	2002	1952
Wet density, kN/m³	16.99	17.48	19.25	20.42	19.64	19.15
Dry density γ, kN/m³	16.69	16.94	18.10	18.22	17.20	16.33



Optimum moisture = 10.8 % Maximum dry density = 18.70 kN/m³

COMPACTION TEST Data Sheet 10

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 1

Description of Soil Reddish Brwon Well graded Clayey Gravel

Test Performed By E. C. O. Date of Test

Blows/Layer 26/3 No. of Layers 3 Wt. of Hammer 24.5 N

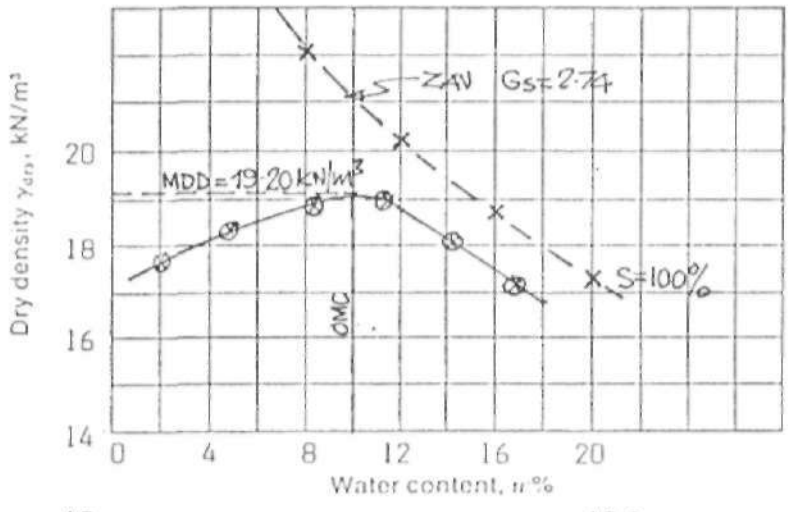
Mold dimensions: Diam. 10.3 cm Ht. 12 cm Vol. 1000 cm³

Water Content Determination

Sample no.	1		2		3		4		5		6	
Moisture can no.	21	8	73	13	49	55	19	11	48	78	10	84
Wt. of can + wet soil	43.9	40.1	43.5	41.6	40.5	40.3	45.6	44.10	38.70	42.3	44.9	36.5
Wt. of can + dry soil	43.2	39.5	41.9	40.00	38.2	37.9	42.10	40.5	37.7	38.2	39.9	32.5
Wt. of water	0.7	0.6	1.60	1.60	2.30	2.40	3.50	3.60	4.0	4.10	5.00	4.00
Wt. of can	9.3	9.00	9.5	8.9	9.10	9.2	9.30	9.00	9.5	8.9	9.3	9.2
Wt. of dry soil	33.90	30.5	32.4	31.10	28.9	28.7	32.8	31.5	28.2	29.3	30.6	23.3
Water content, w%	2.06	1.96	4.93	5.14	7.95	8.36	10.67	11.42	14.18	13.99	16.33	17.16

Density Determination

Assumed water content						
Water content, w% (Ay)	2.01	5.03	8.15	11.04	14.08	16.74
Wt. of soil + mold	5900	6000	6110	6200	6160	6100
Wt. of mold	4048	4048	4048	4048	4048	4048
Wt. of soil in mold, g	1852	1952	2062	2152	2112	2052
Wet density, kN/m³	18.17	19.15	20.23	21.11	20.72	20.13
Dry density γ, kN/m³	17.80	18.23	18.71	19.01	18.16	17.24



Optimum moisture = 10 % Maximum dry density = 19.2 kN/m³

APPENDIX 6

Results of Triaxial tests with Stress/Strain diagrams and corresponding mohr circle diagrams.

TRIAXIAL COMPRESSION TEST

Data Sh

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No PIT 2 Sample No. 1

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /di

	Deform dial reading (×____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Dev str k
1	2	3	4	5	6	7	
	0	0.00	0.00	0.00	1.000	11.40	0
	15	20.10	31.96	0.50	0.995	11.46	27.
	30	25.10	39.91	1.00	0.99	11.52	34.
	60	32.80	52.15	2.00	0.98	11.63	44.
	90	45.00	71.55	3.00	0.97	11.75	60.
	120	52.20	82.99	4.00	0.96	11.88	69.
	150	60.60	96.35	5.00	0.95	12.00	80.
	180	68.00	108.12	6.00	0.94	12.13	89.
	210	72.00	114.48	7.00	0.93	12.26	93.4
	240	76.00	120.84	8.00	0.92	12.39	97.5
	270	84.10	133.72	9.00	0.91	12.53	106.7
	300	86.60	137.69	10.00	0.91	12.67	108.
	360	94.00	149.46	12.00	0.88	12.95	115.
	420	102.10	162.34	14.00	0.86	13.26	122.
	480	108.20	172.04	16.00	0.84	13.57	126.
	540	124.40	197.80	18.00	0.82	13.90	142.
	600	132.80	211.15	20.00	0.80	14.25	148.
	660	142.00	225.78	22.00	0.78	14.62	154.
	720	151.00	240.09	24.00	0.76	15.00	160.
	780	148.20	235.64	26.00	0.74	15.41	152.

Note: Insert units in column headings as necessary

*The Deviator stress computation shown is based on taring the loading system so that the load ring readi deviator load value. σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 310KN/m^2

Maximum deviator stress (from stress-strain curve) σ = 162KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 472KPa

TRIAXIAL COMPRESSION TEST — Data Sheet

Project SOKOTO-RIMA RIVER BASIN H/Q — Job No. MSc Civil 1989.

Location of Project SOKOTO — Boring No PIT 2 — Sample No. 2

Description of Soil: Reddish Brown Well graded Clayey Gravel. — Depth of Sample 2.0m

Tested By E.C.O. — Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ___)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	10.00	15.9	0.50	0.995	11.46	13.9
	30	15.20	24.17	1.00	0.99	11.52	20.9
	60	24.00	38.16	2.00	0.98	11.63	32.8
	90	32.00	50.88	3.00	0.97	11.75	43.3
	120	40.80	64.87	4.00	0.96	11.88	54.6
	150	46.00	73.14	5.00	0.95	12.00	60.9
	180	52.00	82.68	6.00	0.94	12.13	68.2
	210	56.00	89.04	7.00	0.93	12.26	72.6
	240	60.60	96.35	8.00	0.92	12.39	77.8
	270	68.80	109.39	9.00	0.91	12.53	87.3
	300	72.00	114.48	10.00	0.91	12.67	90.4
	360	76.00	120.84	12.00	0.88	12.95	93.3
	420	86.60	137.69	14.00	0.86	13.26	103.8
	480	92.80	147.55	16.00	0.84	13.57	108.7
	540	96.10	152.80	18.00	0.82	13.90	109.9
	600	102.00	162.18	20.00	0.80	14.25	113.8
	660	103.00	163.77	22.00	0.78	14.62	112.0
	720	102.00	162.18	24.00	0.76	15.00	108.1
	780			26.00	0.74	15.41	

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is deviator load value. σ = (Col. 3 × LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 207KP^a

Maximum deviator stress (from stress-strain curve) σ = 114KP^a

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 321KP^a

TRIAXIAL COMPRESSION TEST

Data She

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 2 Sample No. 3.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Devia stres kP
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.
	15	8.00	12.72	0.50	0.995	11.46	11.
	30	12.20	19.40	1.00	0.99	11.52	16.
	60	13.40	21.31	2.00	0.98	11.63	18.
	90	20.10	31.96	3.00	0.97	11.75	27.
	120	24.30	38.64	4.00	0.96	11.88	32.
	150	28.80	45.79	5.00	0.95	12.00	38.
	180	36.60	58.19	6.00	0.94	12.13	48.
	210	40.00	63.6	7.00	0.93	12.26	51.
	240	42.00	66.78	8.00	0.92	12.39	53.
	270	46.70	74.25	9.00	0.91	12.53	59.
	300	48.20	76.64	10.00	0.91	12.67	60.
	360	56.60	90.00	12.00	0.88	12.95	69.
	420	62.20	98.90	14.00	0.86	13.26	74.
	480	68.10	108.28	16.00	0.84	13.57	79.
	540	74.10	117.82	18.00	0.82	13.90	84.
	600	80.00	127.20	20.00	0.80	14.25	89.
	660	80.00	127.20	22.00	0.78	14.62	87.
	720	78.80	125.29	24.00	0.76	15.00	83.
	780			26.00	0.74	15.41	

Note: Insert units in column headings as necessary.

"The Deviator stress computation shown is based on taring the loading system so that the load ring readin deviator load value σ = (Col. 3 × LRC)/ A'.

Computed Data

Lateral pressure on test specimen σ_3 = 103.4KPa

Maximum deviator stress (from stress-strain curve) σ = 90.0KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 193KPa

TRIAXIAL COMPRESSION TEST — Data Sheet

Project: SOKOTO-RIMA RIVER BASIN H/Q — Job No. MSc Civil 1989.

Location of Project SOKOTO — Boring No. PIT 2 — Sample No. 1.

Description of Soil: Reddish Brown Well graded Clayey Gravel. — Depth of Sample 3.0m

Tested By E.C.O. — Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform dial reading (×___)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (Cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	22.30	34.98	0.50	0.995	11.46	30.5
	30	25.40	40.39	1.00	0.99	11.52	35.1
	60	35.00	55.65	2.00	0.98	11.63	47.9
	90	45.00	71.55	3.00	0.97	11.75	60.9
	120	52.80	83.95	4.00	0.96	11.88	70.7
	150	60.80	96.67	5.00	0.95	12.00	80.6
	180	66.00	104.94	6.00	0.94	12.13	86.5
	210	70.00	111.3	7.00	0.93	12.26	90.8
	240	74.00	117.66	8.00	0.92	12.39	95.0
	270	76.80	122.11	9.00	0.91	12.53	97.5
	300	79.10	125.77	10.00	0.91	12.67	99.3
	360	90.20	143.42	12.00	0.88	12.95	110.8
	420	103.40	164.4	14.00	0.86	13.26	124.0
	480	115.20	183.20	16.00	0.84	13.57	135.0
	540	136.50	217.04	18.00	0.82	13.90	156.1
	600	150.00	238.5	20.00	0.80	14.25	167.4
	660	160.20	254.72	22.00	0.78	14.62	174.2
	720	154.00	244.86	24.00	0.76	15.00	163.2
	780	147.00	233.73	26.00	0.74	15.41	151.7

Note: Insert units in column headings as necessary

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is t
deviator load value. σ = (Col. 3× LRC)/A'

Computed Data

Lateral pressure on test specimen σ_3 = 310KPa

Maximum deviator stress (from stress-strain curve) σ = 175KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 485KPa

TRIAXIAL COMPRESSION TEST

Data Sheet 19

Project SOKOTO-RIMA RIVER BASIN B/O Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No PIT 2 Sample No. 2

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (×____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'_2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	20.20	32.12	0.50	0.995	11.46	28.0
	30	22.80	36.25	1.00	0.99	11.52	31.5
	60	26.50	42.14	2.00	0.98	11.63	36.2
	90	30.20	48.02	3.00	0.97	11.75	40.9
	120	36.80	58.51	4.00	0.96	11.88	49.3
	150	42.30	67.26	5.00	0.95	12.00	56.1
	180	46.80	74.41	6.00	0.94	12.13	61.3
	210	50.00	79.50	7.00	0.93	12.26	64.8
	240	53.10	84.43	8.00	0.92	12.39	68.1
	270	56.50	89.84	9.00	0.91	12.53	71.7
	300	58.00	92.22	10.00	0.91	12.67	72.8
	360	64.40	102.40	12.00	0.88	12.95	79.1
	420	67.80	107.8	14.00	0.86	13.26	81.3
	480	75.40	119.89	16.00	0.84	13.57	88.3
	540	85.00	135.15	18.00	0.82	13.90	97.2
	600	93.00	147.87	20.00	0.80	14.25	103.8
	660	102.50	162.98	22.00	0.78	14.62	111.5
	720	102.00	162.18	24.00	0.76	15.00	108.1
	780	98.00	155.82	26.00	0.74	15.41	101.1

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is th deviator load value. σ = (Col. 3× LRC)/ A'

Computed Data

Lateral pressure on test specimen σ_3 = 207KPa

Maximum deviator stress (from stress-strain curve) σ = 112KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 319KPa

TRIAXIAL COMPRESSION TEST Data Shee

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 2 Sample No. 3

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.0
	15	13.50	21.47	0.50	0.995	11.46	18.7
	30	13.50	21.47	1.00	0.99	11.52	18.6
	60	15.80	25.12	2.00	0.98	11.63	21.6
	90	20.80	33.07	3.00	0.97	11.75	28.2
	120	24.40	38.80	4.00	0.96	11.88	32.7
	150	28.80	45.79	5.00	0.95	12.00	38.2
	180	34.80	55.33	6.00	0.94	12.13	45.6
	210	36.60	58.19	7.00	0.93	12.26	47.5
	240	38.10	60.58	8.00	0.92	12.39	48.9
	270	42.10	66.94	9.00	0.91	12.53	53.4
	300	46.60	74.09	10.00	0.91	12.67	58.5
	360	52.80	83.95	12.00	0.88	12.95	64.8
	420	56.80	90.3	14.00	0.86	13.26	68.1
	480	62.80	99.9	16.00	0.84	13.57	73.6
	540	68.20	108.4	18.00	0.82	13.90	78.0
	600	72.50	115.3	20.00	0.80	14.25	80.9
	660	76.20	121.16	22.00	0.78	14.62	82.9
	720	80.50	127.99	24.00	0.76	15.00	85.3
	780	78.20	117.98	26.00	0.74	15.41	76.6

Note: Insert units in column headings as necessary

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is deviator load value. σ = (Col. 3× LRC)/A'

Computed Data

Lateral pressure on test specimen σ_3 = 103KPa

Maximum deviator stress (from stress-strain curve) σ = 86KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 189KPa

TRIAXIAL COMPRESSION TEST — Data Sheet 19

Project SOKOTO-RIMA RIVER BASIN H/Q — Job No. MSc Civil 1989.

Location of Project SOKOTO — Boring No PIT 3 — Sample No. 1

Description of Soil: Reddish Brown Well graded Clayey Gravel. — Depth of Sample 2.0m

Tested By E.C.O. — Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ___)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.0
	15	97.00	154.23	0.50	0.995	11.46	134.6
	30	190.00	302.10	1.00	0.99	11.52	262.2
	60	350.00	556.50	2.00	0.98	11.63	478.5
	90	372.50	592.28	3.00	0.97	11.75	504.1
	120	380.50	604.99	4.00	0.96	11.88	509.2
	150	383.00	608.97	5.00	0.95	12.00	507.5
	180	387.20	615.65	6.00	0.94	12.13	507.5
	210	389.00	618.51	7.00	0.93	12.26	504.5
	240	390.00	620.10	8.00	0.92	12.39	500.5
	270	392.00	623.28	9.00	0.91	12.53	497.4
	300	394.00	626.46	10.00	0.91	12.67	494.4
	360	396.40	630.28	12.00	0.88	12.95	486.70
	420	402.60	640.13	14.00	0.86	13.26	482.7
	480	406.10	645.70	16.00	0.84	13.57	475.8
	540	409.70	651.42	18.00	0.82	13.90	468.6
	600	403.20	641.09	20.00	0.80	14.25	449.9
	660	401.00	637.59	22.00	0.78	14.62	436.1
	720			24.00	0.76	15.00	
	780			26.00	0.74	15.41	

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value σ = (Col. 3× LRC)/A'

Computed Data

Lateral pressure on test specimen σ_3 = 310KPa

Maximum deviator stress (from stress-strain curve) σ = 510KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 820KPa

TRIAXIAL COMPRESSION TEST — Data Sheet 1

Project SOKOTO-RIMA RIVER BASIN H/Q — Job No. MSc Civil 1989.

Location of Project SOKOTO — Boring No PIT 3 — Sample No. 2

Description of Soil: Reddish Brown Well graded Clayey Gravel. — Depth of Sample 2.0m

Tested By E.C.O. — Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0°	0.00	0.00	0.00	1.000	11.40	0.0
	15	90.00	143.1	0.50	0.995	11.46	124.9
	30	160.00	254.4	1.00	0.99	11.52	220.8
	60	228.00	362.52	2.00	0.98	11.63	311.7
	90	242.00	384.78	3.00	0.97	11.75	327.5
	120	246.30	391.62	4.00	0.96	11.88	329.6
	150	253.00	402.27	5.00	0.95	12.00	335.2
	180	260.20	413.72	6.00	0.94	12.13	341.1
	210	262.80	417.85	7.00	0.93	12.26	340.8
	240	264.20	420.08	8.00	0.92	12.39	339.0
	270	265.80	422.62	9.00	0.91	12.53	337.3
	300	268.00	426.12	10.00	0.91	12.67	336.3
	360	269.80	428.98	12.00	0.88	12.95	331.2
	420	272.80	433.75	14.00	0.86	13.26	327.1
	480	275.80	438.52	16.00	0.84	13.57	323.2
	540	276.80	440.11	18.00	0.82	13.90	316.6
	600	280.20	445.52	20.00	0.80	14.25	312.6
	660	277.50	441.23	22.00	0.78	14.62	301.8
	720	273.00	434.07	24.00	0.76	15.00	289.4
	780			26.00	0.74	15.41	

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is th deviator load value. σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 207KPa

Maximum deviator stress (from stress-strain curve) σ = 342KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 549KPa

TRIAXIAL COMPRESSION TEST — Data Sheet 1

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (×____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (Cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.0
	15	120.00	190.8	0.50	0.995	11.46	166.5
	30	145.00	230.55	1.00	0.99	11.52	200.1
	60	151.50	240.89	2.00	0.98	11.63	207.1
	90	155.00	246.45	3.00	0.97	11.75	209.7
	120	160.00	254.4	4.00	0.96	11.88	214.1
	150	163.80	260.44	5.00	0.95	12.00	217.0
	180	166.40	264.58	6.00	0.94	12.13	218.1
	210	168.00	267.12	7.00	0.93	12.26	217.9
	240	169.80	269.98	8.00	0.92	12.39	217.9
	270	172.00	273.5	9.00	0.91	12.53	218.3
	300	174.00	276.66	10.00	0.91	12.67	218.3
	360	176.80	281.11	12.00	0.88	12.95	217.07
	420	179.80	285.88	14.00	0.86	13.26	215.6
	480	183.00	290.97	16.00	0.84	13.57	214.4
	540	185.20	294.20	18.00	0.82	13.90	211.8
	600	187.80	298.60	20.00	0.80	14.25	209.5
	660	192.50	306.08	22.00	0.78	14.62	209.3
	720	196.00	311.64	24.00	0.76	15.00	207.8
	780	193.50	307.67	26.00	0.74	15.41	199.7

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value. σ = (Col. 3 × LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 103KPa

Maximum deviator stress (from stress-strain curve) σ = 219KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 322KPa

TRIAXIAL COMPRESSION TEST — Data Sheet

Project SOKOTO-RIMA RIVER BASIN H/Q — Job No. MSc Civil 1989.

Location of Project SOKOTO — Boring No. PII 3 — Sample No. 1

Description of Soil: Reddish Brown Well graded Clayey Gravel. — Depth of Sample 3.0m

Tested By E.C.O. — Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	10.00	15.9	0.50	0.995	11.46	13.9
	30	21.00	33.39	1.00	0.99	11.52	28.9
	60	46.50	73.94	2.00	0.98	11.63	63.6
	90	68.00	108.12	3.00	0.97	11.75	92.0
	120	74.00	117.66	4.00	0.96	11.88	99.0
	150	80.80	128.47	5.00	0.95	12.00	107.1
	180	84.00	133.56	6.00	0.94	12.13	110.1
	210	86.00	136.74	7.00	0.93	12.26	111.5
	240	94.00	149.46	8.00	0.92	12.39	120.6
	270	96.00	152.64	9.00	0.91	12.53	121.8
	300	122.00	193.98	10.00	0.91	12.67	153.1
	360	132.00	209.88	12.00	0.88	12.95	162.1
	420	136.00	216.24	14.00	0.86	13.26	163.1
	480	156.00	248.04	16.00	0.84	13.57	182.8
	540	168.00	267.12	18.00	0.82	13.90	192.2
	600	178.00	283.02	20.00	0.80	14.25	198.6
	660	186.00	295.74	22.00	0.78	14.62	202.3
	720	198.00	314.82	24.00	0.76	15.00	209.9
	780	196.80	311.64	26.00	0.74	15.41	202.2

Note. Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is t
deviator load value. σ = (Col. 3× LRC)/A'

Computed Data

Lateral pressure on test specimen σ_3 = 310KPa

Maximum deviator stress (from stress-strain curve) σ = 210KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 520KPa

TRIAXIAL COMPRESSION TEST — Data Sheet

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No. 2.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0°	0.00	0.00	0.00	1.000	11.40	0.00
	15	12.00	19.08	0.50	0.995	11.46	16.7
	30	14.00	22.26	1.00	0.99	11.52	19.3
	60	20.80	33.07	2.00	0.98	11.63	28.4
	90	26.00	41.34	3.00	0.97	11.75	35.2
	120	34.80	55.33	4.00	0.96	11.88	46.6
	150	42.80	68.05	5.00	0.95	12.00	96.7
	180	54.00	85.86	6.00	0.94	12.13	70.78
	210	58.10	92.38	7.00	0.93	12.26	75.35
	240	64.00	101.76	8.00	0.92	12.39	82.1
	270	68.50	108.92	9.00	0.91	12.53	86.9
	300	74.80	118.93	10.00	0.91	12.67	93.9
	360	84.00	133.56	12.00	0.88	12.95	103.1
	420	94.00	149.46	14.00	0.86	13.26	112.7
	480	104.00	165.36	16.00	0.84	13.57	121.9
	540	126.00	200.34	18.00	0.82	13.90	144.1
	600	146.00	232.14	20.00	0.80	14.25	162.9
	660	160.00	254.4	22.00	0.78	14.62	174.00
	720	167.00	265.53	24.00	0.76	15.00	177.0
	780	167.00	265.53	26.00	0.74	15.41	172.3

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is deviator load value. σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 207KPa

Maximum deviator stress (from stress-strain curve) σ = 170KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 377KPa

TRIAXIAL COMPRESSION TEST — Data Sheet

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0°	0.00	0.00	0.00	1.000	11.40	0.0
	15	10.00	15.90	0.50	0.995	11.46	13.9
	30	12.80	20.35	1.00	0.99	11.52	17.7
	60	18.00	28.62	2.00	0.98	11.63	24.6
	90	34.00	54.06	3.00	0.97	11.75	46.0
	120	54.00	85.86	4.00	0.96	11.88	72.3
	150	58.00	92.23	5.00	0.95	12.00	76.9
	180	65.50	104.15	6.00	0.94	12.13	85.9
	210	68.00	108.12	7.00	0.93	12.26	88.2
	240	72.00	114.48	8.00	0.92	12.39	92.4
	270	76.00	120.84	9.00	0.91	12.53	96.4
	300	78.00	124.02	10.00	0.91	12.67	97.9
	360	89.00	141.51	12.00	0.88	12.95	109.3
	420	95.00	151.05	14.00	0.86	13.26	113.9
	480	103.80	165.04	16.00	0.84	13.57	121.6
	540	108.50	172.5	18.00	0.82	13.90	124.1
	600	111.00	176.49	20.00	0.80	14.25	123.9
	660	111.00	176.49	22.00	0.78	14.62	120.7
	720	111.00	176.49	24.00	0.76	15.00	117.7
	780			26.00	0.74	15.41	

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is t deviator load value. σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 103KPa

Maximum deviator stress (from stress-strain curve) σ = 125KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 228KPa.

TRIAXIAL COMPRESSION TEST — Data Sheet 19

Project SOKOTO-RIMA RIVER BASIN H/Q — Job No. MSc Civil 1989.

Location of Project SOKOTO — Boring No. PII — Sample No. 1

Description of Soil: Reddish Brown Well graded Clayey Gravel. — Depth of Sample 2.0m

Tested By E.C.O. — Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.0
	15	15.00	23.85	0.50	0.995	11.46	20.8
	30	15.80	25.12	1.00	0.99	11.52	21.8
	60	16.80	26.71	2.00	0.98	11.63	22.9
	90	20.10	31.96	3.00	0.97	11.75	27.2
	120	25.00	39.75	4.00	0.96	11.88	33.5
	150	32.10	51.04	5.00	0.95	12.00	42.5
	180	38.50	61.22	6.00	0.94	12.13	50.5
	210	42.00	66.78	7.00	0.93	12.26	54.5
	240	48.10	76.48	8.00	0.92	12.39	61.7
	270	56.10	89.20	9.00	0.91	12.53	71.2
	300	58.50	93.01	10.00	0.91	12.67	73.4
	360	68.50	108.92	12.00	0.88	12.95	84.1
	420	78.80	125.3	14.00	0.86	13.26	94.5
	480	95.00	150.1	16.00	0.84	13.57	110.6
	540	103.20	163.77	18.00	0.82	13.90	117.8
	600	115.60	183.80	20.00	0.80	14.25	128.9
	660	125.00	198.75	22.00	0.78	14.62	135.9
	720	118.20	187.94	24.00	0.76	15.00	125.3
	780	116.00	184.44	26.00	0.74	15.41	119.7

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value. σ = (Col. 3 × LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 310KPa

Maximum deviator stress (from stress-strain curve) σ = 136KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 446KPa

TRIAXIAL COMPRESSION TEST — Data Sheet 1

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No. 2.

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.0
	15	12.30	19.56	0.50	0.995	11.46	17.1
	30	14.50	23.06	1.00	0.99	11.52	20.0
	60	18.20	28.94	2.00	0.98	11.63	24.9
	90	25.00	39.75	3.00	0.97	11.75	33.8
	120	28.10	44.68	4.00	0.96	11.88	37.6
	150	32.20	51.20	5.00	0.95	12.00	42.7
	180	38.20	60.74	6.00	0.94	12.13	50.1
	210	40.50	64.40	7.00	0.93	12.26	52.5
	240	42.50	67.58	8.00	0.92	12.39	54.5
	270	46.50	73.94	9.00	0.91	12.53	57.0
	300	50.80	80.77	10.00	0.91	12.67	63.7
	360	55.60	88.40	12.00	0.88	12.95	68.3
	420	59.60	94.76	14.00	0.86	13.26	71.5
	480	64.80	103.03	16.00	0.84	13.57	75.9
	540	72.40	115.12	18.00	0.82	13.90	82.8
	600	76.80	122.11	20.00	0.80	14.25	85.7
	660	80.40	127.84	22.00	0.78	14.62	87.4
	720	80.00	127.2	24.00	0.76	15.00	84.8
	780	78.20	124.34	26.00	0.74	15.41	80.7

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is th
deviator load value. σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 207KPa

Maximum deviator stress (from stress-strain curve) σ = 88KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 295KPa

TRIAXIAL COMPRESSION TEST Data Sheet 19

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 2.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	10.20	16.22	0.50	0.995	11.46	14.2
	30	10.80	17.17	1.00	0.99	11.52	14.9
	60	12.80	20.35	2.00	0.98	11.63	17.5
	90	14.20	22.58	3.00	0.97	11.75	19.2
	120	16.50	26.24	4.00	0.96	11.88	22.1
	150	20.20	32.12	5.00	0.95	12.00	26.8
	180	24.30	38.64	6.00	0.94	12.13	31.8
	210	26.80	42.61	7.00	0.93	12.26	34.8
	240	28.00	44.52	8.00	0.92	12.39	35.9
	270	30.20	48.02	9.00	0.91	12.53	38.3
	300	33.40	53.11	10.00	0.91	12.67	41.9
	360	38.10	60.58	12.00	0.88	12.95	46.8
	420	42.50	67.58	14.00	0.86	13.26	51.0
	480	46.10	73.30	16.00	0.84	13.57	54.0
	540	50.80	80.77	18.00	0.82	13.90	58.1
	600	56.00	89.04	20.00	0.80	14.25	62.5
	660	62.00	98.58	22.00	0.78	14.62	67.4
	720	70.00	111.3	24.00	0.76	15.00	74.2
	780	67.50	106.53	26.00	0.74	15.41	69.1

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value. σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 103.4KPa

Maximum deviator stress (from stress-strain curve) σ = 75.KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 178KPa

TRIAXIAL COMPRESSION TEST

Data Sheet 19

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No. 1

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E.C.O. Date of Testing W = 18%

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (×____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	20.00	31.8	0.50	0.995	11.46	27.8
	30	21.00	33.39	1.00	0.99	11.52	29.0
	60	24.80	39.43	2.00	0.98	11.63	33.9
	90	28.00	44.52	3.00	0.97	11.75	37.9
	120	32.00	50.88	4.00	0.96	11.88	42.8
	150	38.00	60.42	5.00	0.95	12.00	50.4
	180	42.00	66.78	6.00	0.94	12.13	55.1
	210	46.00	73.14	7.00	0.93	12.26	59.7
	240	50.00	79.5	8.00	0.92	12.39	64.2
	270	52.00	82.68	9.00	0.91	12.53	66.0
	300	54.00	85.86	10.00	0.91	12.67	67.8
	360	62.00	98.58	12.00	0.88	12.95	76.1
	420	66.00	104.94	14.00	0.86	13.26	77.3
	480	72.00	114.48	16.00	0.84	13.57	84.4
	540	76.00	120.84	18.00	0.82	13.90	86.9
	600	82.00	130.38	20.00	0.80	14.25	91.5
	660	90.00	143.10	22.00	0.78	14.62	97.9
	720	94.00	149.46	24.00	0.76	15.00	99.6
	780	96.00	152.64	26.00	0.74	15.41	99.1

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value. σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 310KPa

Maximum deviator stress (from stress-strain curve) σ = 100KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 410KPa

TRIAXIAL COMPRESSION TEST — Data Sheet 19

Project SOKOTO-RIMA RIVER BASIN H/Q — Job No. MSc Civil 1989.

Location of Project SOKOTO — Boring No. PIT 3 — Sample No. 2

Description of Soil: Reddish Brown Well graded Clayey Gravel. — Depth of Sample 3.0m

Tested By E.C.O. — Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = _____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (× ____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor 1-ε	Correct. area A'2 (Cm^2)	Deviator stress* kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	15.00	23.85	0.50	0.995	11.46	20.81
	30	15.50	24.65	1.00	0.99	11.52	21.40
	60	18.00	28.62	2.00	0.98	11.63	24.8
	90	22.00	34.98	3.00	0.97	11.75	29.8
	120	24.00	38.16	4.00	0.96	11.88	32.1
	150	26.20	41.66	5.00	0.95	12.00	34.7
	180	30.10	47.86	6.00	0.94	12.13	39.5
	210	32.00	50.88	7.00	0.93	12.26	41.5
	240	34.00	54.06	8.00	0.92	12.39	43.6
	270	38.20	60.74	9.00	0.91	12.53	48.5
	300	42.80	68.05	10.00	0.91	12.67	53.7
	360	46.20	73.46	12.00	0.88	12.95	56.7
	420	52.10	82.84	14.00	0.86	13.26	62.5
	480	56.20	89.36	16.00	0.84	13.57	65.9
	540	60.20	95.72	18.00	0.82	13.90	68.9
	600	64.30	102.24	20.00	0.80	14.25	71.7
	660	69.10	109.87	22.00	0.78	14.62	75.2
	720	72.00	114.48	24.00	0.76	15.00	76.3
	780	71.60	113.84	26.00	0.74	15.41	73.9

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value σ = (Col. 3× LRC)/A'.

Computed Data

Lateral pressure on test specimen σ_3 = 207KPa

Maximum deviator stress (from stress-strain curve) σ = 77KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 284KPa

TRIAXIAL COMPRESSION TEST Data Sheet 19

Project SOKOTO-RIMA RIVER BASIN H/Q Job No. MSc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil: Reddish Brown Well graded Clayey Gravel. Depth of Sample 3.0m

Tested By E.C.O. Date of Testing

Sample Data: Area A_o = 11.4 cm^2 Length L_o = 7.5cm (1.59N/div)

Machine Data: Load rate = ____ /min Load ring constant LRC = 0.162Kg /div.

	Deform. dial reading (×____)	Load dial reading	Compressive Load (N)	Strain %	Area correction factor $1-\epsilon$	Correct. area A' (cm^2)	Deviator stress*, kPa
1	2	3	4	5	6	7	8
	0	0.00	0.00	0.00	1.000	11.40	0.00
	15	10.00	15.90	0.50	0.995	11.46	13.9
	30	11.00	17.49	1.00	0.99	11.52	15.2
	60	12.80	20.35	2.00	0.98	11.63	17.5
	90	14.80	23.53	3.00	0.97	11.75	20.0
	120	16.00	25.44	4.00	0.96	11.88	21.4
	150	16.80	26.71	5.00	0.95	12.00	22.3
	180	19.40	30.85	6.00	0.94	12.13	25.4
	210	20.20	32.12	7.00	0.93	12.26	26.5
	240	22.00	34.98	8.00	0.92	12.39	28.2
	270	23.00	36.57	9.00	0.91	12.53	29.2
	300	24.00	38.16	10.00	0.91	12.67	30.1
	360	26.20	41.66	12.00	0.88	12.95	32.2
	420	30.50	48.50	14.00	0.86	13.26	36.6
	480	36.00	57.24	16.00	0.84	13.57	42.2
	540	42.60	67.73	18.00	0.82	13.90	48.7
	600	46.00	73.14	20.00	0.80	14.25	51.3
	660	50.80	80.77	22.00	0.78	14.62	55.2
	720	54.00	85.86	24.00	0.76	15.00	56.2
	780	50.00	79.50	26.00	0.74	15.41	51.6

Note: Insert units in column headings as necessary.

*The Deviator stress computation shown is based on taring the loading system so that the load ring reading is the deviator load value. σ = (Col. 3× LRC)/A'.

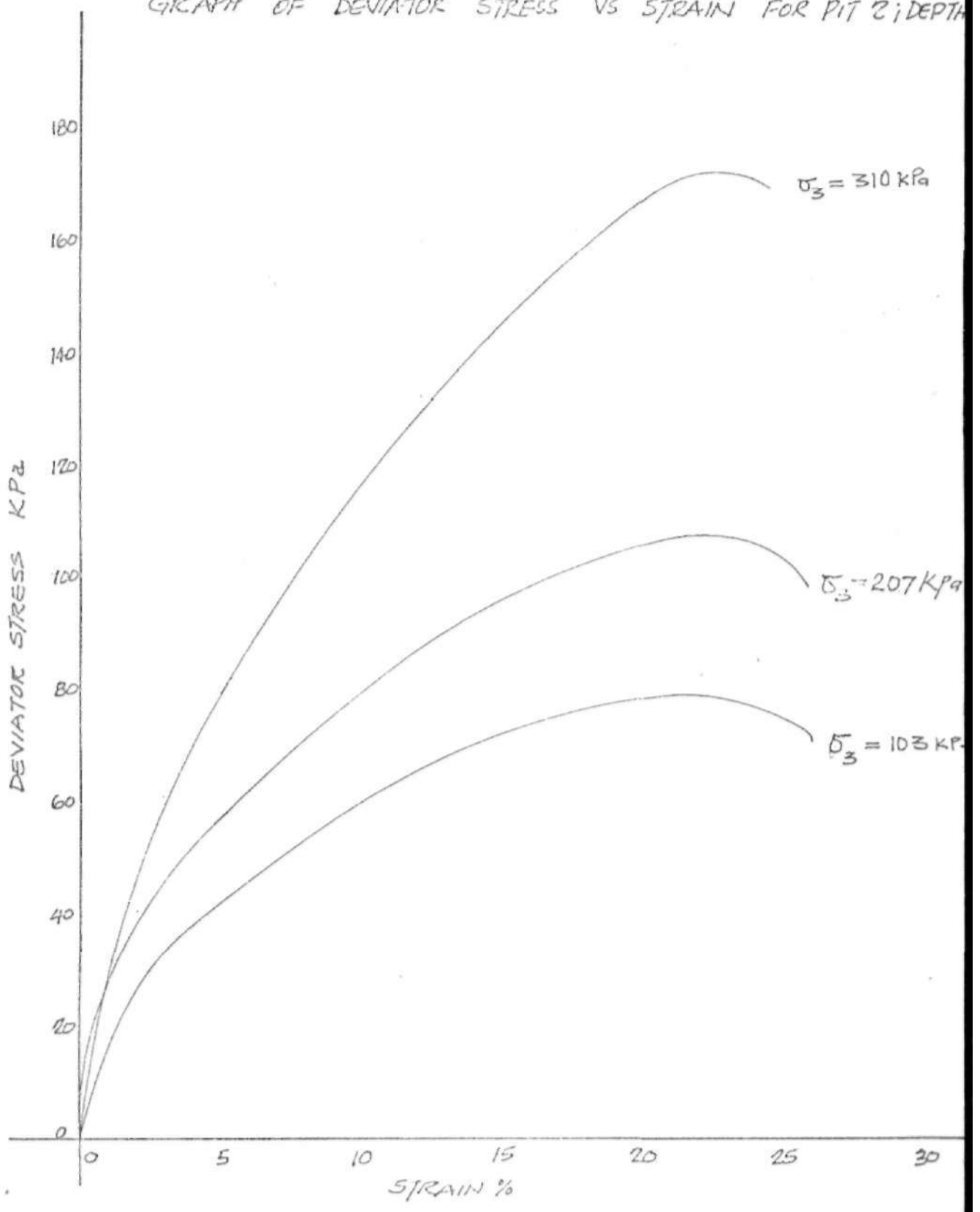
Computed Data

Lateral pressure on test specimen σ_3 = 103KPa

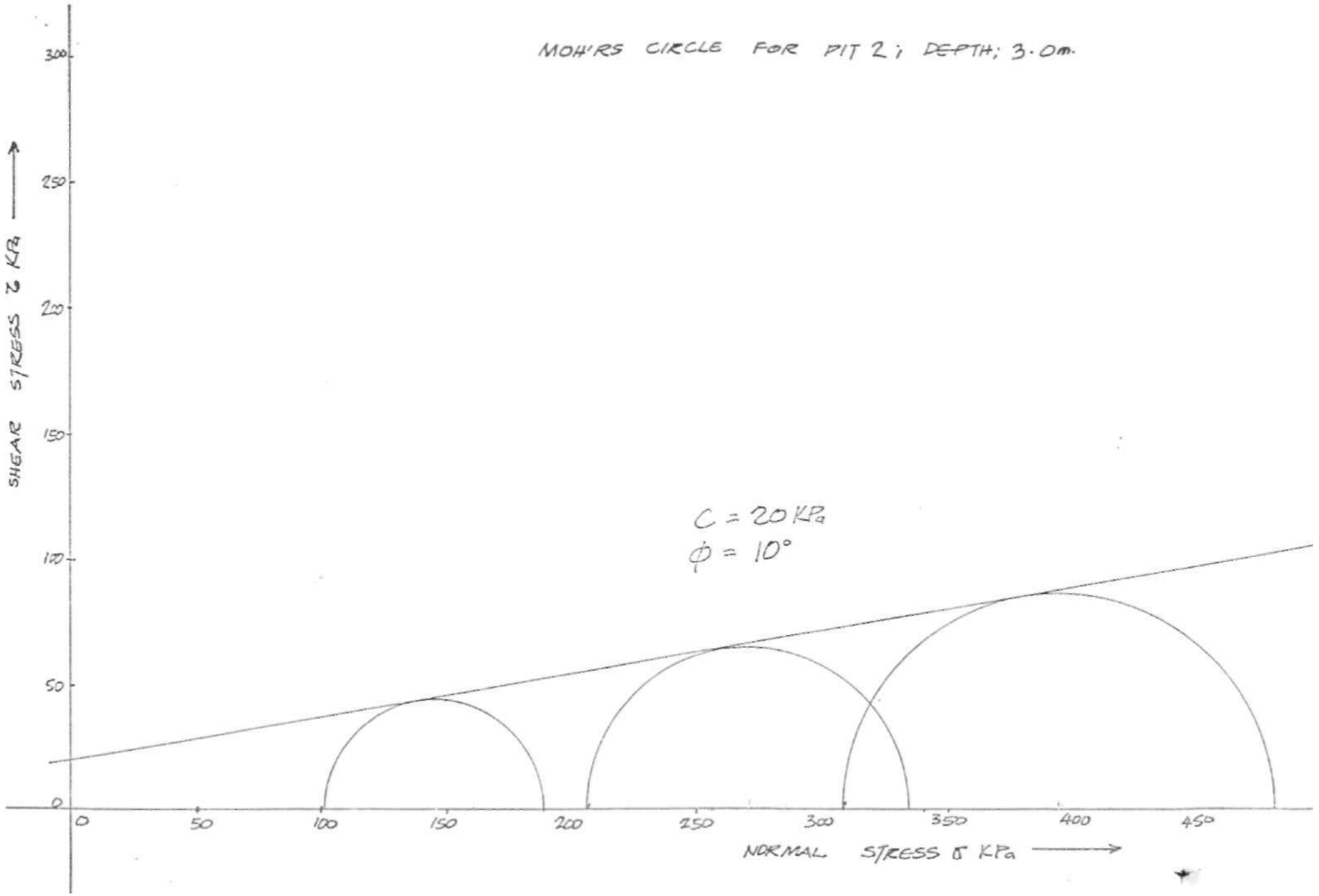
Maximum deviator stress (from stress-strain curve) σ = 58KPa

Maximum value of vertical stress $\sigma_1 = \sigma + \sigma_3$ = 161KPa

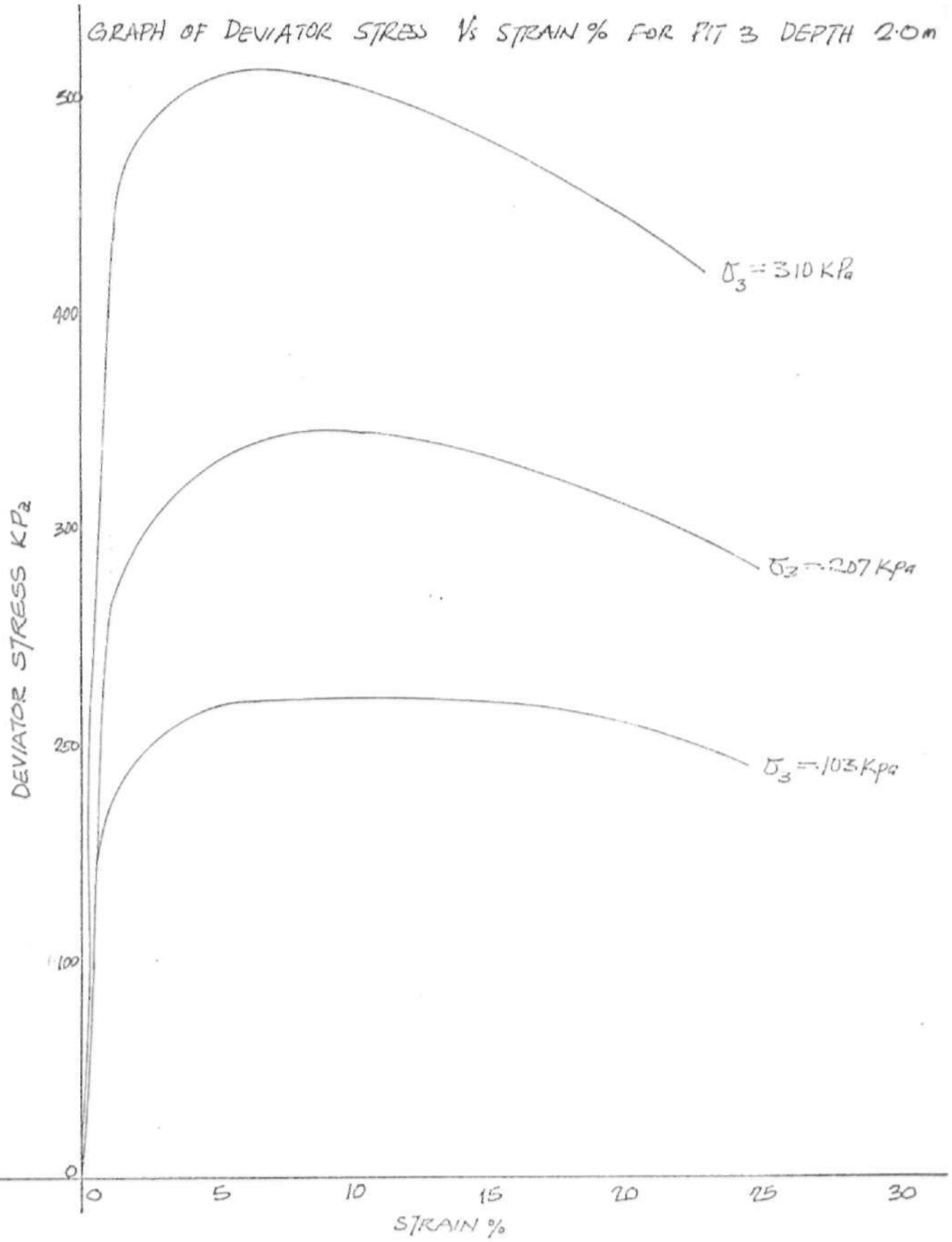
GRAPH OF DEVIATOR STRESS Vs STRAIN FOR PIT 2; DEPTH



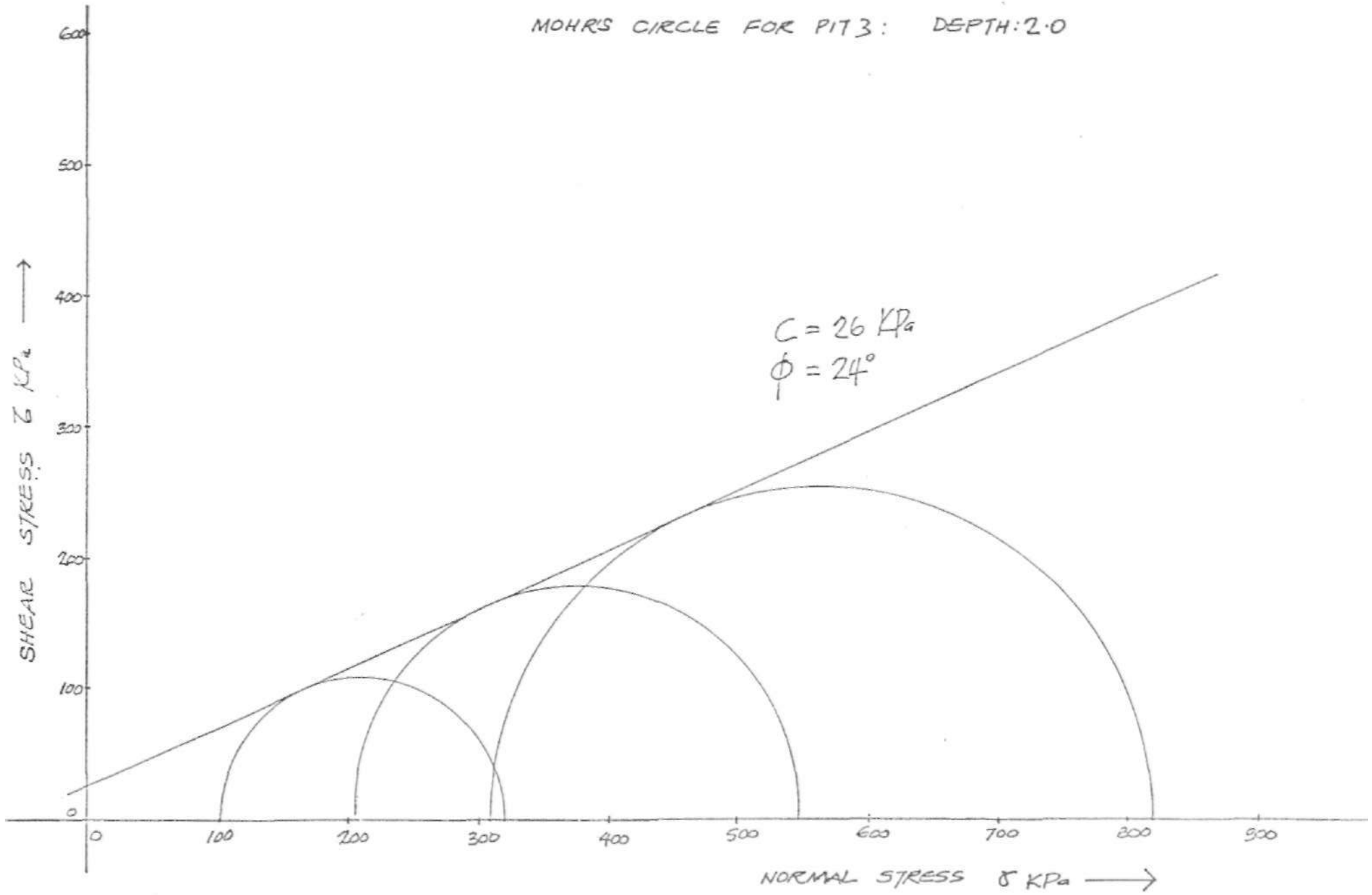
MOHR'S CIRCLE FOR PIT 2; DEPTH: 3.0m.

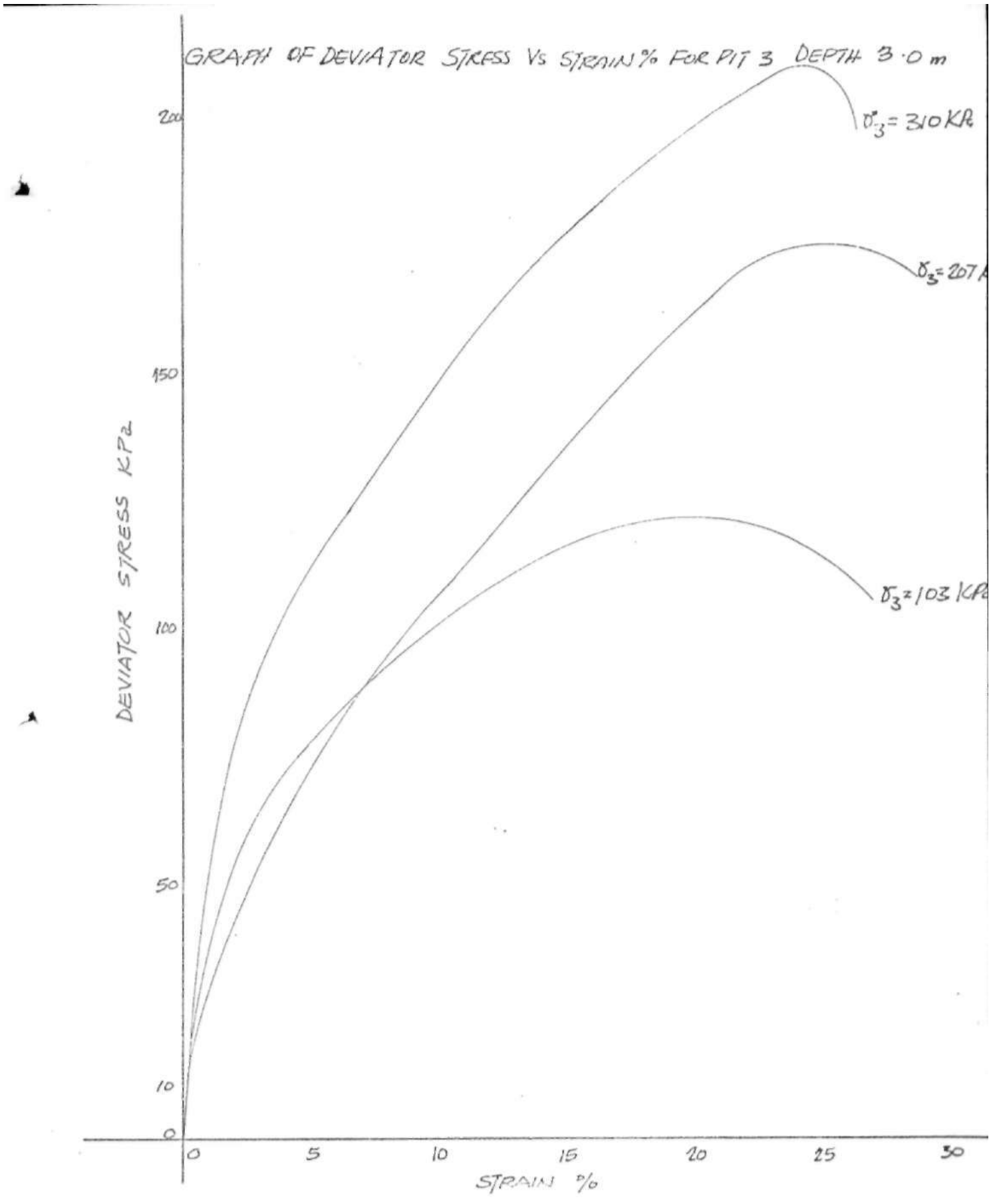


GRAPH OF DEVIATOR STRESS Vs STRAIN % FOR PIT 3 DEPTH 2·0m

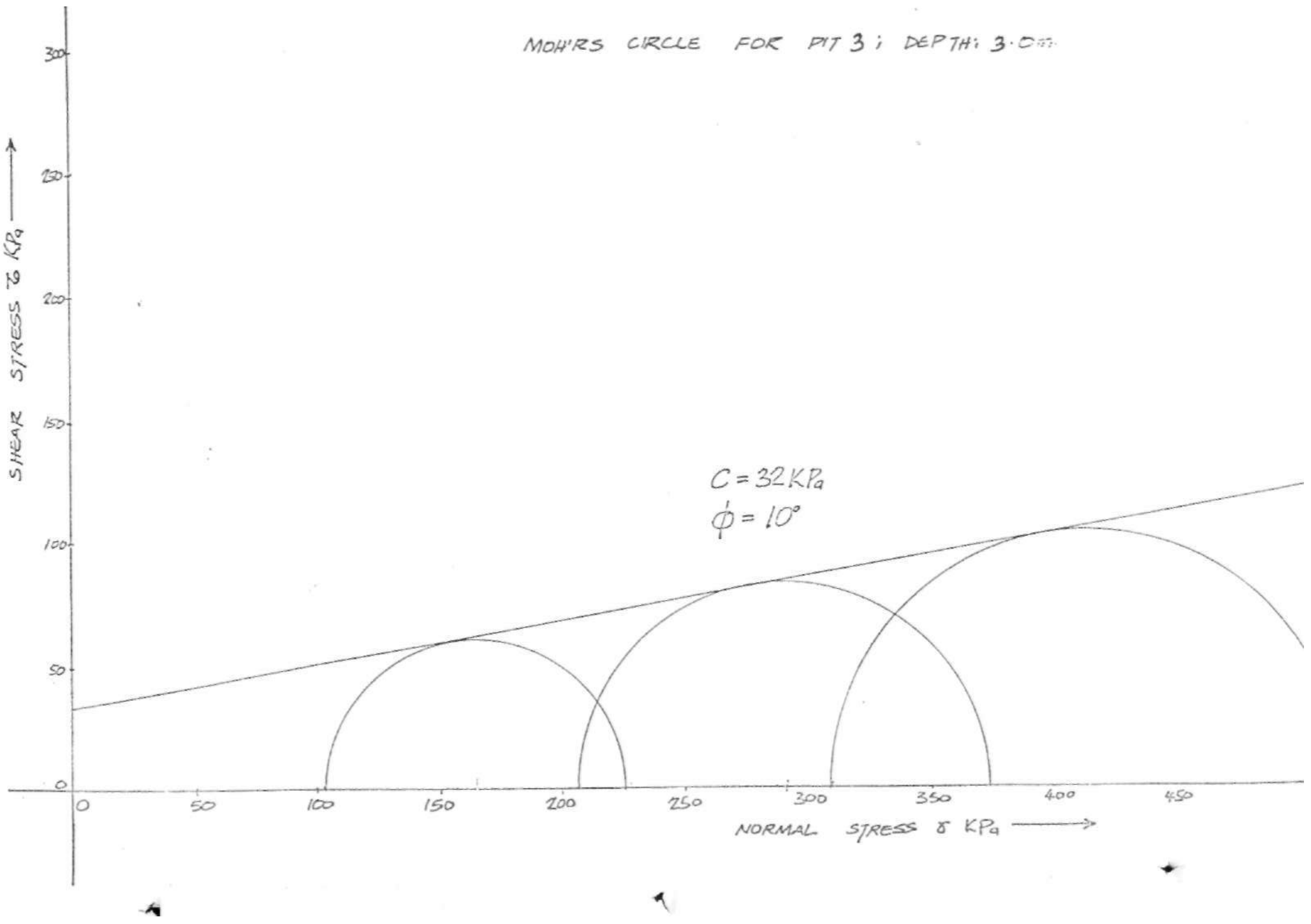


MOHRS CIRCLE FOR PIT 3: DEPTH: 2·0



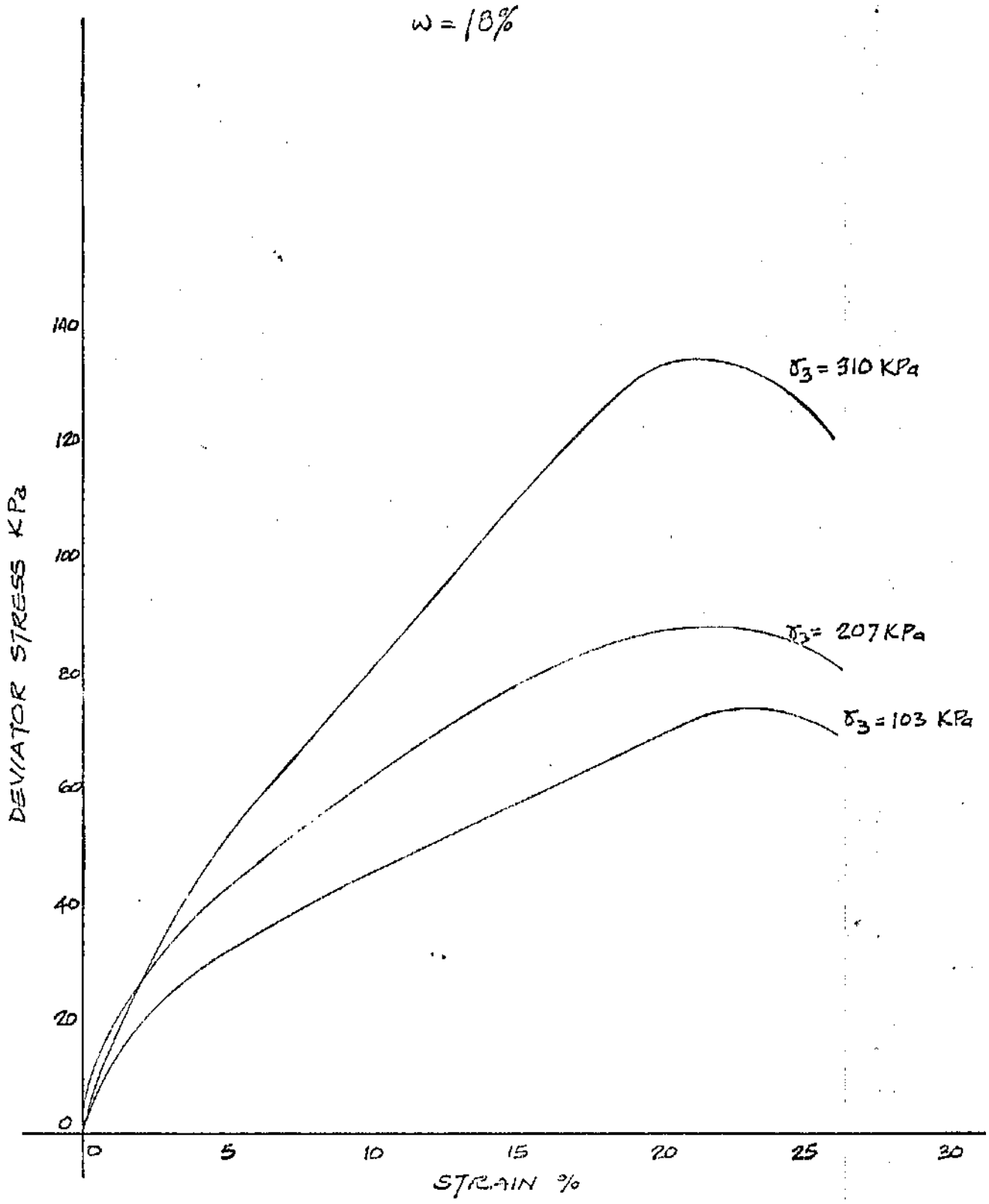

GRAPH OF DEVIATOR STRESS Vs STRAIN % FOR PIT 3 DEPTH 3·0 m
DEVIATOR STRESS KPa
STRAIN %
200
150
100
50
10
0
0
5
10
15
20
25
30
σ_3 = 310 KPa
σ_3 = 207 KPa
σ_3 = 103 KPa

MOH'RS CIRCLE FOR PIT 3 ; DEPTH: 3·0m



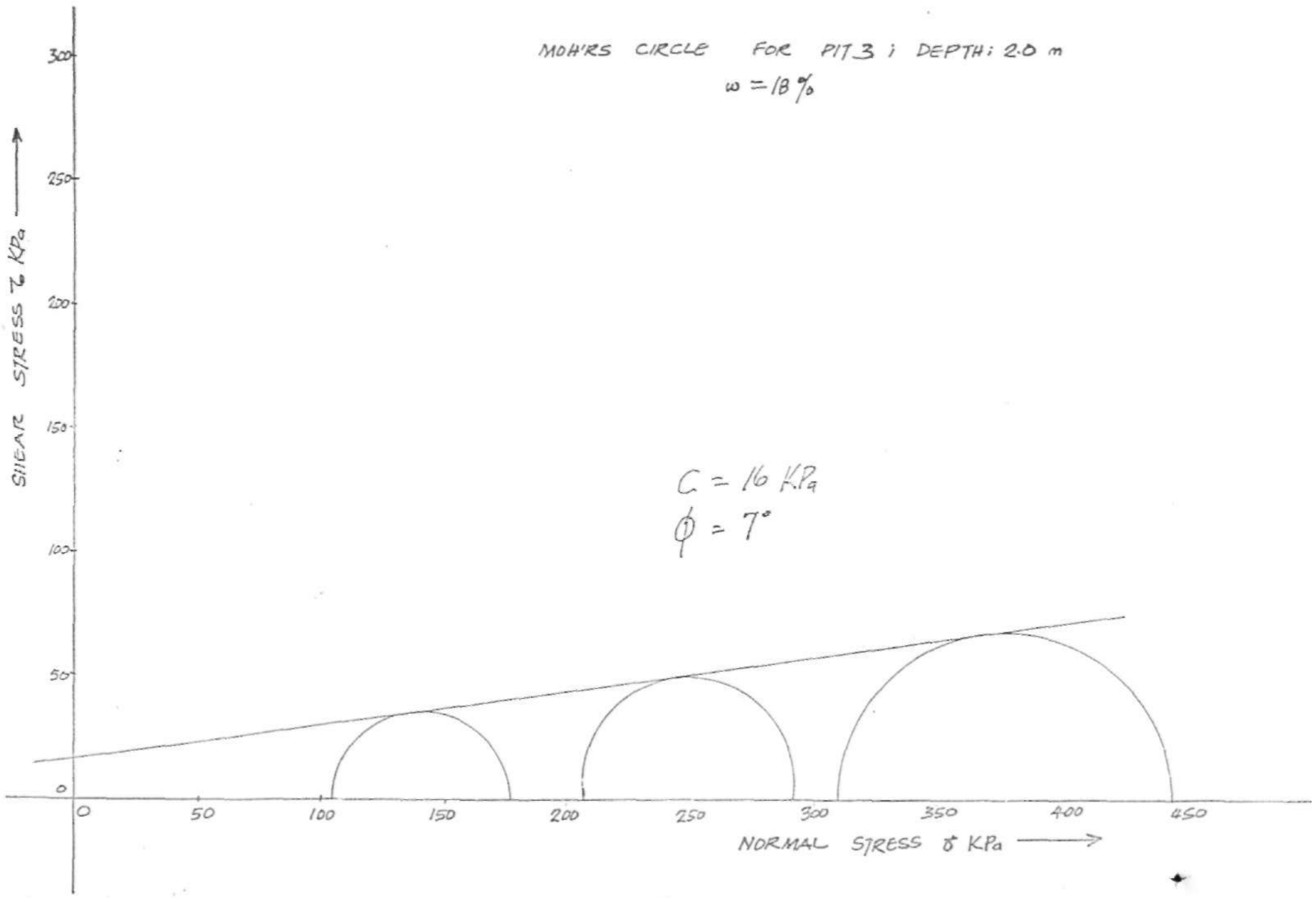
GRAPH OF DEVIATOR STRESS Vs STRAIN % FOR PIT 3 DEPTH: 2.0 m.

$w = 18\%$

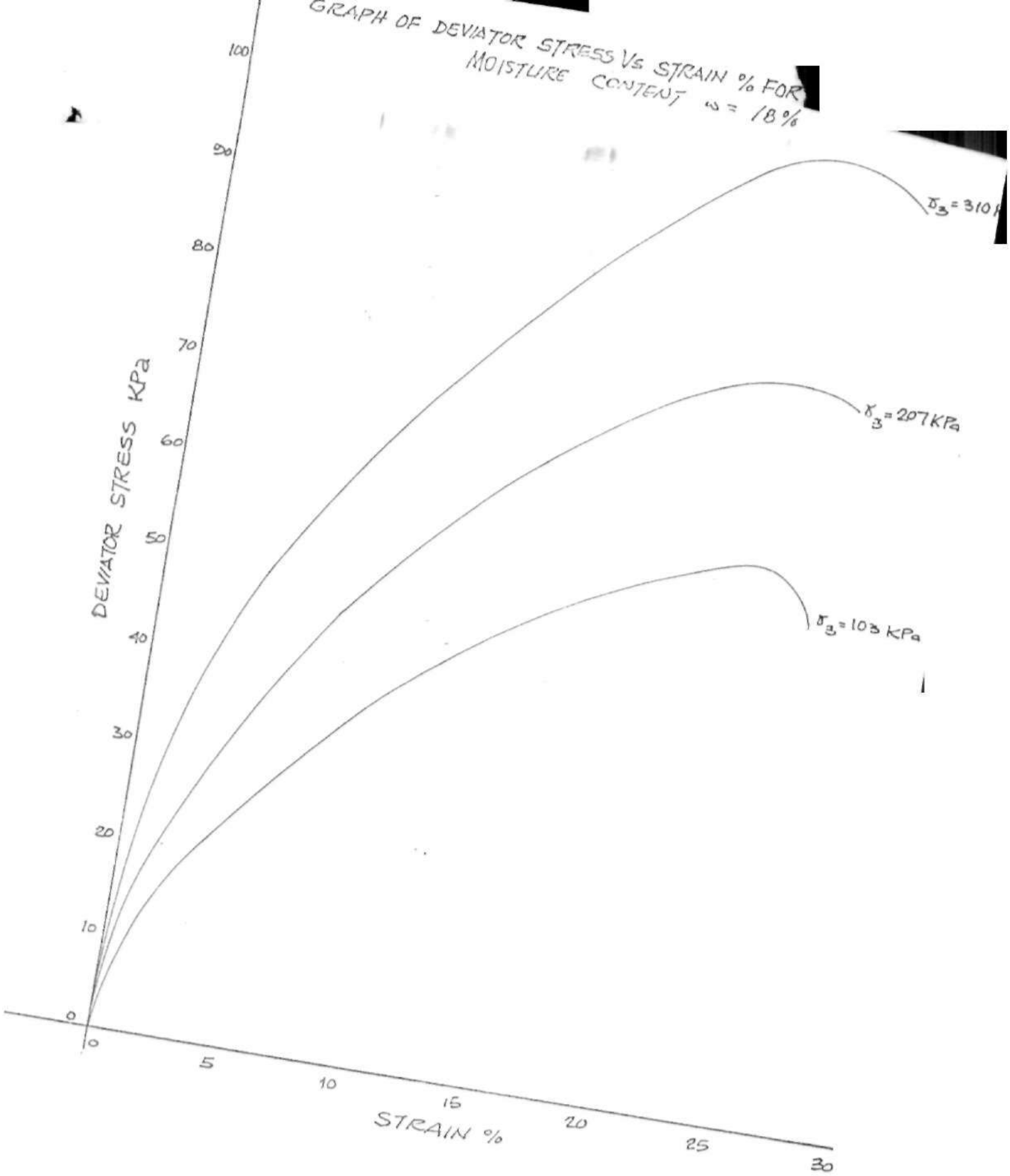


MOH'RS CIRCLE FOR PIT 3 ; DEPTH: 2.0 m

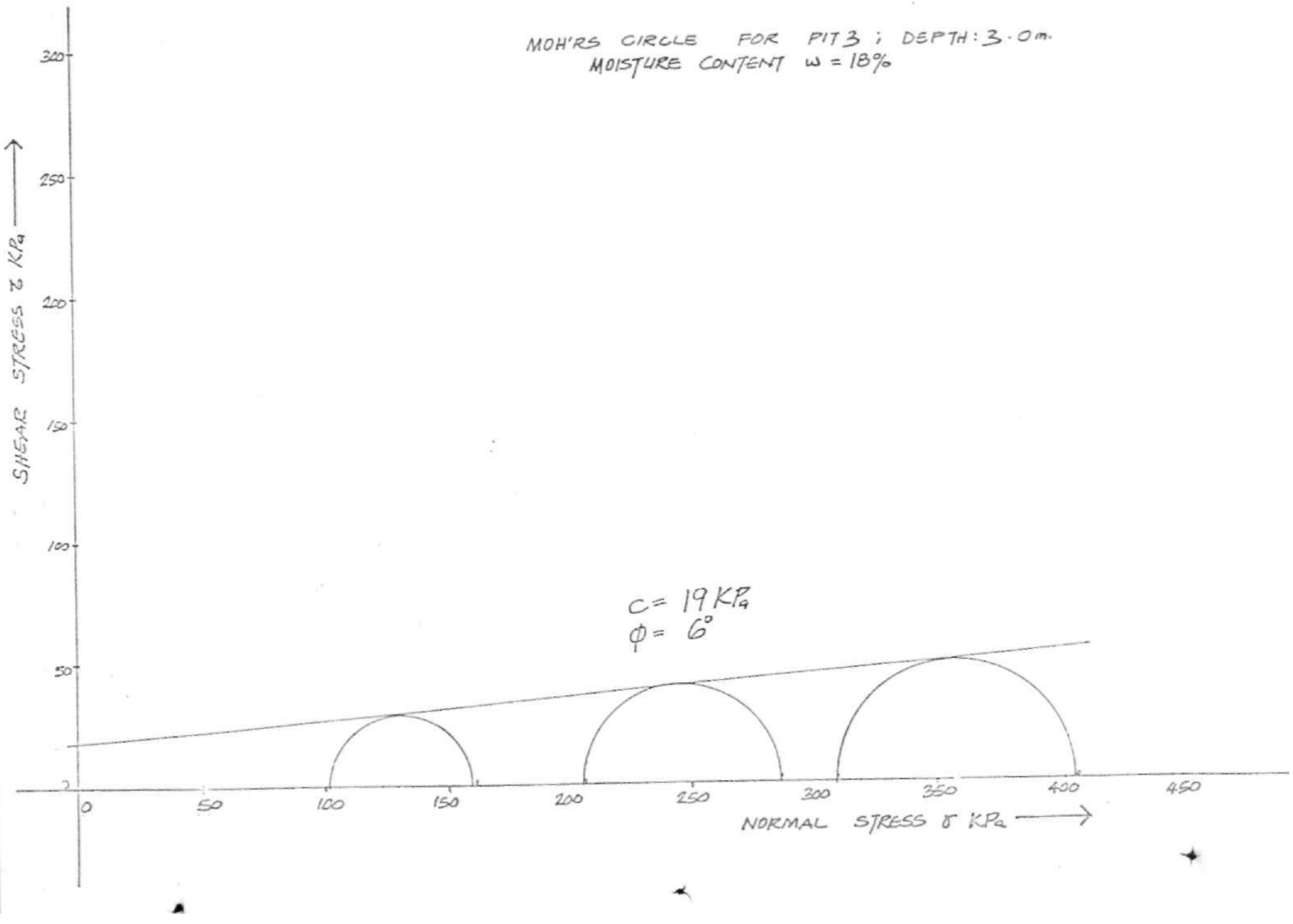
$\omega = 18\%$



GRAPH OF DEVIATOR STRESS Vs STRAIN % FOR
MOISTURE CONTENT ω = 18%



MOH'RS CIRCLE FOR PIT 3 ; DEPTH: 3.0m.
MOISTURE CONTENT $w = 18\%$



APPENDIX 7

Consolidation Test Result on samples at various Moisture contents.

CONSOLIDATION TEST — Data Sheet 14

Project Sokoto Rima River Basin H/Quarters — Job No. M.Sc. Civil 1989

Location SOKOTO — Boring No. PIT 3 — Sample No. 1

Description of Soil Reddish Brown Clayey Gravel — Depth of Sample 2.5m

Tested By E. C. O. — Date of Testing 5-11/6/89

Consolidometer type Fixed Ring — Ring no. CN 1

Multiplication ratio of load device 25.4×10^{-3} mm/div

Ring dimensions: Diam. 7.0cm — Area, A $38.5 cm^2$ — Ht. 2.0cm

Initial ht. of soil, H_i = 2.0cm

Specific gravity of soil, G_s = 2.74

Wt. of ring + specimen at beginning of test = ____

Wt. of ring = 91.0g

Wt. of wet soil, W_i = ____

Computed dry weight of soil, W_s = ____

Oven dry wt. of soil,[a] W_s = 165.1g

Water content determination

Wt. of can + wet soil = 43.0g

Wt. of can + dry soil = 40.0g

Wt. of can = 9.0g

Wt. of water = 3.0g

Wt. of dry soil = 31.0g

Initial water content w_i = 9.67%

Computed Ht. of solids[b] H_s = (2(38.5) − 0.142 x 38.5 + 28.4) / 38.5 = 1.12cm

Initial Ht. of voids, $H_v = H_i - H_s$ = 2.0 − 1.12 = 0.88cm

Initial degree of saturation, $S_i = (W_i - W_s)/(H_v A)$ = ____

Initial void ratio $e_i = H_v/H_s$ = 0.88/1.12 = 0.79

Final Test Data (obtained at end of load testing)

Initial dial reading 595.0

Final dial reading 539.00

Change in sample ht. $56 \times 25.4 \times 10^{-3}$ = 1.42mm

Final ht. of voids, H_f $\frac{28.4}{38.5}$ = 0.74cm

Final void ratio, $e_f = H_f/H_s$ $\frac{0.74}{1.12}$ = 0.66

Final water content determination

Final wet wt. + ring[c] 284.5g

Final dry wt. + ring 256.10g

Oven dry wt. of soil, W_s 165.10g

Final water content, w_f $\frac{28.4}{165.1} \times 100$ 17.2%

Final degree of sat. S ____ %

[a]Obtained from Final Water-Content data.
[b]Use either G_s or final water-content data for $S = 100\%$.
[c]Be sure to include any soil extruded from ring which is in consolidometer.

CONSOLIDATION **TEST** (Tlmo-compresslon dolo)

Piojocl **Sokoto Rima River Basin H/QuartfifS**.....

I OCllh.Mf.l |»|l.lj(M:l SOKOTO

DcscilpiJonolSoli **Readlsh Brown .Clayey Gravel**

T.v.lcrinv E.C.O.

l.'ualin;[Test !),i ta

tn;,(l 2.Q9KPa kl'n

Onln **npplirjtJ 5th June, 1**

JCI

Dorlny No.PJT._3.._ Sample No.

Depth of Sample 2.5m

Dalo of Terming 5th June,198(

Lond -_____^2 B _ kPa

Daloap plied _6th June, 1988

CONSOLIDATION TEST (Time-compression data) Data S

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989.

Location of Project SOKOTO Boring No. PIT 3 Sample No.

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing 7th June, 1988

Loading Test Data

Load 11.56 kPa Load 23.12 kPa

Date applied 7th June, 1988 Date applied 8th June, 1988

Applied by E. C. O. Applied by E. C. O.

Clock time and date	Elapsed time min	*Dial readings × 25x10-3 mm/div Original	Adjusted (mm)	Clock time and date	Elapsed time, min	Dial readings × Original	mm Ad
10:23AM 7/6/88	0.00	588.00	0.18mm	10:30AM 8/6/88	0.00	580.9	0
	0.25	582.00	0.33		0.25	574.50	0
	1.00	582.00	0.33		1.00	574.00	0
	2.25	582.00	0.33		2.25	574.00	0
	4.00	282.00	0.33		4.00	573.80	0
	6.25	581.80	0.34		6.25	573.50	0
	9.00	581.60	0.34		9.00	573.10	0.
	12.25	581.40	0.35		12.25	573.00	0.
	16.00	581.30	0.35		16.00	573.00	0.
	20.25	581.20	0.35		20.25	572.80	0.
	25.00	581.20	0.35		25.00	572.50	0.
	36.00	581.00	0.36		36.00	572.40	0.
	49.00	581.00	0.36		49.00	572.30	0.
	64.00	581.00	0.36		64.00	572.20	0.
	81.00	580.90	0.36		81.00	572.10	0.
	100.00	580.90	0.36mm		100.00	572.00	0.

*Insert name subdivisions 0.01 mm/div. etc.

CONSOLIDATION TEST (Time-compression data) Data S

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No.

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Loading Test Data

Load 46.24 kPa Load 92.48 kPa

Date applied 9th June, 1988 Date applied 10th June, 1988

Applied by E. C. O. Applied by E. C. O.

Clock time and date	Elapsed time min	*Dial readings × 25.4×10⁻³ mm/div Original	Adjusted (mm)	Clock time and date	Elapsed time. min	Dial readings × Original	m A
10:20AM 9/6/88	0.00	572.00	0.58mm	10:00AM 10/6/88	0.00	555.20	
	0.25	558.00	0.94		0.25	540.00	
	1.00	557.50	0.95		1.00	539.20	
	2.25	557.00	0.96		2.25	539.20	
	4.00	556.00	0.99		4.00	539.10	
	6.25	555.80	1.00		6.25	539.00	
	9.00	555.50	1.00		9.00	539.00	
	12.25	555.50	1.00		12.25	539.00	
	16.00	555.40	1.01		16.00	539.00	
	20.25	555.30	1.01		20.25	539.00	
	25.00	555.30	1.01		25.00	539.00	
	36.00	555.20	1.01		36.00	539.00	
	49.00	555.20	1.01		49.00	539.00	
	64.00	555.20	1.01		64.00	539.00	
	81.00	555.20	1.01		81.00	539.00	
	100.00	555.20	1.01mm		100.00	539.00	

*Record major subdivisions 0.01 mm/div, etc

CONSOLIDATION TEST (Time-compression data) Data Sheet 1

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No PIT 3 Sample No. 1

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Loading Test Data

Load 184.96 kPa Load _____ kPa

Date applied 11th June, 1988 Date applied _____

Applied by E. C. O. Applied by _____

Clock time and date	Elapsed time, min	*Dial readings × 25.4×10⁻³ mm/div Original	Adjusted mm	Clock time and date	Elapsed time, min	Dial readings × 25.4x Original	Adjusted
10:10AM 11/6/88	0.00	539.00	1.42mm		0.00		
	0.25	539.00	1.42mm		0.25		
	1.00	539.00	1.42		1.00		
	2.25	539.00	1.42		2.25		
	4.00	539.00	1.42		4.00		
	6.25	539.00	1.42		6.25		
	9.00	539.00	1.42		9.00		
	12.25	539.00	1.42		12.25		
	16.00	539.00	1.42		16.00		
	20.25	539.00	1.42		20.25		
	25.00	539.00	1.42		25.00		
	36.00	539.00	1.42		36.00		
	49.00	539.00	1.42		49.00		
	64.00	539.00	1.42		64.00		
	81.00	539.00	1.42		81.00		
	100.00	539.00	1.42mm		100.00		

*Insert proper subdivisions 0.01 mm/div, etc.

CONSOLIDATION TEST (Computation sheet for e and c_v) **Data Sheet 16**

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 1

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Sample Data

Initial sample vol. V_i = 77cm^3 Dry wt. of soil solids, W_s = 165.1g

Specific gravity of soil, G_s = 2.74 Ht. of solids, H_s = 1.12cm

Initial ht. of voids, H_v = 0.88 Initial void ratio, e_i = 0.79

Load Incr.	Del. dial reading at end of load,[a] ( )	Change in sample ht., ΔH, (mm)	$\epsilon = \Delta H/H_i$ or $\epsilon = \Delta H/H_s$	Inst. void ratio e	Average ht. for load, (C)[c]	H,[d] ( )	Time[e] for 50% consol. t_{50}, min	Coeff. of consol. c_v, ( )/min
0	595.0	0	0	0.79				
2.89KPa	591.30	0.09	0.008	0.782	1.996			
5.78	588.0	0.18	0.016	0.774	1.987			
11.56	580.9	0.38	0.034	0.756	1.972			
23.12	572.0	0.58	0.052	0.738	1.952			
46.24	555.2	1.01	0.09	0.70	1.921			
92.48	539.0	1.42	0.13	0.66	1.879			
184.96	539.0	1.42	0.13	0.66	1.858			

Note: Insert units in column headings as necessary.

[a] Final dial reading of preceding load = Initial dial reading of following load.

[b] Use either strain or void ration computation (or both).

[c] Average ht. for load increment = Ht. at beginning of load $-\frac{1}{2}\Delta H$.

[d] H = length of longest drainage path; for floating ring consolidation = $\frac{1}{2}$ average ht. for the given load increment.

[e] From the dial reading vs log t curves

CONSOLIDATION TEST — Data Sheet 14

Project Sokoto Rima River Basin Dev. H/Qtrs — Job No. M.Sc Civil 1989

Location SOKOTO — Boring No. PIT 3 — Sample No. 2

Description of Soil Reddish Brown Clayey Gravel — Depth of Sample 2.5m

Tested By E. C. O. — Date of Testing 5th June, 1988

Consolidometer type Fixed Ring — Ring no. CN 2

Multiplication ratio of load device 12.7×10^{-3} mm/div

Ring dimensions: Diam. 7.0cm — Area, A $38.5cm^2$ — Ht. 2.0cm

Initial ht. of soil, H_i 2.0cm

Specific gravity of soil, G_s = 2.74

Wt. of ring + specimen at beginning of test =

Wt. of ring = 114.80g

Wt. of wet soil, W_i =

Computed dry weight of soil, W_s' =

Oven dry wt. of soil,[a] W_s = 158.2g

Water content determination

Wt. of can + wet soil = 29.8g

Wt. of can + dry soil = 27.2g

Wt. of can = 8.2g

Wt. of water = 2.6g

Wt. of dry soil = 19.0g

Initial water content w_i = 13.68%

Computed Ht. of solids[b] H_s = $(2\,(38.5) - (0.153 \times 38.5 + 28.7))/38.5 = 1.10$cm

Initial Ht. of voids, $H_v = H_i - H_s$ = 2.0 − 1.10 = 0.9cm

Initial degree of saturation, $S_i = (W_i - W_s)/(H_v A)$ =

Initial void ratio $e_i = H_v/H_s$ = $\frac{0.9}{1.10} = 0.82$

Final Test Data (obtained at end of load testing)

Initial dial reading 799.5

Final dial reading 679.1

Change in sample ht. $120 \times 12.7 \times 10^{-3} = 1.53$mm

Final ht. of voids, H_v $\frac{28.7}{38.5} = 0.75$

Final void ratio, $e_f = H_{vf}/H_s$ $\frac{0.75}{1.1} = 0.68$

Final water content determination

Final wet wt. + ring[c] 298.00g

Final dry wt. + ring 269.3g

Oven dry wt. of soil, W_s 154.5g

Final water content, w_f $\frac{28.7}{154.5} \times = 18.6\%$

Final degree of sat. S ____%

[a]Obtained from Final Water-Content data.
[b]Use either G_s or final water-content data for $S = 100\%$.
[c]Be sure to include any soil extruded from ring which is in consolidometer.

CONSOLIDATION TEST (Time-compression data)

Data S

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 2

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Loading Test Data

Load 2.89 kPa

Date applied 5th June, 1988

Applied by E. C. O.

Load 5.78 kPa

Date applied 6th June, 1988

Applied by E. C. O.

Clock time and date	Elapsed time, min	*Dial readings × 25.4×5×10⁻³ mm/div Original	Adjusted (mm)
	0.00	799.5	0.00
	0.25	793.5	0.08
	1.00	792.5	0.09
	2.25	792.5	0.09
	4.00	790.3	0.09
	6.25	792.3	0.09
	9.00	792.2	0.09
	12.25	792.2	0.09
	16.00	792.2	0.09
	20.25	792.2	0.09
	25.00	792.1	0.09
	36.00	792.1	0.09
	49.00	792.1	0.09
	64.00	792.1	0.09
	81.00	792.1	0.09
	100.00	792.1	0.09

Clock time and date	Elapsed time, min	Dial readings × 1 mm Original	Ad
	0.00	792.10	0
	0.25	790.20	0
	1.00	790.10	0
	2.25	790.00	0
	4.00	790.00	0
	6.25	790.00	0
	9.00	790.00	0
	12.25	790.00	0
	16.00	789.00	0
	20.25	789.00	0
	25.00	788.00	0
	36.00	788.00	0
	49.00	788.00	0
	64.00	788.00	0
	81.00	788.00	0
	100.00	788.00	0

*Insert gauge subdivisions 0.01 mm/div. etc

CONSOLIDATION TEST (Time-compression data) Data Sh

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 2

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Loading Test Data

Load 11.56 kPa Load 23.12 kPa

Date applied 7th June, 1988 Date applied 8th June, 1988

Applied by E. C. O. Applied by E. C. O.

Clock time and date	Elapsed time, min	*Dial readings 12.7×10^{-3} mm/div Original	Adjusted	Clock time and date	Elapsed time, min	Dial readings × 12 Original	mm/ Adjus
	0.00	788.00	0.15		0.00	785.80	0.1
	0.25	786.5	0.17		0.25	775.00	0.3
	1.00	786.40	0.17		1.00	773.00	0.3
	2.25	786.20	0.17		2.25	772.80	0.3
	4.00	786.20	0.17		4.00	772.60	0.3
	6.25	786.10	0.17		6.25	772.60	0.3
	9.00	786.00	0.17		9.00	772.60	0.3
	12.25	765.90	0.17		12.25	772.50	0.3
	16.00	785.80	0.17		16.00	772.20	0.3
	20.25	785.80	0.17		20.25	772.20	0.3
	25.00	785.80	0.17		25.00	772.20	0.3
	36.00	785.80	0.17		36.00	772.20	0.3
	49.00	785.80	0.17		49.00	772.00	0.3
	64.00	785.80	0.17		64.00	772.00	0.3
	81.00	785.80	0.17		81.00	772.00	0.3
	100.00	785.80	0.17		100.00	772.00	0.3

*Insert gage subdivisions 0.01 mm/div. etc. . .

CONSOLIDATION TEST (Time-compression data) Data Sh

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 2

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5

Tested By E. C. O. Date of Testing

Loading Test Data

Load 46.24 kPa Load 92.48 kPa

Date applied 9th June, 1988 Date applied 10th June, 1988

Applied by E. C. O. Applied by E. C. O.

Clock time and date	Elapsed time, min	*Dial readings ×		Clock time and date	Elapsed time, min	Dial readings ×	
		Original	Adjusted			Original	Adj
	0.00	772.0	0.35		0.00	717.20	1.
	0.25	718.50	1.03		0.25	689.00	1
	1.00	718.00	1.03		1.00	688.60	1
	2.25	717.30	1.04		2.25	688.00	1
	4.00	717.30	1.04		4.00	687.80	1
	6.25	717.30	1.04		6.25	687.60	1
	9.00	717.20	1.04		9.00	687.40	1
	12.25	717.20	1.05		12.25	687.40	1
	16.00	717.20	1.05		16.00	687.40	1
	20.25	717.20	1.05		20.25	687.40	1
	25.00	717.20	1.05		25.00	687.40	1
	36.00	717.20	1.05		36.00	687.40	1
	49.00	717.20	1.05		49.00	687.40	1
	64.00	717.20	1.05		64.00	687.40	1
	81.00	717.20	1.05		81.00	687.40	1
	100.00	717.20	1.05		100.00	687.40	1

*Insert gage subdivisions 0.01 mm/div, etc

CONSOLIDATION TEST (Time–compression data) Data S

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 2

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By Date of Testing

Loading Test Data

Load 184.96 kPa Load kPa

Date applied Date applied

Applied by Applied by

Clock time and date	Elapsed time, min	*Dial readings ×		Clock time and date	Elapsed time, min	Dial readings ×	
		Original	Adjusted			Original	Ad
	0.00	687.40	1.42		0.00		
	0.25	680.50	1.51		0.25		
	1.00	680.40	1.51		1.00		
	2.25	680.30	1.51		2.25		
	4.00	680.20	1.51		4.00		
	6.25	680.20	1.51		6.25		
	9.00	680.00	1.52		9.00		
	12.25	679.40	1.52		12.25		
	16.00	679.30	1.52		16.00		
	20.25	679.20	1.53		20.25		
	25.00	679.20	1.53		25.00		
	36.00	679.20	1.53		36.00		
	49.00	679.20	1.53		49.00		
	64.00	679.20	1.53		64.00		
	81.00	679.10	1.53		81.00		
	100.00	679.10	1.53		100.00		

*Insert gage subdivisions 0.01 mm/div, etc.

CONSOLIDATION TEST (Computation sheet for e and c_v) Data Sheet 16

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 2

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing 5th June, 1988

Sample Data

Initial sample vol. V_s = 77cm^3 Dry wt. of soil solids, W_s = 114.8g

Specific gravity of soil, G_s = 2.74 Ht. of solids, H_s = 1.10cm

Initial ht. of voids, H_v = 0.9 Initial void ratio, e_i = 0.82

Load Incr.	Def. dial reading at end of load,[a] ( )	Change in sample ht., ΔH, (mm)	$\epsilon = \Delta H/H_i$, or $e = \Delta H/H_s$	Inst. void ratio e	Average[c] ht. for load, ( )	H,[d] ( )	Time[e] for 50% consol. t_{50}, min	Coeff. of consol. c_v, ( )/min
0	799.5			0.82		0.00		
2.89KPa	792.1	0.09	0.008	0.812	1.996			
5.78	788.0	0.15	0.014	0.806	1.988			
11.56	785.80	0.17	9.015	0.805	1.984			
23.12	772.00	0.35	0.032	0.788	1.974			
46.24	717.20	1.05	0.095	0.725	1.930			
92.48	687.40	1.42	0.129	0.691	1.877			
184.96	679.10	1.53	0.139	0.681	1.853			

Note: Insert units in column headings as necessary.

[a]Final dial reading of preceding load = Initial dial reading of following load.

[b]Use either strain or void ration computation (or both).

[c]Average ht. for load increment = Ht. at beginning of load $-\frac{1}{2}\Delta H$.

[d]H = length of longest drainage path; for floating ring consolidation = $\frac{1}{2}$ average ht. for the given load increment.

[e]From the dial reading vs log t curves.

CONSOLIDATION TEST — Data Sheet 14

Project Sokoto Rima River Basin H/Qtrs — Job No. M.Sc Civil 1989

Location SOKOTO — Boring No. PIT 3 — Sample No. 3

Description of Soil Reddish Brown Clayey Gravel — Depth of Sample 2.5m

Tested By E. C. O. — Date of Testing 22nd August, 1988

Consolidometer type Fixed Ring — Ring no. CN 2

Multiplication ratio of load device 12.7×10^{-3} mm/div

Ring dimensions: Diam. 7.0cm — Area, A $38.5cm^2$ — Ht. 2.0cm

Initial ht. of soil, H_1 2.0cm

Specific gravity of soil, G_s = 2.74

Wt. of ring + specimen at beginning of test =

Wt. of ring = 91.0g

Wt. of wet soil, W =

Computed dry weight of soil, W_s =

Oven dry wt. of soil,[c] W_s = 160.2g

Water content determination

Wt. of can + wet soil = 44.9g

Wt. of can + dry soil = 40/3g

Wt. of can = 9.0g

Wt. of water = 4.6g

Wt. of dry soil = 31.3g

Initial water content w_1 = 15.89%

Computed Ht. of solids[b] H_s = (2 (38.5) - (0.16 x 38.5 + 31.2)) / 38.5 = 1.03cm

Initial Ht. of voids, $H_v = H_1 - H_s$ = 2.0 - 1.03 = 0.97cm

Initial degree of saturation, $S_1 = (W_1 - W_s)/(H_v A)$ =

Initial void ratio $e_1 = H_v/H_s$ = $\frac{0.97}{1.03}$ = 0.94

Final Test Data (obtained at end of load testing)

Initial dial reading 221.0

Final dial reading 91.4

Change in sample ht. $129 \times 12.7 \times 10^{-3}$ = 1.65

Final ht. of voids, H_{vf} $\frac{31.2}{38.5}$ = 0.81

Final void ratio, $e_f = H_{vf}/H_s$ $\frac{0.81}{1.03}$ = 0.78

Final water content determination

Final wet wt. + ring[c] 282.4g

Final dry wt. + ring 251.2g

Oven dry wt. of soil, W_s 160.2g

Final water content, w_f $\frac{31.2}{160.2} \times 100$ = 19.5%

Final degree of sat. S %

[a] Obtained from Final Water-Content data.
[b] Use either G_s or final water-content data for S = 100%.
[c] Be sure to include any soil extruded from ring which is in consolidometer.

CONSOLIDATION TEST (Time-compression data) Data Sheet 15

Project Sokoto Rima River Basin H/Qtrs Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Loading Test Data

Load 2.89 kPa Load 5.78 kPa

Date applied 22nd August 1988 Date applied 23rd August, 1988

Applied by E. C. O. Applied by E. C. O.

Clock time and date	Elapsed time, min	*Dial readings × 12.7x10^{-3} mm/div Original	Adjusted (mm)	Clock time and date	Elapsed time, min	Dial readings × 12.7x10^{-3} mm, Original	Adjusted (mm)
	0.00	221.0	0.00		0.00	213.20	0.10
	0.25	220.0	0.01		0.25	203.20	0.23
	1.00	218.0	0.04		1.00	203.00	0.23
	2.25	217.8	0.04		2.25	203.00	0.23
	4.00	217.6	0.04		4.00	203.00	0.23
	6.25	217.6	0.04		6.25	203.00	0.23
	9.00	217.6	0.04		9.00	203.00	0.23
	12.25	217.4	0.05		12.25	203.00	0.23
	16.00	216.00	0.06		16.00	203.00	0.23
	20.25	215.80	0.07		20.25	203.00	0.23
	25.00	215.60	0.07		25.00	203.00	0.23
	36.00	214.00	0.09		36.00	203.00	0.23
	49.00	213.80	0.09		49.00	203.00	0.23
	64.00	213.60	0.09		64.00	203.00	0.23
	81.00	213.20	0.10		81.00	203.00	0.23
	100.00	213.20	0.10		100.00	202.80	0.23

*Insert gage subdivisions 0.01 mm/div, etc.

CONSOLIDATION TEST (Time-compression data)

Data Sheet 15

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E.C.O Date of Testing

Loading Test Data

Load 11.65 kPa

Date applied 24th August, 1988

Applied by

Load 23.12 kPa

Date applied 25th August, 1988

Applied by

Clock time and date	Elapsed time, min	*Dial readings ×	
		Original	Adjusted
	0.00	202.8	0.23
	0.25	198.80	0.28
	1.00	198.60	0.28
	2.25	198.40	0.29
	4.00	198.20	0.29
	6.25	198.00	0.29
	9.00	197.80	0.29
	12.25	197.80	0.29
	16.00	197.80	0.29
	20.25	197.80	0.29
	25.00	197.80	0.29
	36.00	197.70	0.30
	49.00	197.70	0.30
	64.00	197.70	0.30
	81.00	197.60	0.30
	100.00	197.60	0.30

Clock time and date	Elapsed time, min	Dial readings ×	
		Original	Adjusted
	0.00	197.60	0.30
	0.25	192.00	0.37
	1.00	191.80	0.37
	2.25	191.60	0.37
	4.00	191.40	0.37
	6.25	191.20	0.37
	9.00	191.00	0.38
	12.25	190.80	0.38
	16.00	190.60	0.38
	20.25	190.40	0.38
	25.00	190.20	0.39
	36.00	190.10	0.39
	49.00	190.00	0.39
	64.00	189.80	0.40
	81.00	189.70	0.40
	100.00	189.70	0.40

*Insert gage subdivisions 0.01 mm/div. etc.

CONSOLIDATION TEST (Time-compression data) Data Sheet 15

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Loading Test Data

Load 46.24 kPa

Date applied 26th August, 1988

Applied by E. C. O.

Clock time and date	Elapsed time, min	*Dial readings × ____ Original	Adjusted
	0.00	189.70	0.40
	0.25	141.00	1.02
	1.00	139.00	1.04
	2.25	138.80	1.04
	4.00	138.00	1.04
	6.25	136.00	1.08
	9.00	134.00	1.10
	12.25	132.00	1.13
	16.00	131.80	1.13
	20.25	131.60	1.13
	25.00	131.40	1.14
	36.00	131.00	1.14
	49.00	130.80	1.15
	64.00	130.60	1.15
	81.00	130.40	1.15
	100.00	130.00	1.16

Load 92.48 kPa

Date applied 27th August, 1988

Applied by E. C. O

Clock time and date	Elapsed time, min	Dial readings × ____ Original	Adjusted
	0.00	130.00	1.16
	0.25	116.00	1.33
	1.00	114.80	1.35
	2.25	114.40	1.35
	4.00	114.20	1.36
	6.25	114.00	1.36
	9.00	113.20	1.37
	12.25	113.00	1.37
	16.00	112.80	1.37
	20.25	112.60	1.38
	25.00	112.40	1.38
	36.00	112.20	1.38
	49.00	112.00	1.38
	64.00	111.80	1.38
	81.00	111.60	1.39
	100.00	111.40	1.40

*Insert gage subdivisions 0.01 mm/div. etc.

CONSOLIDATION TEST (Time–compression data) Data Sheet 15

Project Sokoto Rima River Basin H/Q Job No. M.Sc Civil 1989

Location of Project SOKOTO Boring No. PIT 3 Sample No. 3

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing

Loading Test Data

Load 184.96 kPa

Date applied 28th August, 1988

Applied by E. C. O.

Clock time and date	Elapsed time, min	*Dial readings × ____ Original	Adjusted
	0.00	111.40	1.40
	0.25	97.00	1.57
	1.00	95.80	1.59
	2.25	95.60	1.59
	4.00	95.40	1.59
	6.25	95.00	1.60
	9.00	93.80	1.62
	12.25	93.60	1.62
	16.00	93.00	1.63
	20.25	92.20	1.64
	25.00	92.00	1.64
	36.00	92.00	1.64
	49.00	91.80	1.64
	64.00	91.50	1.65
	81.00	91.50	1.65
	100.00	91.40	1.65

Load ____ kPa

Date applied ____

Applied by ____

Clock time and date	Elapsed time, min	Dial readings × ____ Original	Adjusted
	0.00		
	0.25		
	1.00		
	2.25		
	4.00		
	6.25		
	9.00		
	12.25		
	16.00		
	20.25		
	25.00		
	36.00		
	49.00		
	64.00		
	81.00		
	100.00		

*Insert gage subdivisions 0.01 mm/div. etc

CONSOLIDATION TEST (Computation sheet for [illegible])

Data Sheet 16

Project Sokoto Rima River Basin Dev. Authority Job No. M.Sc. Civil 1988

Location of Project SOKOTO Boring No PIT 3 Sample No. 3

Description of Soil Reddish Brown Clayey Gravel Depth of Sample 2.5m

Tested By E. C. O. Date of Testing 22nd August, 1988

Sample Data

Initial sample vol. V_i = 77cm^3 Dry wt. of soil solids, W_s = 160.2g

Specific gravity of soil, G_s = 2.74 Ht. of solids, H_s = 1.03cm

Initial ht. of voids, H_v = 0.97 Initial void ratio, e_i = 0.94

Load Incr.	Def. dial reading at end of load,* ( )	Change in sample ht., ΔH, (mm)	$\epsilon = \Delta H/H_i$, or $\epsilon = \Delta H/H_s$	Inst. void ratio e	Average ht. for load, ( )	H,[d] ( )	Time[e] for 50% consol. t_{50}, min	Coeff. of consol. c_v, ( )/min
0	221.0	0		0.94				
2.8KPa	213.20	0.10	0.01	0.93	1.995			
5.78	202.80	0.23	0.02	0.92	1.984			
11.56	197.60	0.30	0.03	0.91	1.974			
23.12	189.70	0.40	0.04	0.90	1.965			
46.24	130.00	1.16	0.11	0.83	1.922			
92.48	111.40	1.40	0.14	0.80	1.872			
184.96	91.40	1.65	0.16	0.78	1.848			

Note: Insert units in column headings as necessary.

[a]Final dial reading of preceding load = Initial dial reading of following load.

[b]Use either strain or void ration computation (or both).

[c]Average ht. for load increment = Ht. at beginning of load − ΔH.

[d]H = length of longest drainage path for floating ring consolidation = ½ average ht. for the given load increment.

[e]From the dial reading vs log t curves.

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