MAURITIUS RESEARCH COUNCIL

INNOVATION FOR TECHNOLOGY

A PRACTICAL GUIDE TO GEOTECHNICAL SITE CHARACTERIZATION FOR MAURITIUS

Final Report

MAURITIUS RESEARCH COUNCIL

Address:
La Maison de Carné
Royal Road
Rose Hill
Mauritius

Telephone: (230) 465 1235
Fax: (230) 465 1239
e-mail: mrc@intnet.mu
Website: www.mrc.org.mu

This report is based on work supported by the Mauritius Research Council under award number MRC/RUN-9606. Any opinions, findings, recommendations and conclusions expressed herein are the author's and do not necessarily reflect those of the Council.

UNIVERSITY OF MAURITIUS

FACULTY OF ENGINEERING

A PRACTICAL GUIDE
TO GEOTECHNICAL SITE CHARACTERIZATION
FOR MAURITIUS

A study supported by a Mauritius Research Council Grant

Final Report

by

Dr. A. C. W. Chan Chim Yuk

May 2003

EXECUTIVE SUMMARY

Civil engineering consists of the conception, analysis, design, construction, operation and maintenance of a diversity of structures, all of which are built on, in or with soils or rocks. The engineering behaviour of the soils and rocks at the location of any construction project has a major influence on the economy and safety of the works. The increase in the development of infrastructural works in Mauritius during the past two decades was accompanied by a corresponding increase in site investigation activities and more importantly, by the need to determine the required parameters for geotechnical engineering design. These site investigation works have enabled us at the University of Mauritius to acquire extensive geological and geotechnical data. The main aim of the present project is to compile these data into an electronic format and produce simple correlations which can be used as guidelines by civil engineers, as part of a desk study and/or in preliminary design.

This project has as one of its main achievements, the implementation of an electronic data storage system, using a computer software package known as SID. A total of about 400 borehole logs have been archived with more than 200 of them being located in Port-Louis. Using the SID software, it is possible to extract the relevant geological information from the database to draw up geological profiles for a specific location. The geological profiles provide important information for the design of foundations. It is the objective of the author as part of a future project to make the geological database available on the internet for the benefit of civil engineers.

Another output of the project is the production of simple correlations for the basaltic rocks and residual soils of Mauritius; these correlations are meant to help engineers in the determination of parameters (such as strength and compressibility) for geotechnical design. A limited amount of data on corals (considered as a weak rock) has also been included. In the case of local soils, only data on soils formed from the decomposition of basalt have been compiled and analysed. Engineering parameters for both undisturbed and compacted soils have been proposed. Available data on coral sands and estuarine deposits will be analysed as part of a future project.

The present work can be extended to Rodrigues, especially Port Mathurin, for which geological and geotechnical data are also available.

ACKNOWLEDGEMENTS

First of all, I would like to express my sincere gratitude to the Mauritius Research Council for providing funds to undertake this project. I am also indebted to the following:

- students who carried out their final year undergraduate projects under my supervision and whose data have been used in the preparation of this report;
- local civil engineering consulting firms, especially, Servansingh Jadav & Partners, Gibb (Mtius) Ltd., Luxconsult (Mtius) Ltd. who, during the past fifteen years, gave me the opportunity to participate as geotechnical expert in some of their projects and also convinced me of the necessity to produce this guide;
- the drilling contractors, Sotramon Ltee and Aqualia Ltd., with whom I have worked in close collaboration in many site investigation works;
- my colleagues, Mr. U. Armoogum and Dr. M. Nowbuth, with whom I had frequent discussions on various aspects of this project;
- technical staff of the Civil Engineering Laboratory who compiled some of the data presented in this report;
- last but not least, the University of Mauritius for providing testing and other facilities.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Background

In Mauritius, the characteristics of the ground are studied in three main fields, namely:

- hydrogeology
- agriculture
- geotechnical engineering design and construction.

The one common area which underlies these studies is the geology of the island. The hydrogeology of Mauritius has been studied extensively by the Water Resources Unit (WRU) of the Ministry of Public Utilities and a hydrogeological map was published in 1999. The soils of the Mauritius have been researched by the Mauritius Sugar Industry Research Institute (MSIRI); an agricultural soil classification map and a pedological map were published in 1962 and 1983 respectively. While the hydrogeological and pedological studies are well documented, the results of geotechnical investigations can make an important contribution to the existing knowledge on the geology of Mauritius and, if compiled in a user-friendly document, will be helpful to civil engineers. In addition, geotechnical engineering is the area of civil engineering, which requires the most input of local experience.

The increase in the development of infrastructural works on the island during the past two decades was accompanied by a corresponding increase in site investigation activities and more importantly, by the need to determine the required parameters for geotechnical engineering design. These site investigation works have enabled us at the University of Mauritius (UoM) to acquire a large number of rock and soil samples for experimental investigations with the objective of setting up a database that will assist engineers in their design. The author has participated as a consultant in more than 100 civil engineering projects over the past 15 years and has compiled extensive geological and geotechnical data. Field and laboratory data were obtained from commercial testing, which forms part of the activities of the UoM geotechnical testing laboratory, and also from student research projects. The present project has as achievement the implementation of an electronic storage system for geological data in the form of descriptive logs. These data are complemented by the usual engineering parameters, used in geotechnical design, which are not yet integrated in the present electronic storage system. Future work will provide for complete integration of all data which may be accessed through the internet. The electronic storage of geological and geotechnical data are needed so as to facilitate access and interchange of information for optimization of future investigation works.

1.2 Objectives

The aim of the project is to provide guidelines, which will enable civil engineers to make preliminary assessments of their sites as part of their desk study and to identify probable geotechnical design and construction problems. The guidelines are not meant to replace information that should be obtained from the relevant ground investigation on a specific site.

The objectives of the project were to:

1. convert all available geological logs from hard copies into an electronic format.
2. create a computerized database system for the management of the geological data.
3. classify the rocks and soils of Mauritius for geotechnical purposes.
4. develop guidelines for estimating the engineering properties of the rocks and soils of Mauritius.

1.3 Methodology

The study was carried as follows:

1. Description and classification of rocks and soils for engineering purposes

The aim of this activity, which is described in Chapter 2, was to adopt an appropriate scheme which standardizes the description and classification of the rocks and soils relevant to Mauritius. The scheme will be used to compile all geological logs for the purpose of the present project. The British Standards BS 1377 and BS 5930 have been used as base documents and where appropriate, modifications have been proposed to suit the local context.

2. Implementation of a geological data management system

Chapter 3 describes a computerized system used to create a database of geological logs. The system simplifies the input of future data when they become available and facilitates the retrieval of available information.

3. Indexing the engineering properties of local rocks

Chapter 4 presents simple indices which may be used to estimate the strength and deformability of local rocks. These indices will enable an engineer to carry out preliminary design on the basis of available representative data.

4. Engineering behaviour of local soils

Chapter 5 presents simple correlations between mechanical properties and physical indices. These correlations may be used to predict the behaviour of local soils.

CHAPTER 2

DESCRIPTION AND CLASSIFICATION OF ROCKS AND SOILS FOR ENGINEERING PURPOSES

2.1 Introduction

The geology of Mauritius is well documented from the studies of Simpson (1950), Willaime (1984) and Giorgi et al (1999), among others. These studies, because of their very general nature, do not provide the relevant details which are required by a civil engineer when he has to study the local geology of a particular site. The report of Simpson gives an account of the geogical history of Mauritius, that of Willaime concerns the pedology of soils at a depth of not more than 1 m whereas the work of Giorgi is concerned with the hydrogeological characteristics of aquifers.

There are two types of rock in Mauritius, namely, igneous and sedimentary. The igneous basaltic rocks can be divided into three main stratigraphic series, namely:

Ancient or old
Intermediate
Recent or young.

The sedimentary rocks consist mainly of coral formations.

The volcanic rocks have been subjected to varying degrees of weathering which were controlled by intensity of fracturing, porosity and exposure to weathering agents. Most of the weathering took place in-situ producing residual soils but in some cases, the weathered materials have been eroded, transported and deposited elsewhere to form alluvial, colluvial and estuarine soils. Figure 2.1 gives an overview of soil formation in Mauritius. Being given that the local basaltic rocks do not vary much in terms of mineralogical composition, it has been found that weathering and porosity are the main factors affecting the engineering properties of the intact rock. On the other hand, the characteristics of the intact rock and the fractures control the mass properties of the rock.

2.2 Description scheme for rocks

According to BS 5930, both material characteristics and mass characteristics should be observed when describing a rock profile. The essential characteristics in the description of unjointed rock materials are:

- Strength.
- Colour – Munsell chart is normally used.
- Structure – beddings and flow planes.
- Texture – arrangement of individual grains.
- Grain size.
- Rock name.

An assessment of the porosity of the rock material, the size of vesicles and the degree of filling of the vesicles, including amygdales are required in the description of local basalt.

The rock mass characteristics are:

- State of weathering.
- Discontinuities – joint orientations and aperture.
- Fracture state – spacing of discontinuities.

2.2.1 Weathering classification scheme

Weathering of rocks occurs when there are processes of physical disintegration, chemical decomposition and biological activity. It weakens the rock material and increases the structural weaknesses. Weathering causes a decrease in density and strength and increases the compressibility of a rock. The rate of weathering depends on the mineralogical composition, texture, porosity and strength of the rock on the one hand and the characteristics of the discontinuities within the rock mass on the other. In Mauritius, the weathering process is due mainly to chemical decomposition. Willaime (1984) found that climatic conditions in Mauritius favoured the formation of ferrallitic soils. During the weathering process, all primary minerals are weathered by hydrolysis in neutral conditions and much of the silica and bases are removed in solution. The remaining silica combines with alumina to form the clay mineral kaolinite. When there is an excess of alumina, gibbsite minerals are formed.

For engineering purposes, a number of weathering classification schemes are described in the literature but they are mainly based on granitic rocks. For the present project, certain elements in the Ng et al (2001) classification scheme which is based on physical appearance have been substituted in the BS5930 scheme to give the scheme summarised in Table 2.1. However, research work is ongoing at the University of Mauritius with the aim to produce an appropriate weathering index scheme for local basalts.

Fresh, slightly and moderately weathered basalt (grades I, II and III) will be considered to have rock properties whereas the more weathered materials (grades IV, V and VI), also known as saprolite, will be considered to have soil like behaviour. It is interesting to note that, in South Africa, rock is defined as an *"igneous, metamorphic or sedimentary material with an unconfined compressive strength of the intact or unjointed material in excess of 1 MPa"* (Brink and Bruin, 1990).

Descriptive term	Weathering grade	Recognition factors for rock material	Recognition factors for rock mass
Fresh	I	No visible signs of decomposition/ discolouration of rock material.	Fresh rock material except for staining on major discontinuities.
Slightly weathered	II	Partial discoloration or staining of rock material. Original colour and texture of parent rock is recognizable. No major change in strength properties from fresh state.	Some to all of the rock mass is discoloured by slight weathering.
Moderately weathered	III	Completely stained throughout rock material. The parent rock original colour is not recognizable. Significant weakening. Cannot be broken by hand.	Less than half of rock mass is weathered to moderate and above grades. Fresh or slightly weathered rock exists either as a discontinuous framework or as corestones.
Highly weathered	IV	Can be broken by hand into smaller pieces. Does not disintegrate when immersed into water. Completely discoloured when compared to parent rock.	More than half of rock mass is weathered to moderate and above grades. Fresh or slightly weathered rock exists either as a discontinuous framework or as corestones.
Completely weathered	V	Completely discoloured when compared to parent rock. Parent rock texture preserved. Disintegrates when immersed into water.	All rock material is decomposed to soil. Original mass structure is still largely intact.
Residual soil	VI	Original rock texture completely destroyed. Friable.	Original mass structure is completely destroyed.

Table 2.1 Classification of basalt weathering grade

2.2.2 Porosity

Density and porosity are two fundamental properties of rocks, as will be shown in Chapter 4. The density of a rock is influenced by the mineral composition and the amount of voids or porosity. The ancient and intermediate volcanic series are made of alternate layers of fine grained compact olivine basalt flows and coarse agglomerate beds consisting of large subangular or rounded clasts of gravel to boulder size set in a matrix of volcanic ash. On the other hand, the recent basalts which cover about 70% of the island have a characteristic open sugary texture with small phenocrysts of olivine set in a doleritic groundmass of feldspars and pyroxenes. They were formed from very fluid flows with a high gaseous content resulting in vesicular and fractured deposits. The build up of the recent rocks to a maximum height of about 100 m took place by the superposition of lava flows with thickness varying from half to several m. Each individual flow can be distinguished by the high porosity at its boundaries; the porosity is low within the middle of the flow and increases significantly towards the top and bottom. A typical example of the variation of porosity within the upper 12 m of a recent basalt deposit is given in Figure 2.2.

Dry density (Mg/m^3)	Description	Porosity (%)	Description
< 1.8	Very low	> 30	Very high
1.8 – 2.2	Low	15 – 30	High
2.2 – 2.6	Moderate	5 – 15	Medium
2.6 – 2.8	High	1 – 5	Low
> 2.8	Very high	< 1	Very low

Table 2.2 IAEG Classification scheme for density and porosity of rocks (Anon, 1979)

Apart from the top 2 m of the deposit which is highly weathered, the underlying layers are slightly weathered so that the porosity has not been altered significantly by the weathering process. The individual flows which make up the deposit can be delimited at the highest values of porosity which are between 25 and 30%. With further decomposition, the porosity can reach 55% when the rock is completely weathered. Figure 2.3 gives an indication of the distribution of porosity for the number of specimens tested under the present project and Figure 2.4 shows the distribution for dry density. On the basis of these results, it was found that the International Association of Engineering Geologists (IAEG) classification scheme proposed in Table 2.2 by Anon (1979) could be adopted.

2.2.3 Strength of intact rock

Field estimation of the strength of intact rock can be done according to the commonly used scheme proposed BS5930 which is reproduced in Table 2.3. It must noted that extremely strong rock with an unconfined compressive strength higher than 200 MPa has not yet been encountered. Figure 2.4 shows the distribution of strength for the fresh and slightly weathered rock specimens tested. It can be found that the basalts are mostly moderately strong to strong.

Description	Approximate unconfined compressive strength (MPa)	Field estimation
Very weak	< 1.25	Gravel size lumps can be crushed between finger and thumb.
Weak	1.25 – 5	Gravel size lumps can be broken in half by heavy hand pressure.
Moderately weak	5 – 12.5	Only thin slabs, corners or edges can be broken off with heavy hand pressure.
Moderately strong	12.5 – 50	When held in the hand, rock can be broken by hammer blows.
Strong	50 – 100	When resting on a solid surface, rock can be broken by hammer blows.
Very strong	100 – 200	Rock chipped by heavy hammer blows
Extremely strong	> 200	Rock rings on hammer blows. Only broken on sledge hammer.

Table 2.3 BS 5930 Classification of rock strength

While Table 2.3 gives guidelines for the estimation of compressive strength of rocks, simple methods can be used to carry out such an estimate. One of the widely used equipment is the point load tester that provides a simple and quick method to assess the Unconfined Compressive Strength (UCS) of rock cores, especially when samples do not have the required length for the compressive strength test. The test was developed primarily to give an index for strength classification and characterization of intact rocks for engineering applications. In the test, the specimen of known dimensions is subjected to compression between two 60^o cone-shaped platens with a contact point of 5 mm radius. The failure load (P) is measured and the point load index ($I_s = P/D^2$ where D is the diameter of the core specimen or distance between the platen tips for an irregular lump) is calculated. For specimens with dimensions other than 50 mm, the point load index is normalized to 50 mm diameter to give $I_{s(50)}$. However, Bieniawski (1975) considered that NX cores (54 mm diameter) would be a better standard than 50 mm diameter cores. The use of the point load tester requires the multiplication of the test point load index by a correlation factor K to obtain the UCS. Broch & Franklin (1972) recommended that K was 24. Bowden, Lamont-Black and Ullyott (1998) concluded from a study of the literature that there were significant departures from what was considered to be a general K of 24. They reported that K varied from 10 to 30 for a range of rock types and from 7.5 to 18 for the vesicular basalt of New Zealand. From their own experiments, they also concluded that K is strength dependent. Chan Chim Yuk (2002) found that, for the basalts of Mauritius, K varied mostly between 6 and 16 and decreased with increase in porosity. To obtain an estimate of unconfined compressive strength from diametral point load indices on basalt, a coefficient K of 12 was recommended (refer to Chapter 4).

2.2.4 Fracture state

The fracturing of the local basalt originated from weathering along the beds of lava flows, the shrinkage of lavas during cooling and the subsidence/collapse of underlying voids. The behaviour of a rock mass is, to a large extent, determined by the type, spacing and characteristics of the fractures. The type and characteristics of the fractures are classified by means of descriptive terms and the classification of the fracture spacing is done by using the rock quality designation (RQD) of rock cores introduced by Deere (1964). RQD is the sum of the length of the solid core pieces longer than 100 mm expressed as a percentage of the core run. A commonly used RQD classification system is given in Table 2.4.

RQD (%)	Classification
0 – 25	Very poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

Table 2.4 Classification of rock quality based on RQD

2.3 Description scheme for soils

The highly and more weathered (weathering grades IV to VI) basalts tend to behave as soils. The recommendations of Simmons and Blight (1997) for descriptions of tropical soils can be used. The essential elements required in soil description are described below.

2.3.1 Soil classification

The BS5930 soil classification scheme based on particle size distribution and plasticity is used.

2.3.2 Moisture condition

A description of the moisture state is useful as a guide to identification of soil type and to potential soil behaviour. The descriptive terms and field guides are given in Table 2.4.

Term	Field identification
Dry	Cohesive soils hard and brittle. Granular soils free-running.
Moist	Darkened in colour. Cohesive soils can be moulded by hand. Granular soils tend to cohere.
Wet	Darkened in colour. Cohesive soils easily moulded by hand. Granular soils tend to cohere. Free water on hands when remoulding.

Table 2.5 Descriptive terms for moisture conditions (Simmons and Blight, 1997)

2.3.3 Strength and relative density

There are several schemes, such as Simmons and Blight (1997) and BS 5930, for the classification of the strength of cohesive soils and of the relative density of cohesionless soils. In the present study, it was decided to adopt BS 5930 schemes. These schemes are given in Table 2.6 and 2.7.

Term	Undrained shear strength (kPa)	Field test
Very soft	<20	Finger easily pushed in up to 25 mm.
Soft	20 – 40	Finger pushed in up to 25 mm.
Firm	40 – 75	Penetrated by thumb with effort. Moulded by strong finger pressure.
Stiff	75 – 150	Indented by thumb. Cannot be moulded by fingers.
Very stiff	150 – 300	Indented by thumbnail.
Hard	>300	Cannot be indented by thumbnail.

Table 2.6 BS 5930 descriptive terms for classification of cohesive soils in terms of strength

Term	Relative density (%)	SPT N value	Field test
Very loose	< 20	< 4	
Loose	20 – 33	4 – 10	Can be excavated by spade. 50 mm peg can be easily driven. Easily crushed in fingers.
Medium dense	33 – 66	10 – 30	
Dense	66 – 90	30 – 50	Requires pick for excavation. 50 mm peg hard to drive. Crushed by strong finger pressure.
Very dense	90 – 100	> 50	

Table 2.7 Descriptive terms for classification of cohesion soils in terms of relative density (BS 5930; Anon, 1979)

Dry density (Mg/m^3)	Description	Porosity (%)	Description
< 1.4	Very low	> 50	Very high
1.4 – 1.7	Low	45 – 50	High
1.7 – 1.9	Moderate	35 – 45	Medium
1.9 – 2.2	High	30 – 35	Low
> 2.2	Very high	< 30	Very low

Void ratio	Description	Degree of saturation (%)	Description
> 1	Very high	< 25	Naturally dry
0.8 – 1.0	High	25 – 50	Wet
0.55 – 0.8	Medium	50 – 80	Very wet
0.43 – 0.55	Low	80 – 95	Highly saturated
< 0.43	Very low	> 95	Saturated

Table 2.8 IAEG description of dry density, porosity, void ratio and degree of saturation of soils (Anon, 1979)

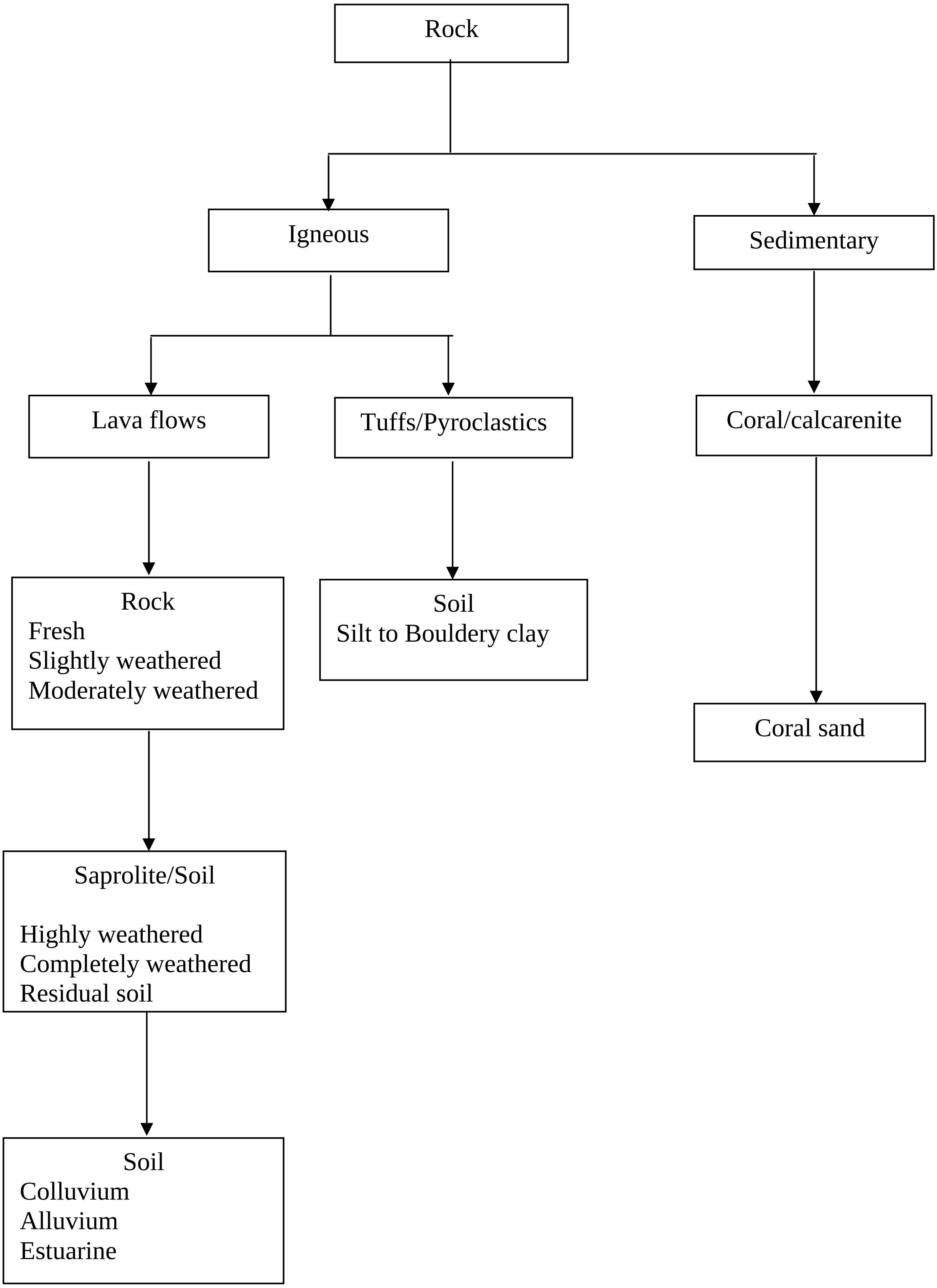


Figure 2.1 Soil formation in Mauritius

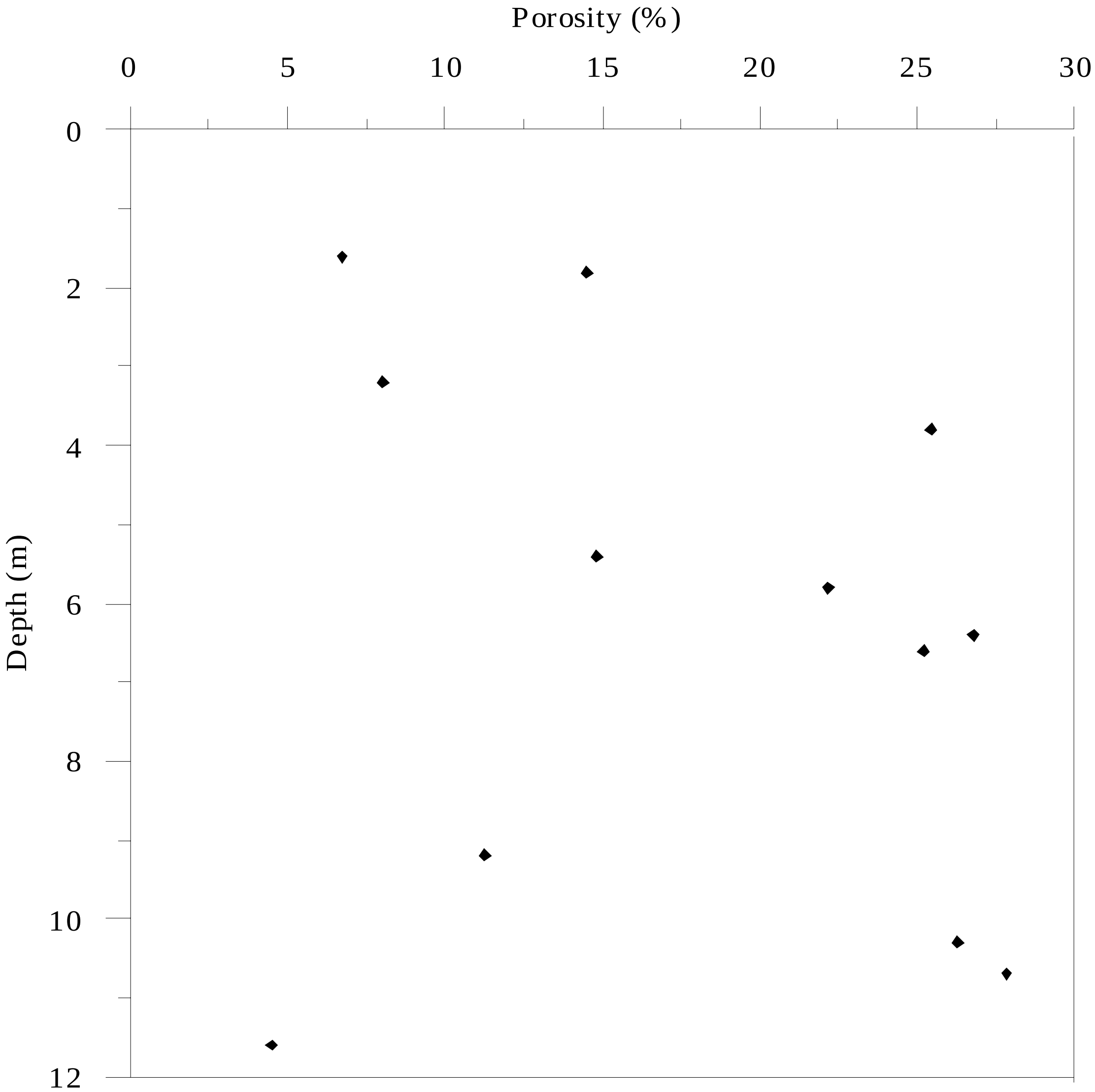


Figure 2.2 Variation of porosity with depth in basalt deposit

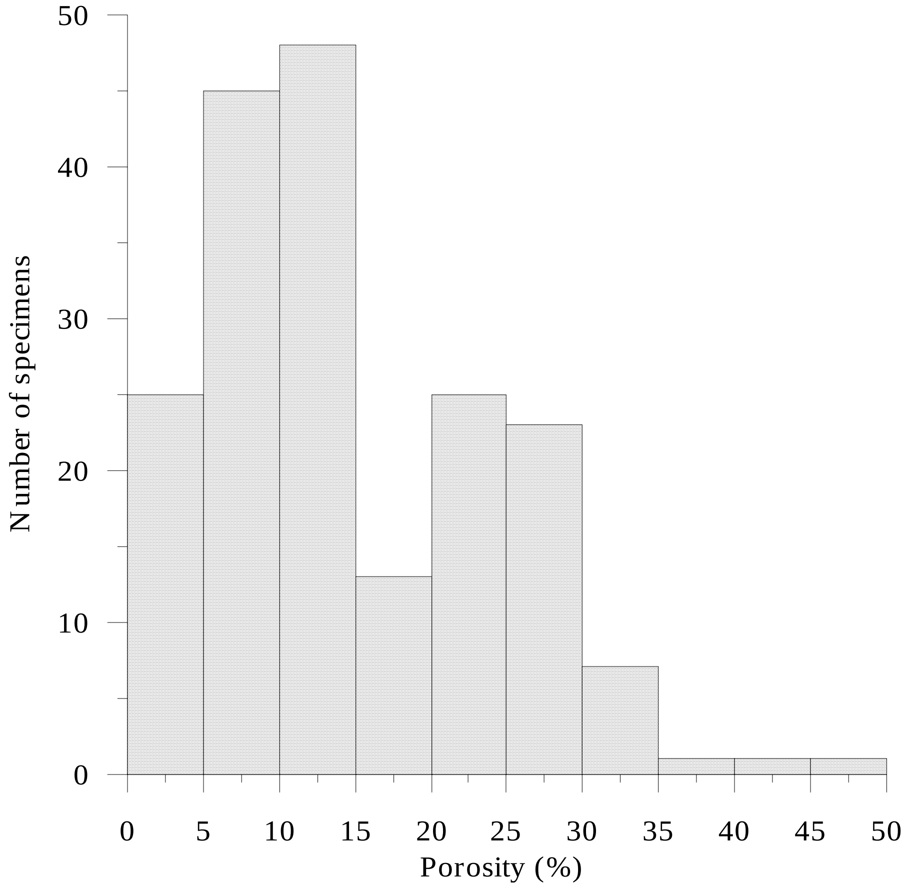


Figure 2.3 Histogram of porosity of fresh and slightly weathered basalt specimens

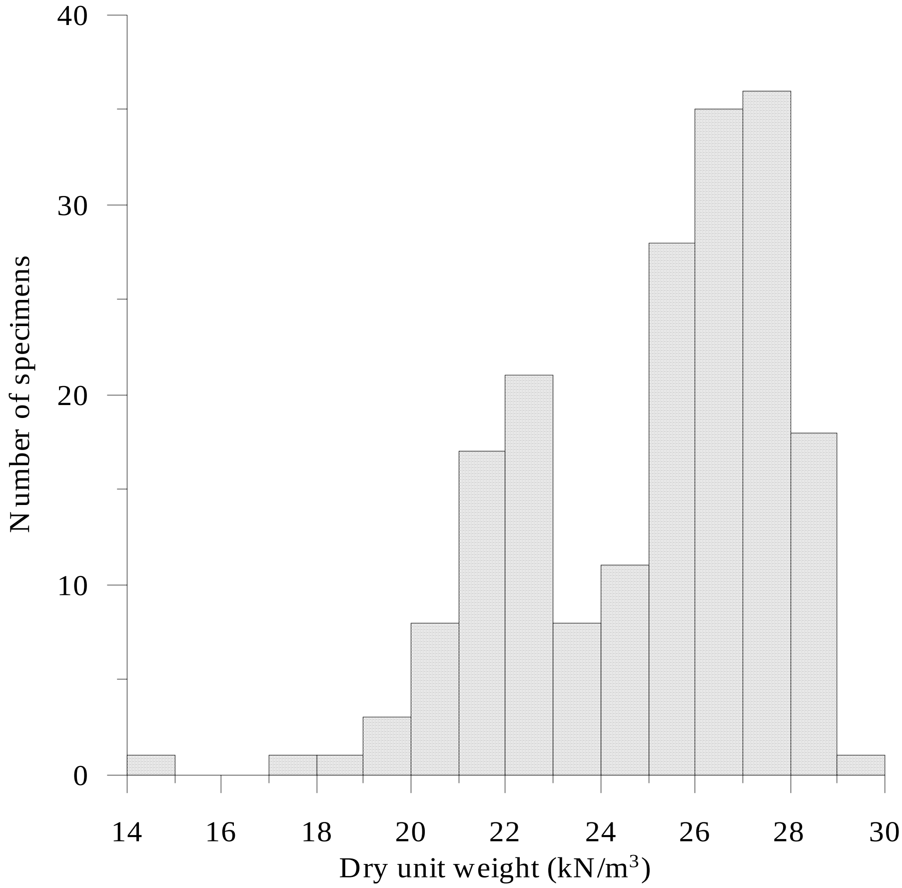


Figure 2.4 Histogram of dry unit weight of fresh and slightly weathered basalt specimens

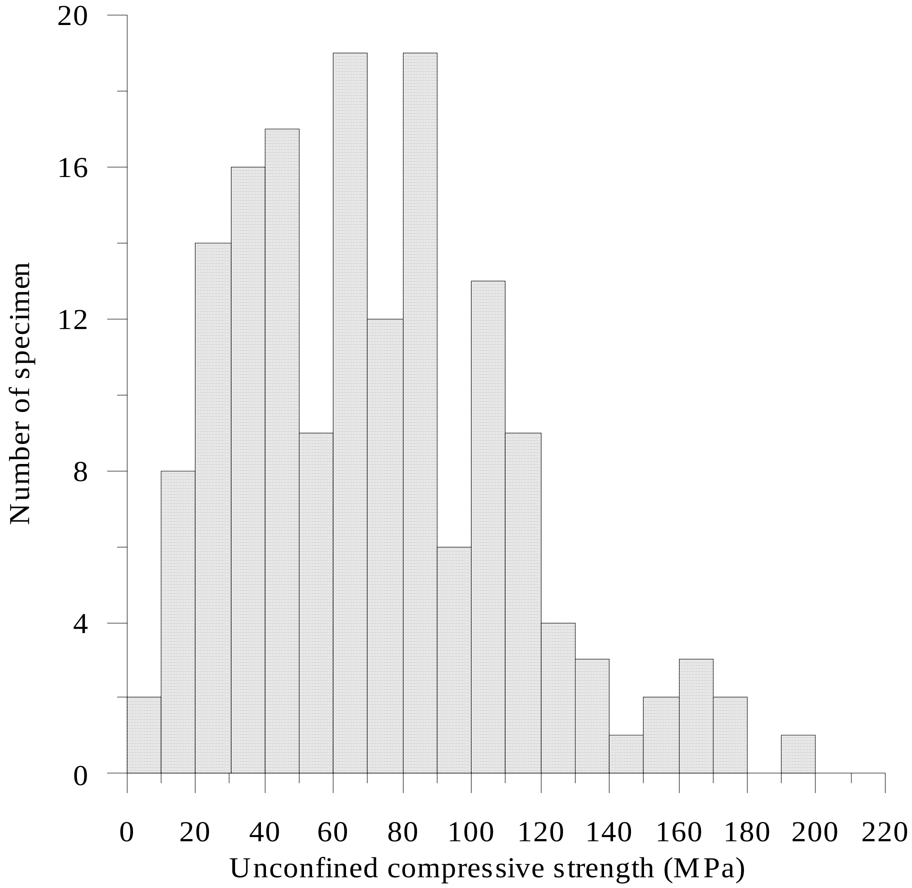


Figure 2.5 Histogram of unconfined compressive strength of fresh and slightly weathered basalt specimens

CHAPTER 3

GEOLOGICAL DATA MANAGEMENT SYSTEM

3.1 Introduction

Our database includes information from over 300 boreholes drilled and trial pits excavated during ground investigations for civil engineering projects. Logs have been produced according to the descriptive terms and classification systems described in Chapter 2. These information have been converted into an electronic format by using a commercial software package SID. The possibility of producing logs and cross-sections readily from a spatial distribution of geological information helps in the understanding of the geology of a study area or a region. It is the objective of the author as part of a future project to make available the electronic information on the internet for the benefits of the engineering community.

3.2 SID database management system

The computer software package SID developed by MZ Associates of the UK was acquired in 1996 to create an electronic version of the geological database available with the author. This package was developed using the FOXPRO database software. Information from site investigations are stored in a standard format as proposed by the Association of Geotechnical and Geoenvironmental Specialists (AGS) of the UK. SID provides a database with more than 600 fully defined data fields, comprehensive data integrity checking, validation and calculation procedures, and a wide range of data import and export file formats.

The outputs are in the form of the following:

- Borehole logs to visualize the strata.
- Cross-sections to show the different types and depth of strata along a given direction.

Example of a log that can be generated by the software is shown in Figure 3.1 and that of a cross-section is shown in Figure 3.2.

As at 2002, the borehole information compiled for each district of Mauritius are indicated in Table 3.1.

District	No. of boreholes
Port Louis	215
Plaines Wilhems/Grand Port	101
Pamplemousses/Riviere du Rempart	50
Moka	16
Flacq	9
Black River	20
Savanne	7

Table 3.1 Available borehole information in database

3.3 Case Study: Geology of Port Louis

The database of geological information for Port-Louis has allowed a better understanding of the geology of Port-Louis. It is known that the Older Volcanic bedrock underlying the Port-Louis area was subjected to severe erosion. Large and deep erosional channels originating from the Moka Mountain Range were cut in the bedrock during a period when the sea level was 60 metres or more below its present level. Subsequently, as a result of sea level rise during the Quaternary Period, sediments were deposited in these channels. The location and extent of the latter are not apparent from the present ground surface topography. With the detailed ground investigation undertaken in recent years as a result of the increasing development in Port-Louis, it has been possible to map part of the buried channel which underlies the present Le Pouce Stream. A section of the channel is shown in Figure 3.3 (Chan Chim

Yuk et al, 1998) which indicates that, at John Kennedy Avenue, the channel is more than 400 m wide and about 30 m deep.

Figure 3.4 shows a typical profile at the foot of a hill where a thick layer of colluvium was found to overlie weathered bedrock of the older volcanic series. At Colline Monneron, it was found to be 8 to 10 m thick, whereas at other locations in Port-Louis, the thickness of the colluvium could be up to about 20 m. The colluvium is made up of the dark magnesium clay mixed with coarse basaltic clasts. The dark magnesium clay can be a problematic soil as will be discussed in Chapter 5.

Figure 3.5 shows a complex geological profile that was encountered near the sea front in Port-Louis. It indicates that that there was a period of clay deposition in a marine environment after the period of older volcanic activities and before submergence by the younger volcanic basalt flows. Finally, due to a rise in sea level, an estuarine mud was deposited on the latter.

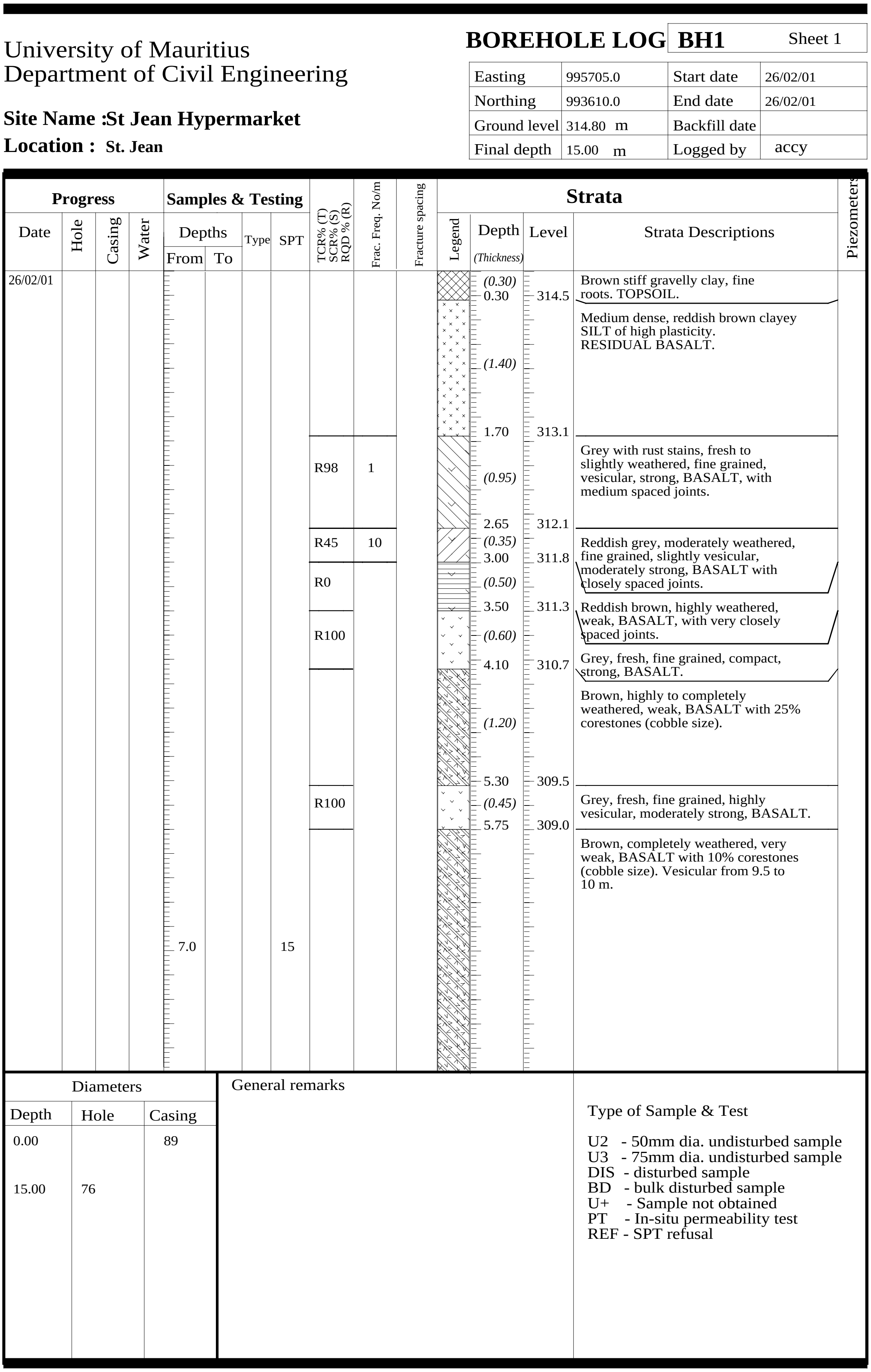


Figure 3.1 Example of log generated by SID software

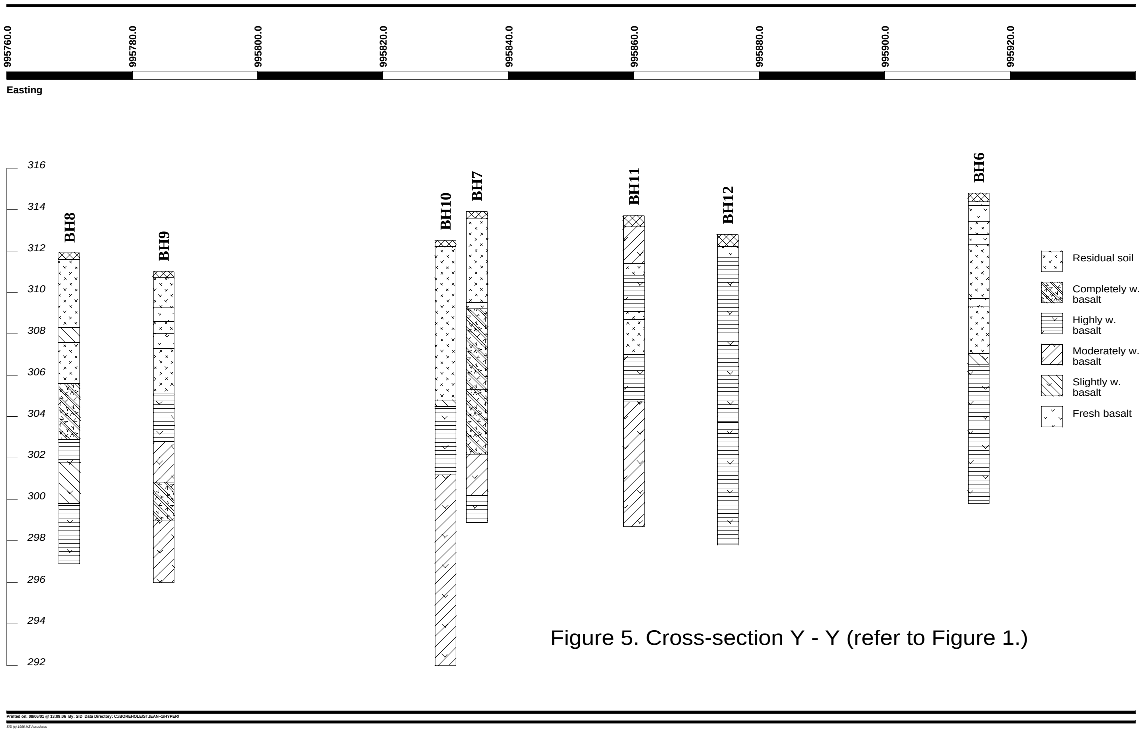


Figure 3.2 Example of cross-section generated by SID software

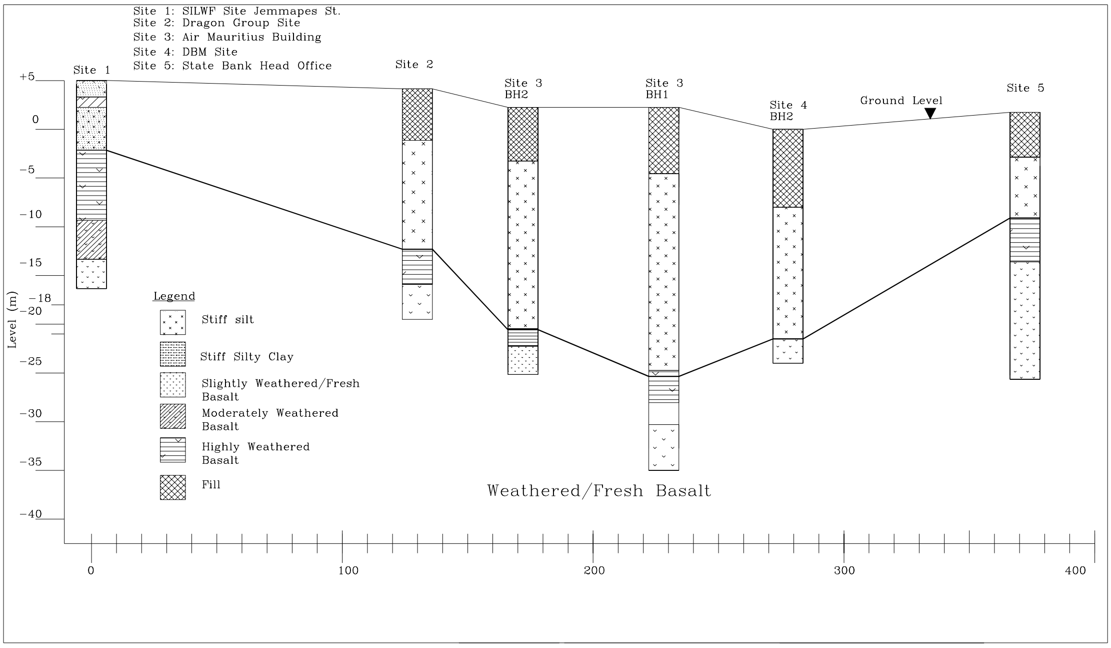


Figure 3.3 Geological profile along John Kennedy Avenue, Port-Louis.

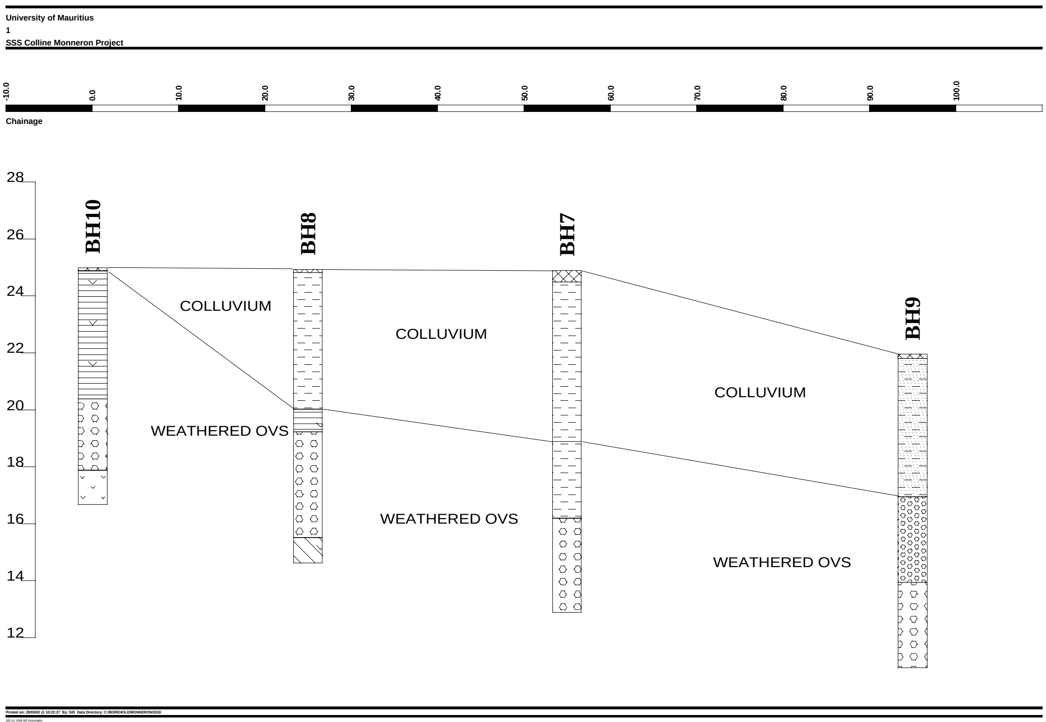


Figure 3.4 Geological profile at the foot of Monneron Hill, Port-Louis.

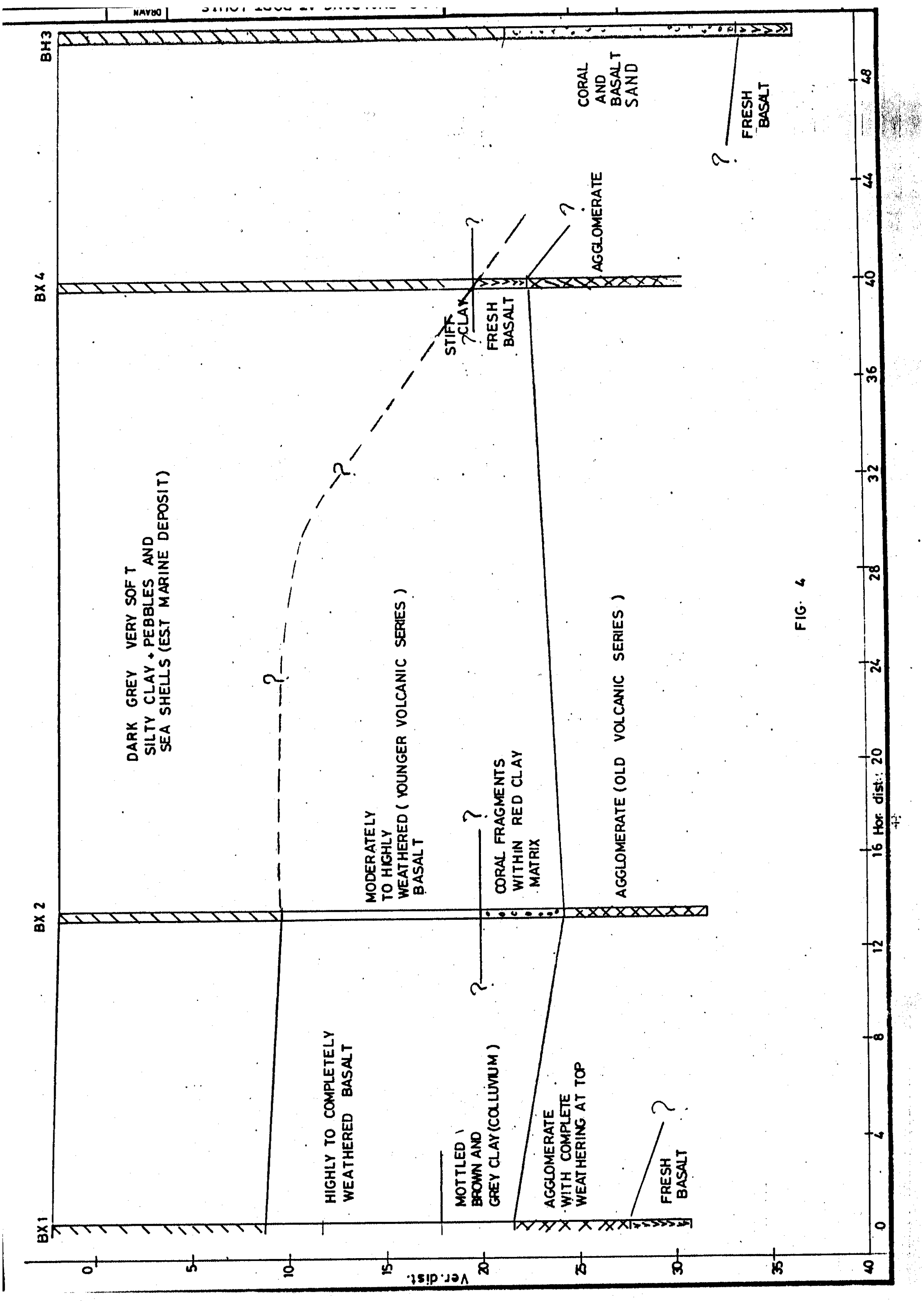


Figure 3.5 Complex geological profile at the sea front in Port-Louis

CHAPTER 4

PROPERTIES OF ROCKS

4.1 Introduction

In this chapter, the basalt with a degree of weathering of less than moderate as described in Table 2.1 is classified as rock. The data on basaltic have been compiled from the reports of Rampertaub (1993), Bissessur (1995), Ramburrun (2000), Doobory (2001) and the author's own database.

Some coastal areas with coral deposits have been encountered in Port-Louis. In certain cases, these deposits have been used as founding strata and therefore, assessment of their properties is essential. A limited amount of data on coral deposits is available and is also included in this chapter.

4.2 Fresh and slightly weathered basalt

4.2.1 Strength

Strength is used to predict the ultimate load carrying capacity of the ground. For rocks, the unconfined compressive strength (UCS) is normally determined and the mass strength is estimated on the basis of the characteristics of the discontinuities and degree of weathering. Because of the variability in rock strength and the fact that it is practically impossible to test a sufficiently large number of specimens to produce statistically significant results, Figure 4.1 which shows the correlation between unconfined compressive strength and porosity can be used to make a preliminary assessment of the rock strength. In addition, Figure 4.1 is useful when rock samples are too fractured to allow for compressive strength testing so that the UCS can be estimated from its porosity, n, using equation 4.1; an exponential relationship for the best-fit curve was derived to relate the compressive strength to porosity (n) expressed as a percentage. On the other hand, Figure 4.2 shows the relationship between UCS and bulk unit weight of basalt. The bulk unit weight can be used to estimate the compressive strength of a rock from equation 4.2. The nonlinear trend is in agreement with observations made by Lutton (1968) on basaltic rocks.

It is to be noted that, for equations 4.1 and 4.2, the coefficient of determination, R^2, of 0.70 indicates a good correlation of UCS with porosity or bulk unit weight, in spite of physical and textural heterogeneities inherent in basalt deposits.

$$UCS = 145\, e^{-0.064n} \tag{4.1}$$

$$UCS = 0.19\, e^{0.22\gamma} \tag{4.2}$$

4.2.2 Compressibility

For rocks and the loading conditions of structures presently being designed in Mauritius, the modulus of elasticity is used to estimate deformation of rock masses. It is common to estimate the modulus of elasticity from the modulus ratio which is the ratio of modulus of elasticity to unconfined compressive strength. In the present project, the modulus of elasticity was computed from the tangent modulus determined at 50% of the ultimate strength. Figure 4.3 shows the histogram of modulus ratio for the tested specimens. The mean value of modulus ratio is 600.

4.2.3 Sonic velocity

Sonic velocity was determined by using the Pundit equipment on some of the rock specimens before compression tests were carried out. From Figure 4.4, the relationship between UCS and sonic velocity (v_s in km/s) was given by equation 4.3. There was more scatter in the test results between UCS and sonic velocity than in the results between UCS and porosity or bulk unit weight, most probably due to the errors inherent in the measurement of sonic velocities.

$$UCS = 0.79\, e^{0.88vs} \tag{4.3}$$

4.2.4 Point load index

The test was developed by Broch and Franklin (1972) primarily to give an index for strength classification and characterization of intact rocks for engineering applications. It is also used in the prediction of uniaxial tensile and compressive strengths. In the test, the specimen of known dimensions is subjected to compression between two 60^o cone-shaped platens with a contact point of 5 mm radius. The failure load (P) is measured and the point load index ($I_s = P/D^2$ where D is the diameter of the core specimen or distance between the platen tips for an irregular lump) is calculated. For specimens with dimensions other than 50 mm, the point load index is normalized to 50 mm diameter to give $I_{s(50)}$. However, Bieniawski (1975) considered that NX cores (54 mm diameter) would be a better standard than 50 mm diameter cores. He also recommended that the test should be limited to rocks with UCS greater than 25 MPa or point load index above 1 MPa. But Bowden, Lamont-Black and Ullyott (1998) obtained consistent results for chalk with strength range of 3 to 8 MPa and concluded that the test was reliable provided that it was performed strictly according to ISRM standards.

The test can be performed in a number of ways such as the diametral and the axial test for cores and the irregular lump test. In the diametral test, Broch and Franklin (1972) recommended that the distance between the contact point and each end of the specimen should be at least 0.7 times the diameter and in the axial test, specimens with a length to diameter ratio of 1 would be suitable. The ISRM standard (ISRM, 1985) stipulates that loading should be applied at an even rate and failure of the specimen should take place between 10 to 60 seconds from start of the loading.

Tests were performed on NX triple tube core barrel samples obtained from both the OVS and YVS basalts and the procedures of ISRM standards (Brown, 1981) were adopted to determine unit weights, porosity, UCS and point load strength. For the point load test, the load was applied by using a compression machine instead of the hydraulic jack supplied with the standard equipment. This was done to ensure that the loading was applied at an even rate, to control the time to failure and for accurate measurement of the failure load. No correction was applied to the point load strength being given that NX cores were being tested.

The study was carried out in two phases. In the first instance, samples were selected for testing if they were more or less homogeneous in terms of porosity and had an intact length of at least 200 mm so that, from the same length of core, it was possible to perform concurrently both a UCS test on a specimen with a height to diameter ratio of 2.5 and a diametral point load test. This series of tests permitted the determination of the diametral correlation factor K(d). However, as a result of this selection criterion, test results were obtained mostly on cores with UCS of 40 to 200 MPa and porosity of less than 15 %. In order to obtain results for less strong and more porous rocks, it was decided to carry out the compression test and point load test separately on independent specimens. Therefore, K could not be determined because the two tests were not conducted on similar specimens but correlations of strength could be derived on the basis of porosity. In this phase of investigation, both axial and diametral point load tests were performed.

The diametral and axial point load test results are plotted in Figure 4.5 (a) and (b) respectively. The correlations of the point load indices with porosity are given in equations 4.4 and 4.5.

$$I_s(d) = 12\ e^{-0.050n}\ \text{MPa} \tag{4.4}$$

$$I_s(a) = 14\ e^{-0.049n}\ \text{MPa} \tag{4.5}$$

It is found that there is more scatter in these results than in those of the compressive strength tests and this may be due to the effects of vesicles within the small contact area over which the load is applied in a point load test. In addition, within the same specimen, there may be heterogeneity in the distribution of the vesicles so that the porosity on the fracture plane may be different from the average porosity determined on the specimen as a whole. On the other hand, there is more scatter between the diametral index, $I_s(d)$, and porosity (R^2 = 0.50) than between the axial index, $I_s(a)$, and porosity (R^2 = 0.64). The ratio of axial index to diametral index is estimated to be 1.2.

This reflects the influence of the beddings in the diametral tests where the load is applied parallel to bedding planes which are planes of weakness. Although it may be desirable to perform the axial test because of the lesser scatter in the results, such a test can only be done after the specimens have been cut at both ends. The cutting facilities are normally not available in the field so that only diametral tests are possible. In such a case, an assessment of K(d) is required. Figure 4.5 also shows that the scatter in I_s is similar throughout the range of porosities tested, implying that the quality of the results for the rocks of high porosity is the same as for the less porous ones.

The results of diametral index and compressive strength determined on pairs of similar specimens selected from the same length of cores are plotted in Figure 4.6. There is an evident linear relationship between $I_s(d)$ and UCS with a high R^2 of 0.94 for the best-fit line through the origin, giving an average K(d) of 12. There is thus an important disagreement with the value of 24 recommended by Broch & Franklin (1972).

4.3 Moderately weathered basalt

A series of tests were conducted on moderately weathered basalt. The degree of weathering was identified using the guidelines of Table 2.1. The range of porosities tested are given in Figure 4.7. The relationship between UCS and porosity is given in equation 4.6. There is more scatter in the data because there is significant variation in the decomposition of the rocks although they are all grouped under one single class. The correlation between UCS and bulk unit weight is very poor as shown in Figure 4.9 and is not reported here.

$$UCS = 198\, e^{-0.107n} \qquad (4.6)$$

Figure 4.10 shows the distribution of modulus ratios with a mean value of 470. As expected, there is a decrease in the modulus ratio with weathering of the rock.

4.4 Corals

Coral strata overlying basalt bedrock have been encountered in the coastal zones of Port Louis and have influenced the design of building foundations. The coral which is a weak rock is very heterogeneous in terms of structure. Hence, the strength properties are highly variable as shown in Figure 4.11 to 4.14. However, these figures and the correlations given in equations 4.7 and 4.8 are useful information and can be used in preliminary designs.

$$UCS = 17.5\, e^{(-0.028n)} \tag{4.7}$$

$$UCS = 0.74\, e^{(0.11\gamma)} \tag{4.8}$$

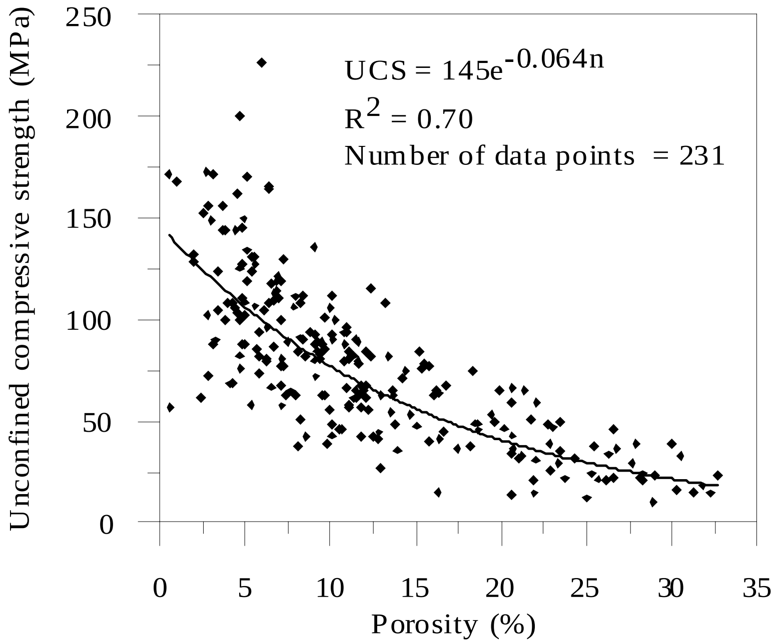


Figure 4.1 Correlation between unconfined compressive strength and porosity

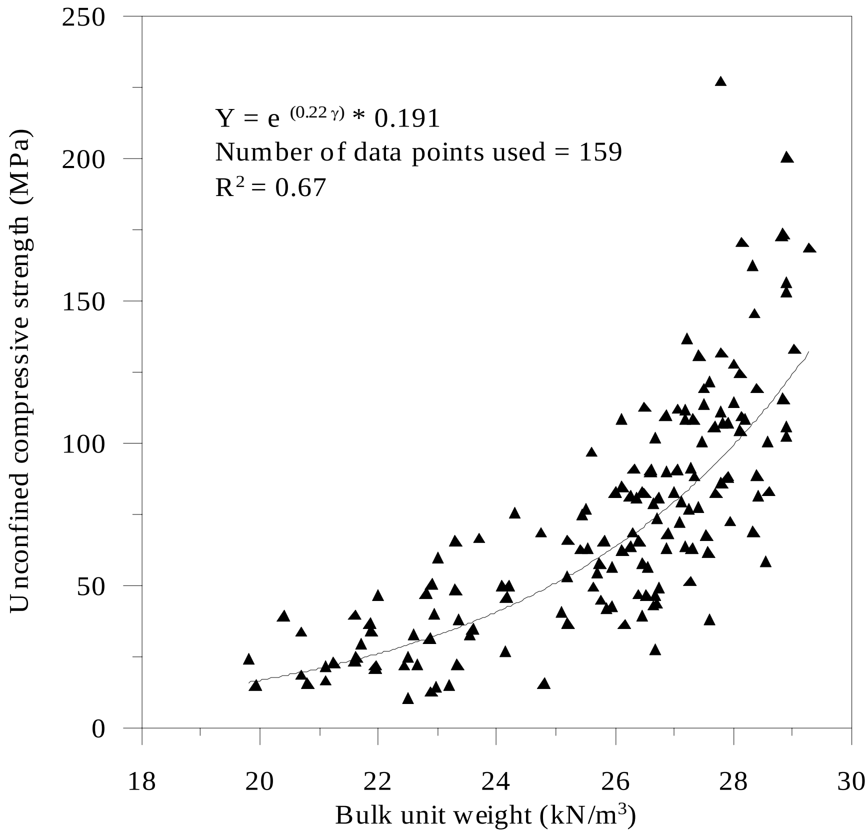


Figure 4.2 Correlation between UCS and bulk unit weight

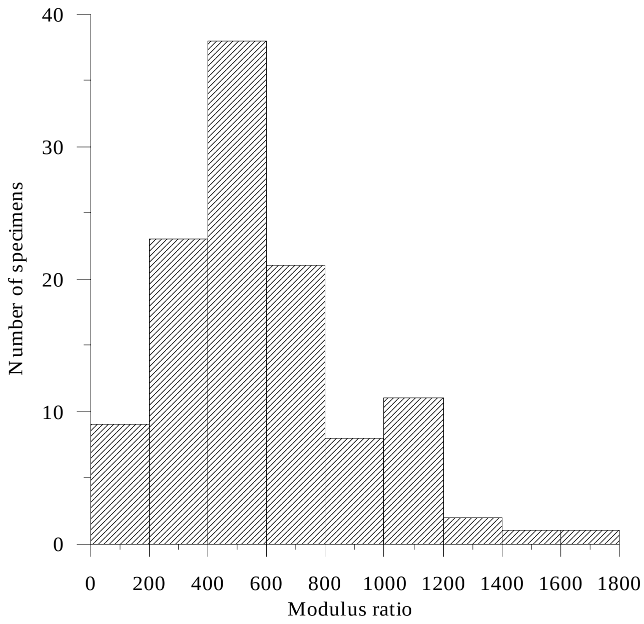


Figure 4.3 Histogram of modulus ratios

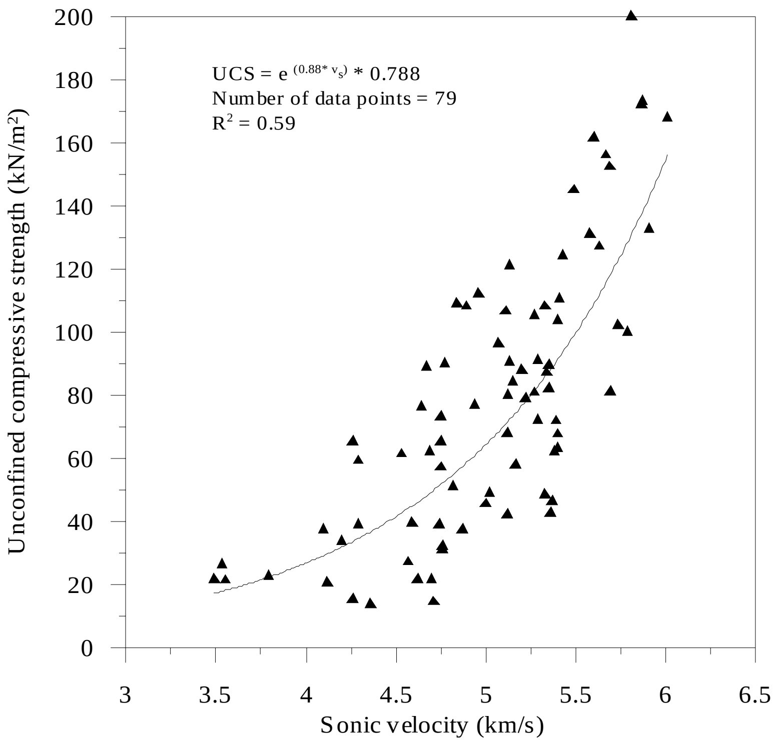


Figure 4.4 Correlation between UCS and sonic velocity

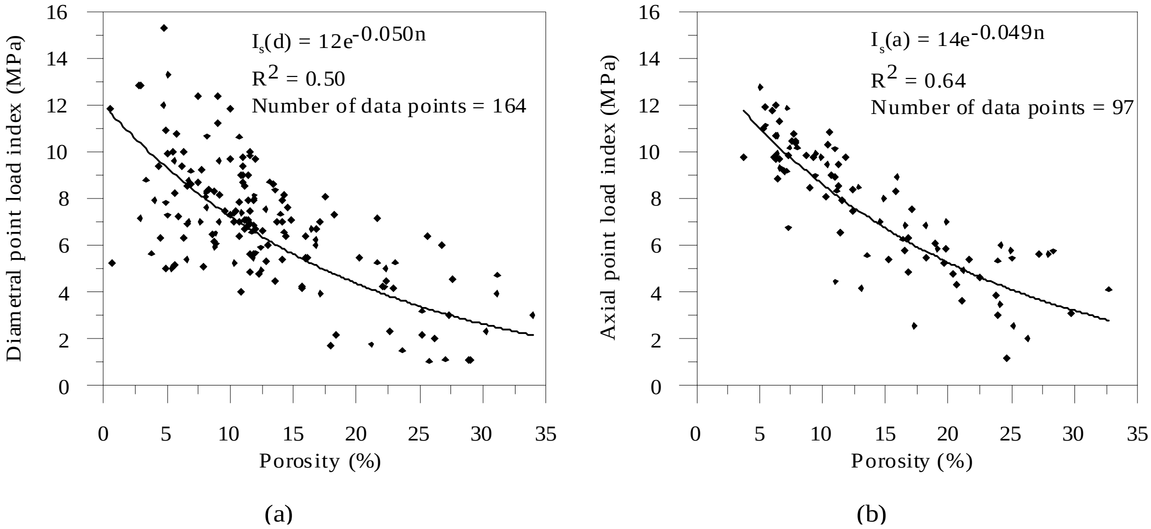


Figure 4.5 Relationship between (a) diametral and porosity
(b) axial point load index and porosity

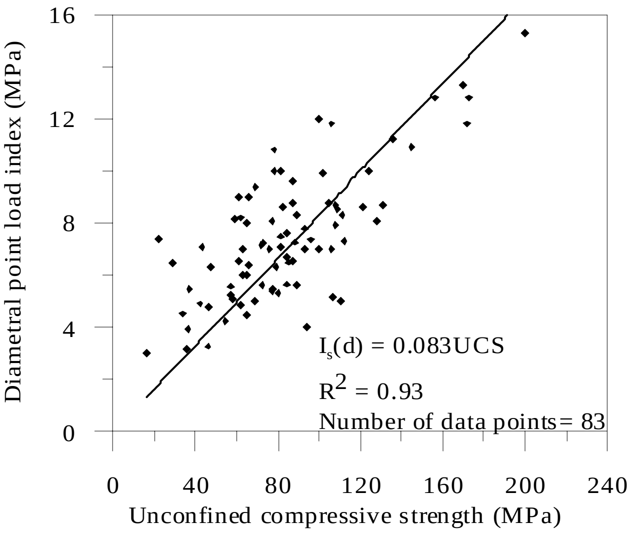


Figure 4.6 Correlation between diametral point load index and UCS

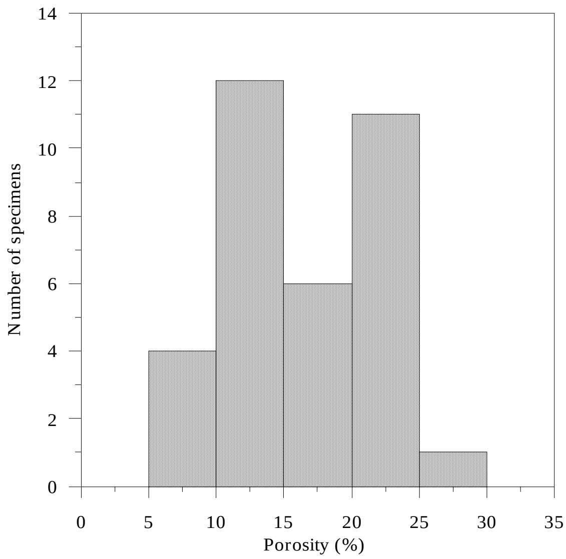


Figure 4.7 Histogram of porosity of moderately weathered basalt specimens

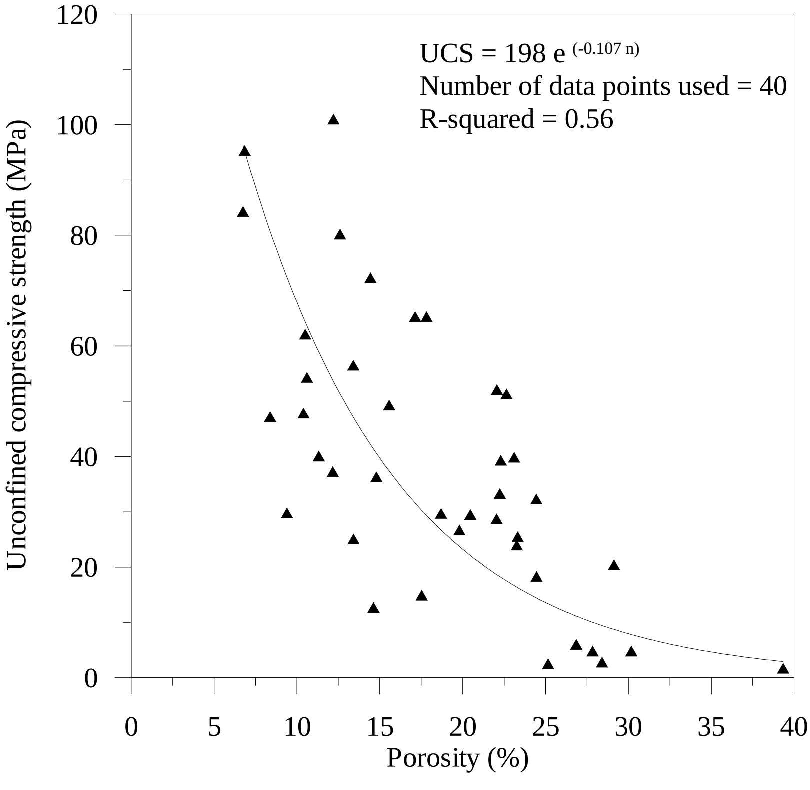


Figure 4.8 Correlation between unconfined compressive strength and porosity of moderately weathered basalt

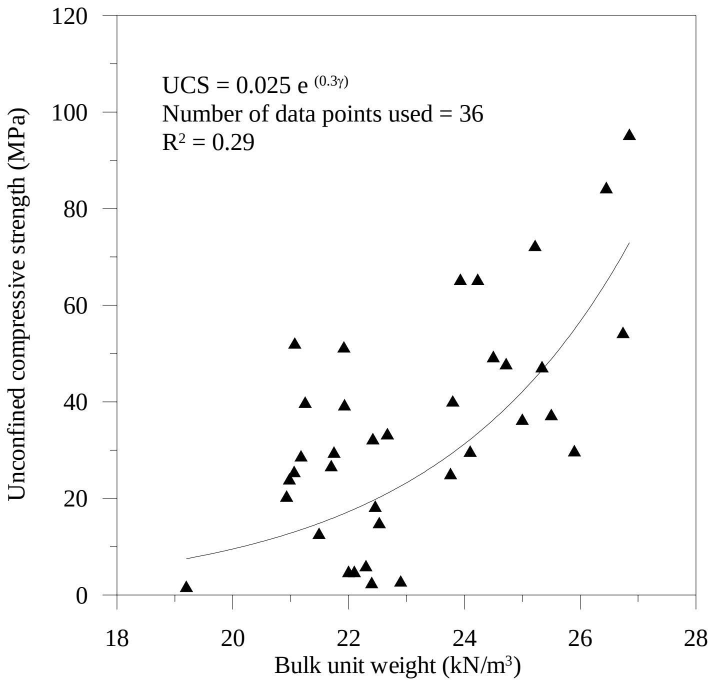


Figure 4.9 Correlation between UCS and bulk unit weight of moderately weathered basalt

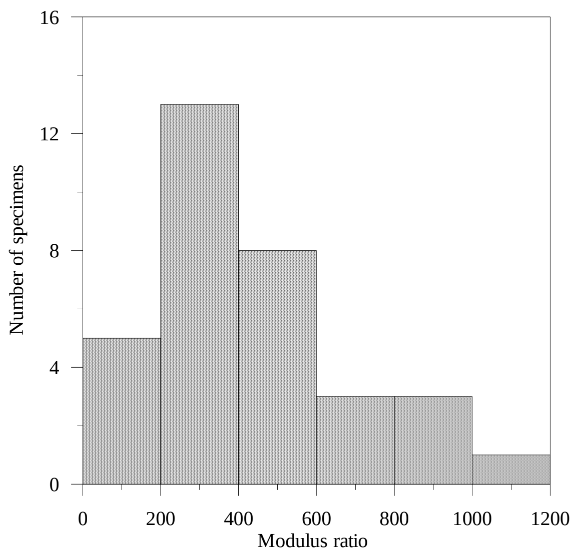


Figure 4.10 Histogram of modulus ratios of moderately weathered basalt

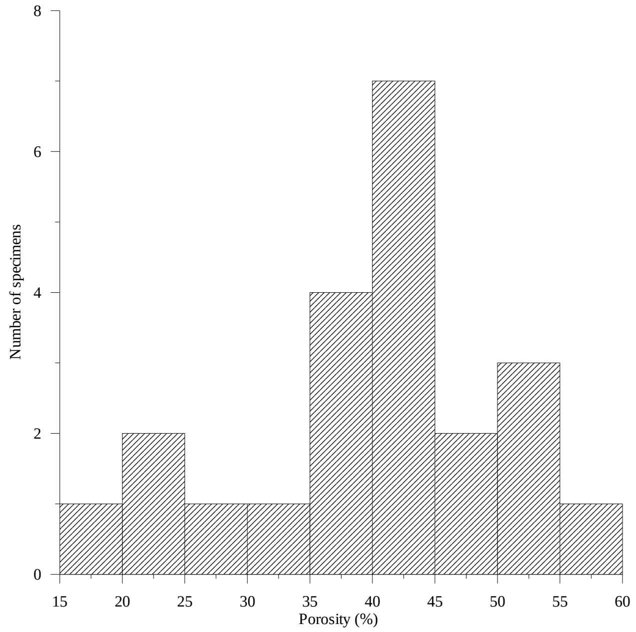


Figure 4.11 Histogram of porosity of coral specimens

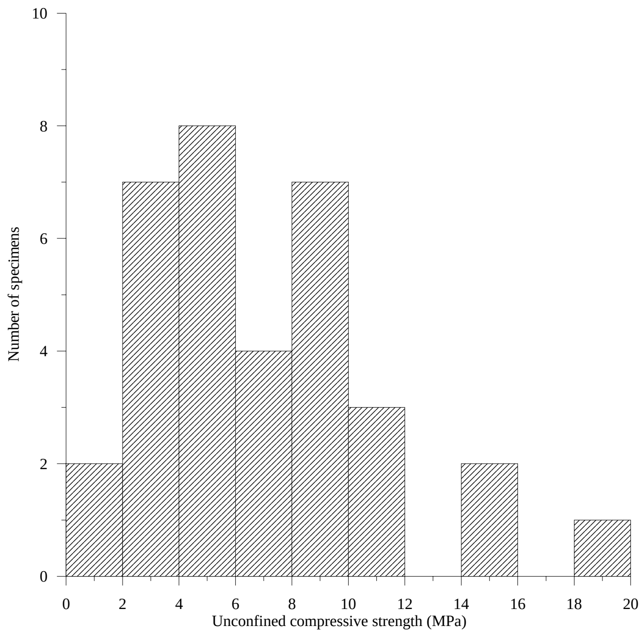


Figure 4.12 Histogram of UCS of coral specimens

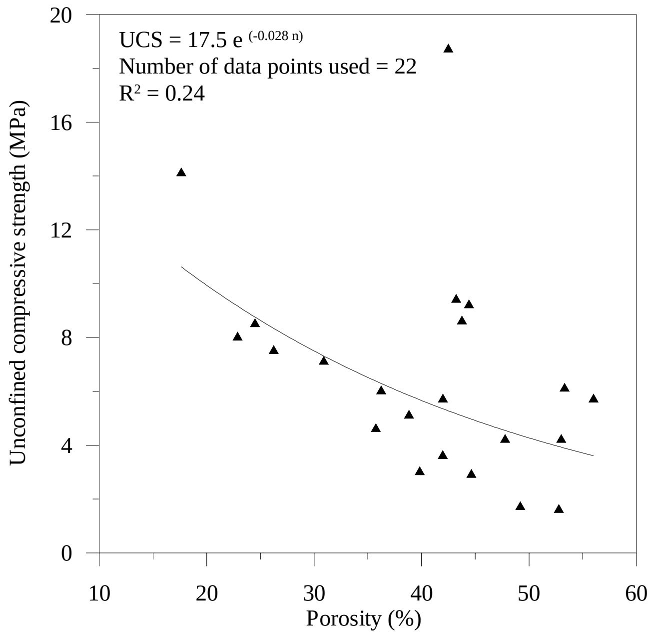


Figure 4.13 Correlation between UCS and porosity of coral

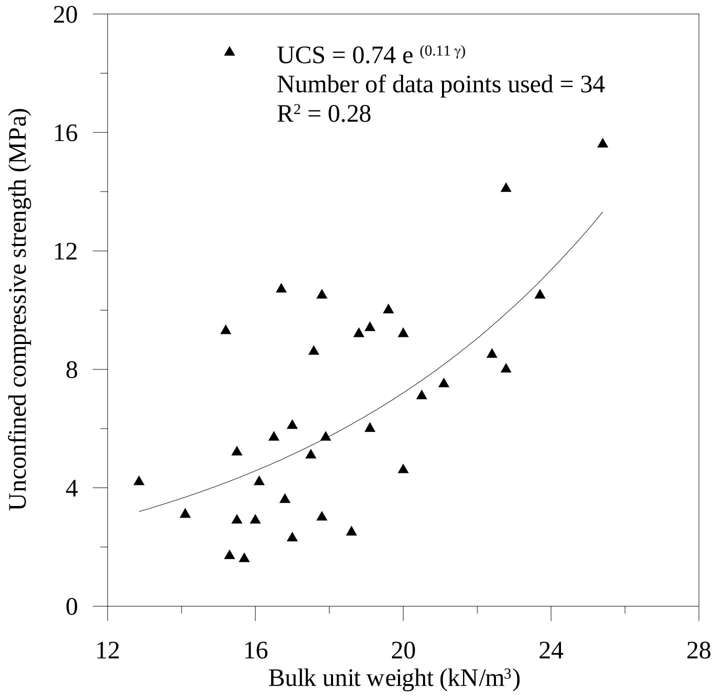


Figure 4.14 Correlation between UCS and bulk unit weight of coral

CHAPTER 5

PROPERTIES OF SOILS

5.1 Introduction

The purpose of this chapter is to present the basic properties of some local soils and the correlations between essential engineering parameters and simple soil indices. Data for the main residual soil groups and the colluvial soils have been analysed and the results are presented. Further work remains to be done on the available data for coral sand and estuarine deposits.

5.2 Soil origin

There are two main types of soils in Mauritius, namely residual and transported.

The highly and more weathered (weathering grades IV to VI) basalts tend to behave in engineering terms as soils. The latter is formed from in-situ physical disintegration and chemical decomposition of basaltic rock and volcanic pyroclastic deposits. The MSIRI Soil Map of Mauritius (Parish and Feillafe, 1965) has been used to group these soils; the MSIRI classification system is shown schematically in Figure 5.1. The five main soil types have been investigated.

On the other hand, a transported soil is one which has been transported by water, wind and gravity. The different types of transported soils are given in Table 5.1.

5.3 Characteristics of residual soils

5.3.1 Specific gravity

Specific gravity is used in determining the void ratio or porosity of a soil mass. As far as possible, the determination of residual soils should be carried out at its natural moisture content because, with some clay minerals, drying can cause the loss of intraparticle water resulting in a lower specific gravity being measured. The range of values obtained for the five soil types are plotted in Figure 5.2 and some statistical data are given in Table 5.2. It can be observed that the mature soils have higher specific gravities than immature soils. This is due to an increase in the iron oxide content (Turner et al, 1977).

Type	Transporting agent	Source	Soil type
Talus/scree/coarse colluvium	Gravity	Mostly physical weathering of rock outcrops	Unsorted angular clasts (gravel to boulder size) within a matrix of dark magnesium clay
Hillwash/fine colluvium	Water	Extreme disintegration, oxidation, strong hydration and restricted leaching in the chemical weathering of olivine basalt	Mostly montmorillonitic clay, locally known as dark magnesium clay
Alluvium	Rivers and streams	Rocks and soils	Boulders, cobbles, sands, silts
Estuarine deposits	Tidal rivers and waters	Terrigenous and marine soils	Sands, silts and clays
Littoral/aeolian deposits	Waves/wind	Coral	Sands

Table 5.1 Types of transported soils

	Latosolic Red Prairie	Latosolic Brown Forest	Low Humic Latosol	Humic Latosol	Humic Ferruginous Latosol
Mean	2.77	2.82	2.80	2.92	2.87
Median	2.78	2.86	2.82	2.93	2.89
Standard deviation	0.08	0.15	0.08	0.07	0.15
Coefficient of variation	0.03	0.05	0.03	0.03	0.05

Table 5.2 Statistical information on specific gravity of soil groups

5.3.2 Particle size distribution

Typical particle size distribution curves in each of the soil groups are given in Figure 5.3. As expected, the immature soils are coarser and contain lesser fractions of fines. The mature soils can be made up of up to 40 to 50% of clay particles.

5.3.3 Plasticity

The consistency limits of the different soil groups have been plotted in Figure 5.4. The majority of the data points plot below the A-line and the soils are classified as silts although the particle size analyses show in some cases significant clay contents. This is probably due to the breakdown of cemented particles during oven-drying of the soils prior to the wet sieving stage for particle size analysis. On the other hand, for plasticity tests, the soils undergo a process of air-drying only so that there is only partial breakdown of the cementation bonds among particles.

The swelling and shrinkage potential of a soil is influenced by its plasticity in terms of its plasticity index. Figure 5.5 shows that there is a good correlation between linear shrinkage (LS) and plasticity index (I_p) which is given by equation 5.1. However, Whitlow (2001) gives the slightly different equation, $I_p = 2.13 \times LS$.

$$I_p = 2.0 \times LS \tag{5.1}$$

According to Chen (1975), the swelling potential can be related to the linear shrinkage, that is, the shrinkage characteristics of a soil can be a useful index in determining the swelling potential. Table 5.3 gives a means of using linear shrinkage to make a qualitative assessment on the swelling potential of a soil.

Linear shrinkage (%)	Swelling potential
> 8	Critical
5 - 8	Marginal
0 - 5	Non - critical

Table 5.3 Relationship between linear shrinkage and swelling potential (Chen, 1975)

Other methods for assessing the swelling potential of soils on the basis of plasticity index and clay content are given by Seed et al (1962) and Williams and Donaldson (1973).

5.3.4 In-situ void ratio

The void ratio of a soil is one of the factors which influence its strength, compressibility and permeability. Figures 5.6 and 5.7 show typical variations of moisture contents and void ratios with depths at some sites. It is found that there is no apparent relationship between moisture content or void ratio and depth as is generally the case in sedimentary soils. However, moisture contents are generally 10 to 20% higher than the plastic limits and void ratios are very high, being greater than 1 but less than 2.

5.3.5 Compressive strength and strength parameters

Because of the heterogeneity in the process of soil formation and in the field moisture content, it is expected that compressive strength of the soil will vary widely. The range of strengths recorded from laboratory unconfined compression tests are shown in Figure 5.8. The compressive strength varies mostly between 50 and 300 kN/m^2. It has been found that the pocket penetrometer is a useful simple device for measuring compressive strength in the field.

Consolidated-undrained triaxial tests have been carried out on saturated samples to provide effective stress parameters. Average values of these parameters are given in Table 5.4. It must be noted that there is no available data on the latosolic red prairie soil group.

	Effective strength parameters	
Soil group	c' (kN/m^2)	ϕ' (^{o})
Brown forest latosolic soils	17	29
Low humic latosols	21	33
Humic latosols	13	36
Humic ferruginous latosols	10	31

Table 5.4 Average effective strength parameters of residual soils

The effective angle of shearing resistance (ϕ') can also be estimated indirectly from Standard Penetration Test N values; ϕ' can be obtained from the chart of Peck et al (1967). Figure 5.9 shows a typical variation of N values with depth from which bearing capacities are calculated.

5.3.6 Compressibility

The secant modulus of elasticity determined from the undrained compression tests has been plotted against compressive strength in Figure 5.10. There is a good correlation between modulus of elasticity and compressive strength. Being given that undrained shear strength is given by half of compressive strength, the ratio of undrained modulus of elasticity (E_u) to shear strength (c_u) is given by Equation 5.2.

$$E_u = 90\ c_u \tag{5.2}$$

The drained modulus of elasticity can be estimated from SPT N values by using the following relationship recommended by Stroud (1989) for normally consolidated sand:

$$E' = 1000N\ (kN/m^2) \text{ where N is the SPT blow count.} \tag{5.3}$$

One-dimensional consolidation test results have been also compiled and relationship between compression index (C_c) and initial void ratio (e_o) is given in Figure 5.10. It is to be noted that C_c gives the compressibility in the normal consolidation pressure range of the soil. Figure 5.11 shows that C_c does not have a strong linear correlation with e_o. Further work is required to filter the data in order to arrive at better correlations.

5.3.7 Characteristics of compacted soils

Figures 5.12 and 5.13 have been compiled after the soils have been grouped in terms of their plasticity index. Only soils with more than 80% of fines (silt and clay sizes) have been selected. As a result good correlations have been obtained between maximum dry density (MDD) and liquid limit and between optimum moisture content (OMC) and liquid limit. These charts are useful for a preliminary estimate of MDD and OMC for compaction works.

Figure 5.14 (a), (b) and (c) gives some charts where the 4-day soaked California Bearing Ratio (CBR) can be estimated on the basis of moisture content and dry density. These charts are useful for preliminary road pavement design.

5.4 Characteristics of dark magnesium clay

The dark magnesium clay is formed under conditions of poor drainage and low rainfall from the degradation of mountain slopes. The clay is formed through a process of extreme disintegration involving oxidation and strong hydration. The process takes place with limited leaching which favours the accumulation of magnesium cations. The warm temperature and alkaline environment lead to the formation of montmorillonite minerals. Montmorillonitic clays exhibit high swelling/shrinkage potentials. The dark magnesium clays of Mauritius are classified as of very high to extremely high plasticity (Figure 5.15). The activity, which is the ratio of plasticity index to % clay content, is between 1 and 1.3 so that the soil is classified as normal to active.

Statistical information on specific gravity values are given in Table 5.5. It is noted that the specific gravity of the dark magnesium clay is lower than for the residual soils.

Minimum	2.68
Maximum	2.85
Mean	2.76
Median	2.77
Standard deviation	0.05
Coefficient of variation	0.02

Table 5.5 Statistical information on the specific gravity of dark magnesium clays

The variation of natural moisture content with depth within the dark magnesium clay deposits is given in Figure 5.16. The data points represent results from 5 different sites obtained at random times of the year. At these sites, the water table is more than 10 metres deep so that it has not influenced the moisture content of the soil samples. In the natural ground, the moisture content of a partially saturated soil is in general equilibrium with the applied stress, the forces due to evaporation and transpiration at the ground surface and the capillary forces. The change in moisture content can be due to surface water infiltration as well as movement of moisture from the moister soil to the drier soil. Therefore, the moisture content keeps on fluctuating all the time. The purpose of Figure 5.16 is to give an example of the range of variation in moisture content; envelopes of the maximum and minimum moisture contents which were measured are drawn in order to give an indication of the possible range of moisture content fluctuations. As expected, the fluctuation is largest at shallower depths.

Fluctuations in the moisture content of the clay result in volume changes of the soil. If the soil is constrained by a structure, stresses will be set up against that structure. Laboratory investigation results showing the variation of the constant volume swell pressure with initial moisture content of the clay, which is allowed to soak in water, are given in Figure 5.17. This figure allows the swell pressure to be estimated when the natural moisture content is known. It can be found that swell pressures in excess of 100 kPa can be generated.

Chan Chim Yuk and Dabee (1997) showed the benefits of lime stabilisation on the properties of the dark magnesium clay. Addition of 4% lime gave the following improvements:

- Reduction in plasticity index
- Reduction in swell pressure
- Significant increase in CBR .

Seeboo (1998) found that lime stabilization had a more significant effect than cement on the engineering properties of the clay.

Effective shear strength parameters of the clay from a number of sites are given in Table 5.18. The low angle of shearing resistance (6 to 12 ^{o}) must be noted.

Strength parameters	Source of data
$c' = 10\ kN/m^2$ $\phi' = 10^o$	La Butte landslide back analysis
$c' = 0\ kN/m^2$ $\phi' = 12^o$	Guibies valley landslide back analysis
$c' = 14\ kN/m^2$ $\phi' = 8^o$	CU (multistage) test result from PL Ring Road Project
$c' = 15\ kN/m^2$ $\phi' = 6^o$	CU (multistage) test on specimen from Bell Village
$c' = 55\ kN/m^2$ $\phi' = 6^o$	CU (multistage) test on specimen from Bell Village
$c' = 38\ kN/m^2$ $\phi' = 8^o$	CU (multistage) test on specimen from Colline Monneron

Table 5.6 Shear strength parameters of dark magnesium clay

The natural void ratio of the clay is 0.8 to 1.3 with a low compression index of 0.1 to 0.2.

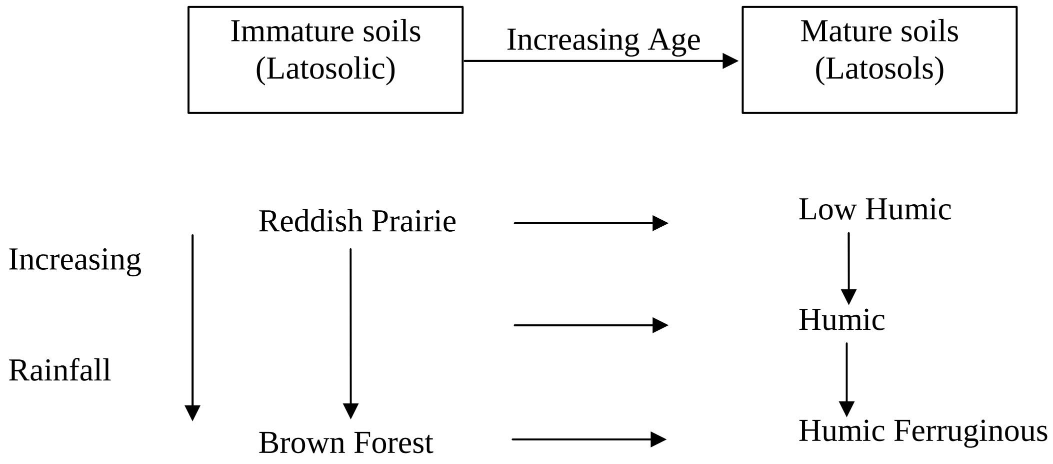


Figure 5.1 Soil classification according to MSIRI Soil Map of Mauritius
(Parish and Feillafe, 1965)

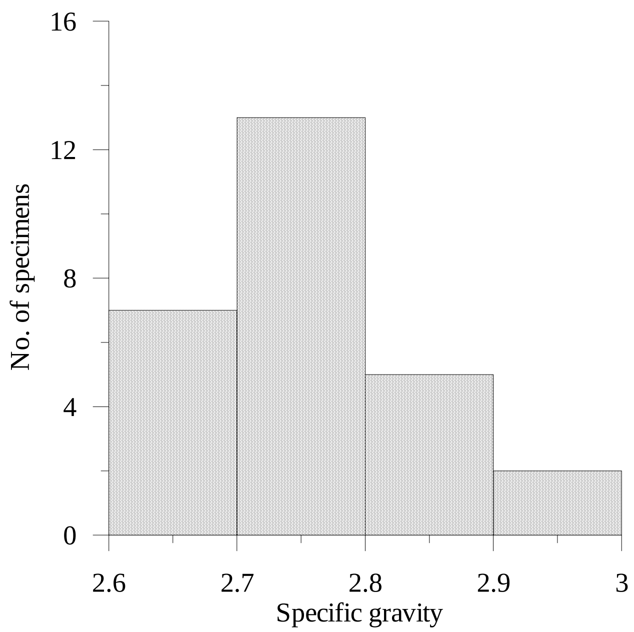


Figure 5.2 (a) Histogram of specific gravity of latosolic red prairie soils

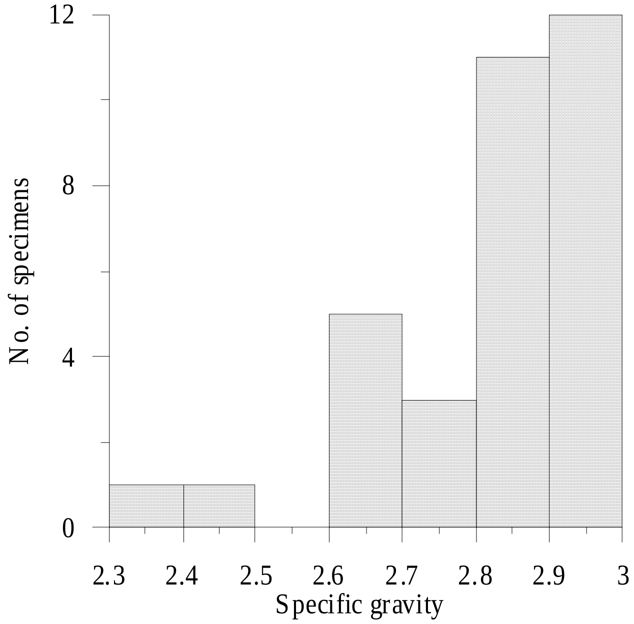


Figure 5.2 (b) Histogram of specific gravity of latosolic brown forest soils

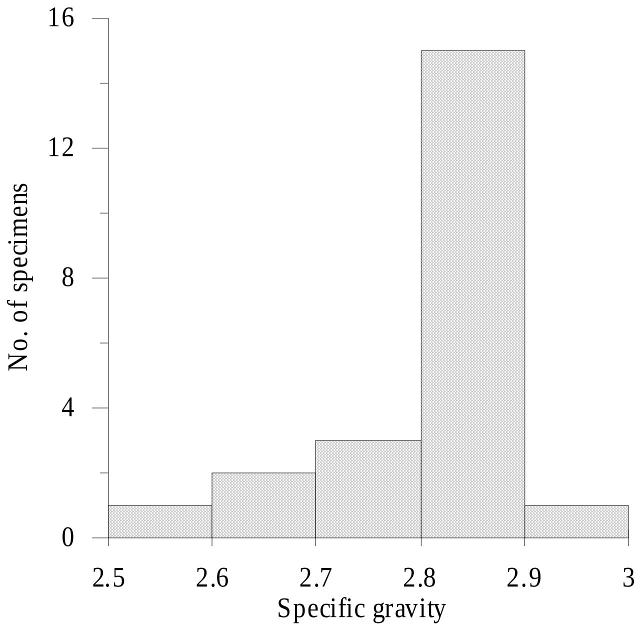


Figure 5.2 (c) Histogram of specific gravity of low humic latosols

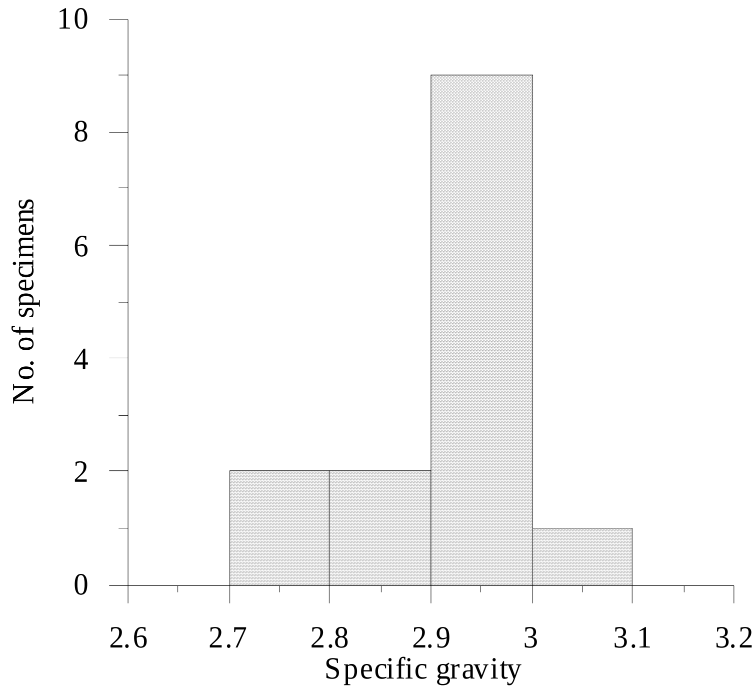


Figure 5.2 (d) Histogram of specific gravity of humic latosols

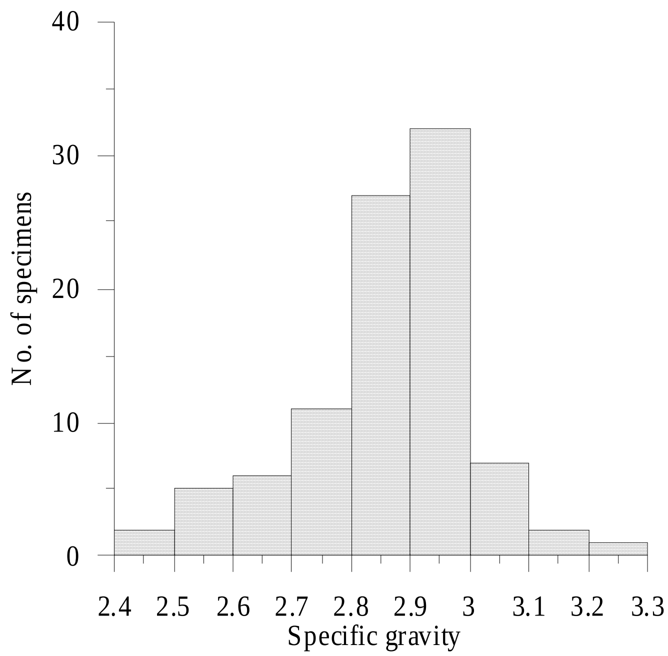


Figure 5.2 (e) Histogram of specific gravity of humic ferruginous latosols

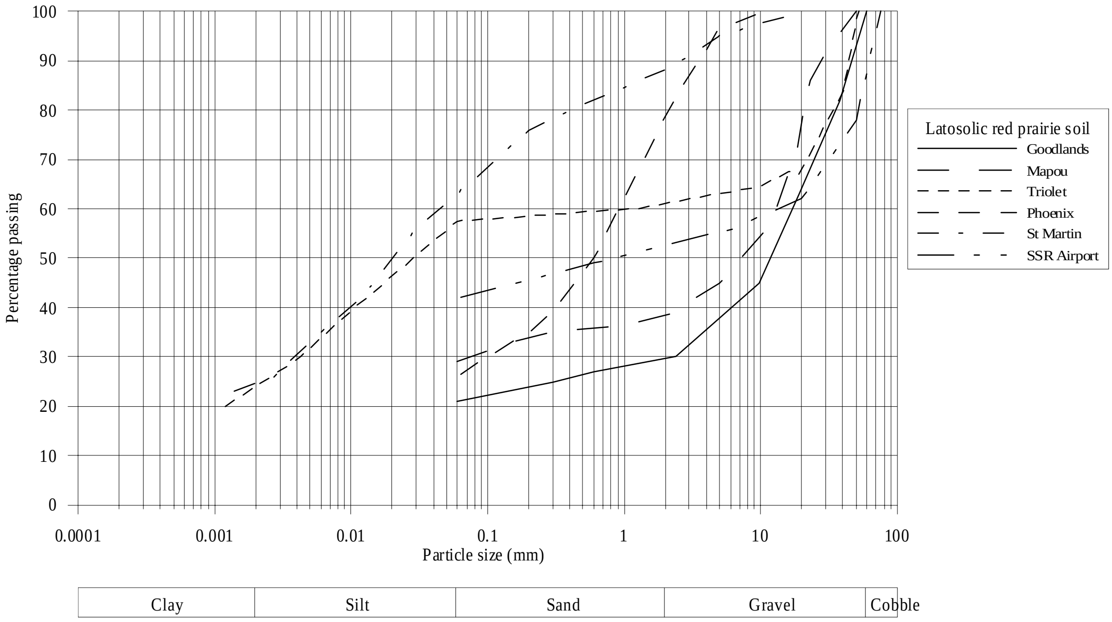


Figure 5.3 (a) Particle size distribution of latosolic red prairie soils

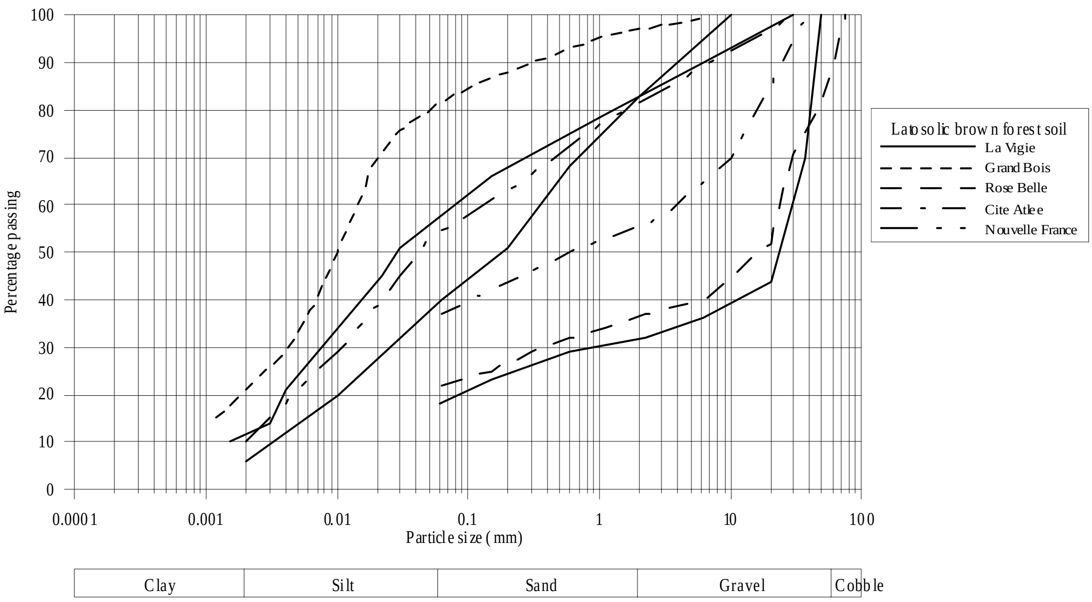


Figure 5.3 (b) Particle size distribution of latosolic brown forest soils

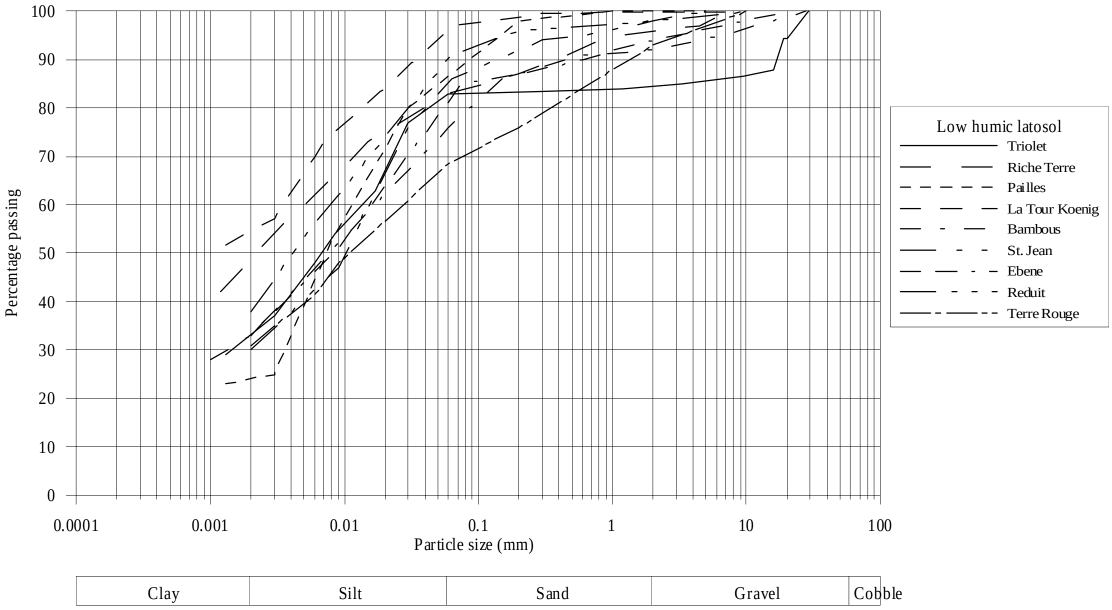


Figure 5.3 (c) Particle size distribution of low humic latosols

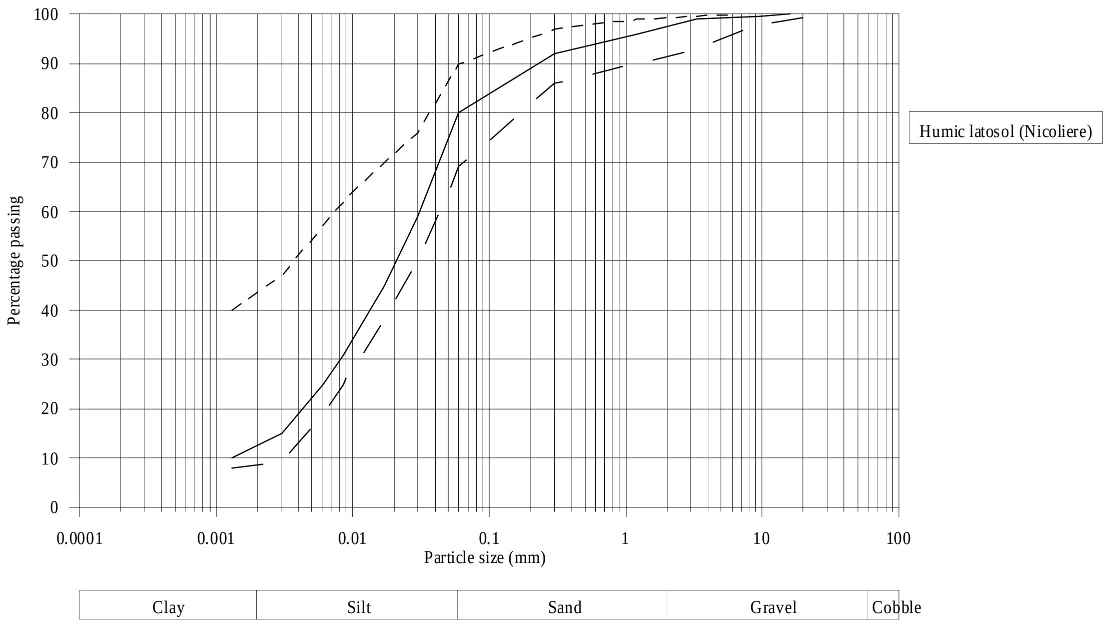


Figure 5.3 (d) Particle size distribution of humic latosols

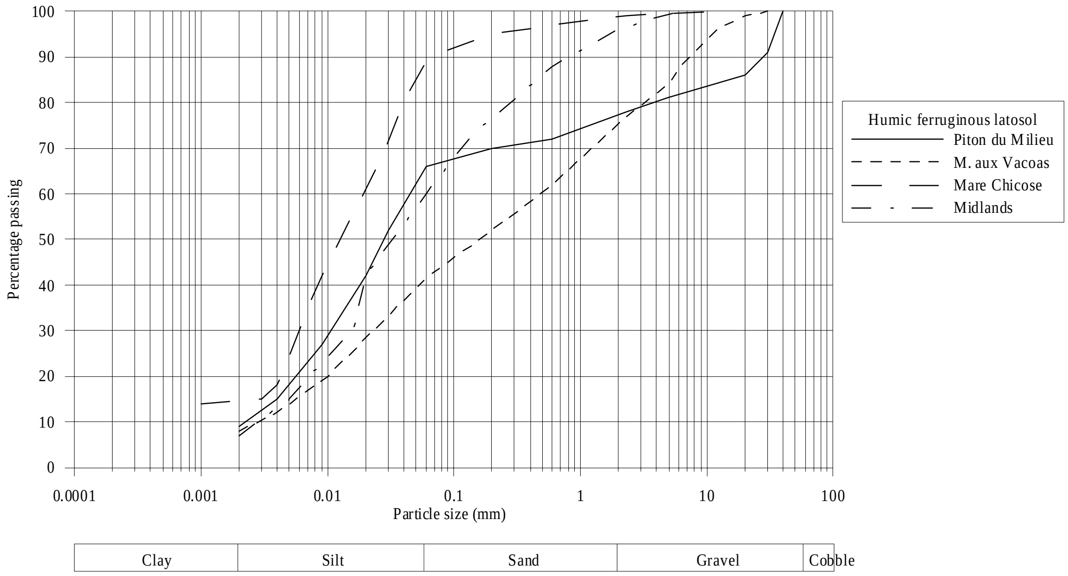


Figure 5.3 (e) Particle size distribution of humic ferruginous latosols

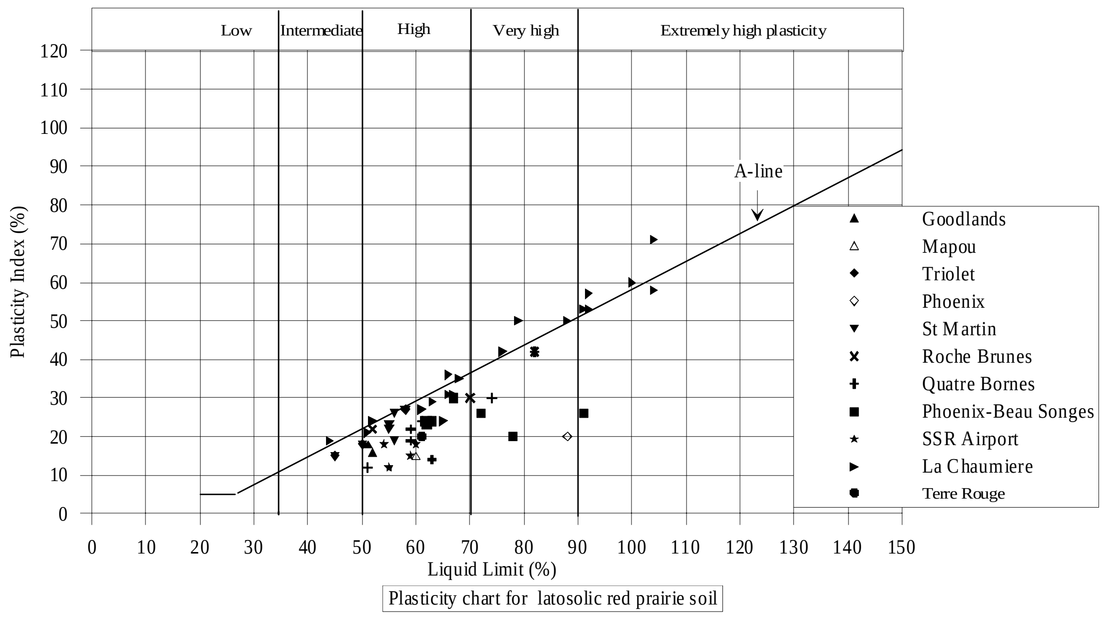


Figure 5.4 (a) Plasticity of latosolic red prairie soils

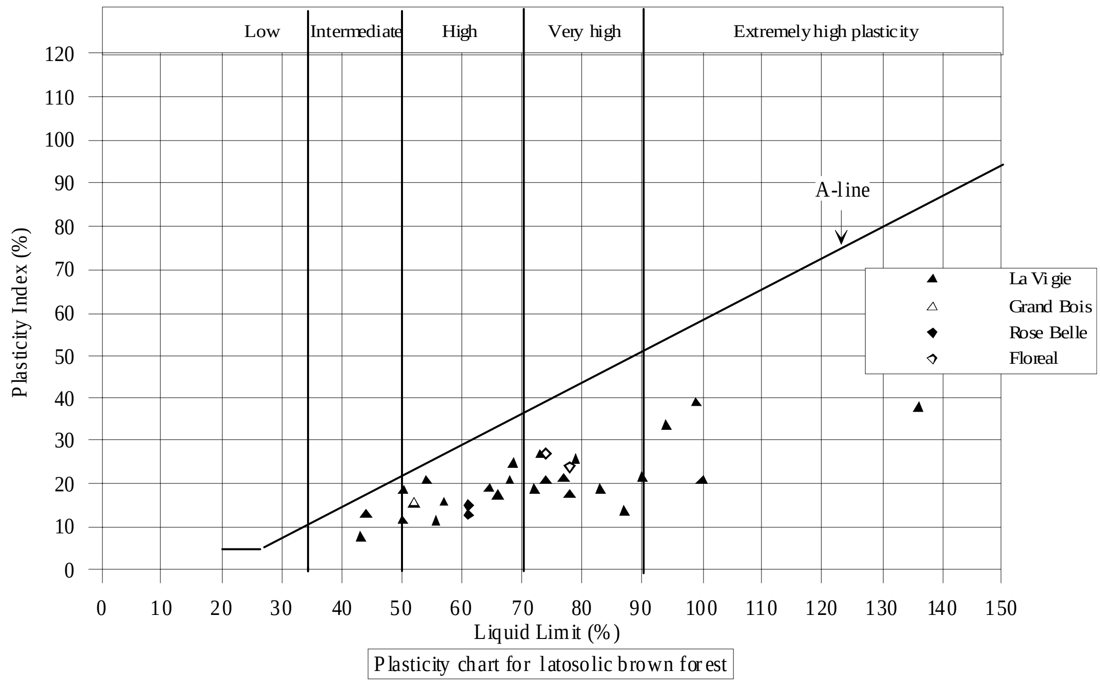


Figure 5.4 (b) Plasticity of latosolic brown forest soils

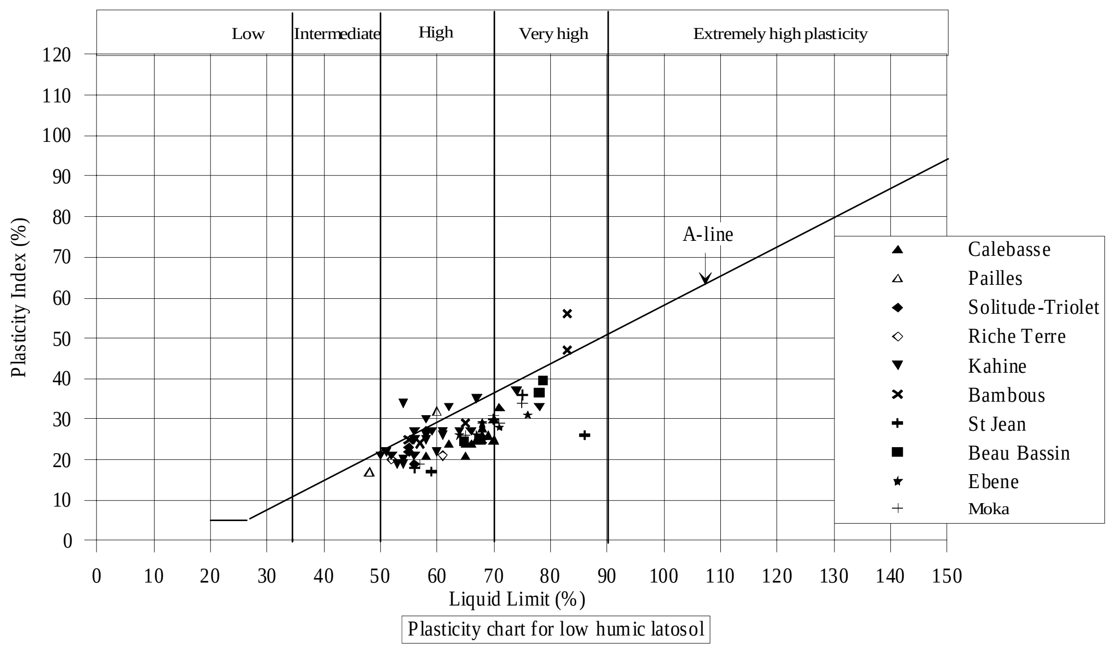


Figure 5.4 (c) Plasticity of low humic latosols

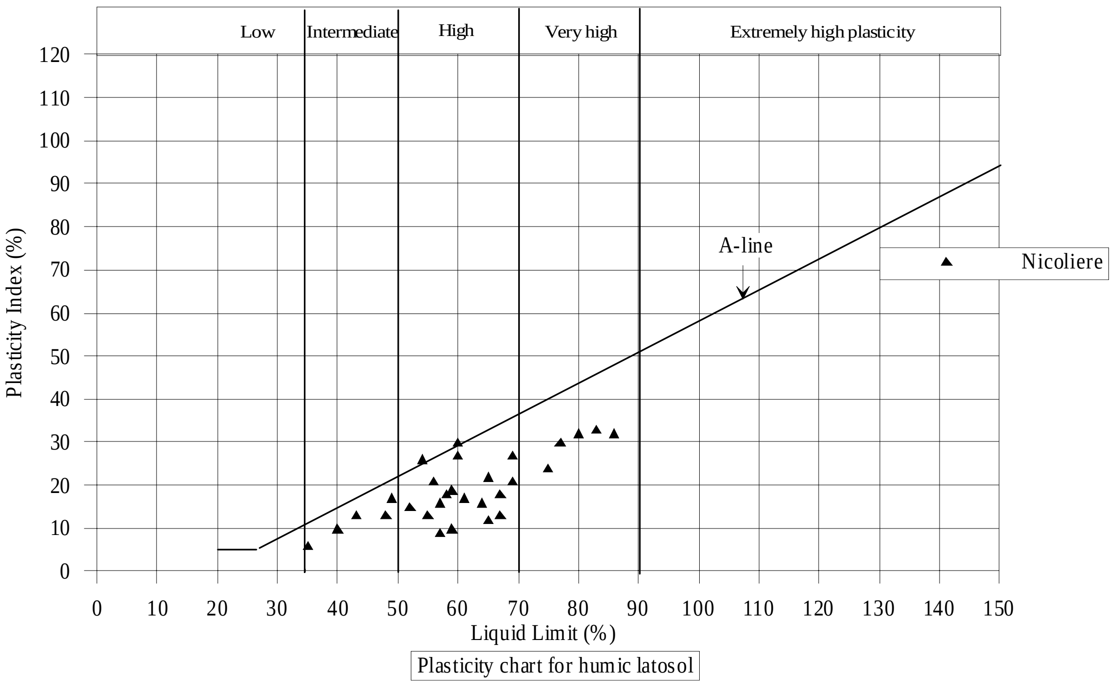


Figure 5.4 (d) Plasticity of humic latosols

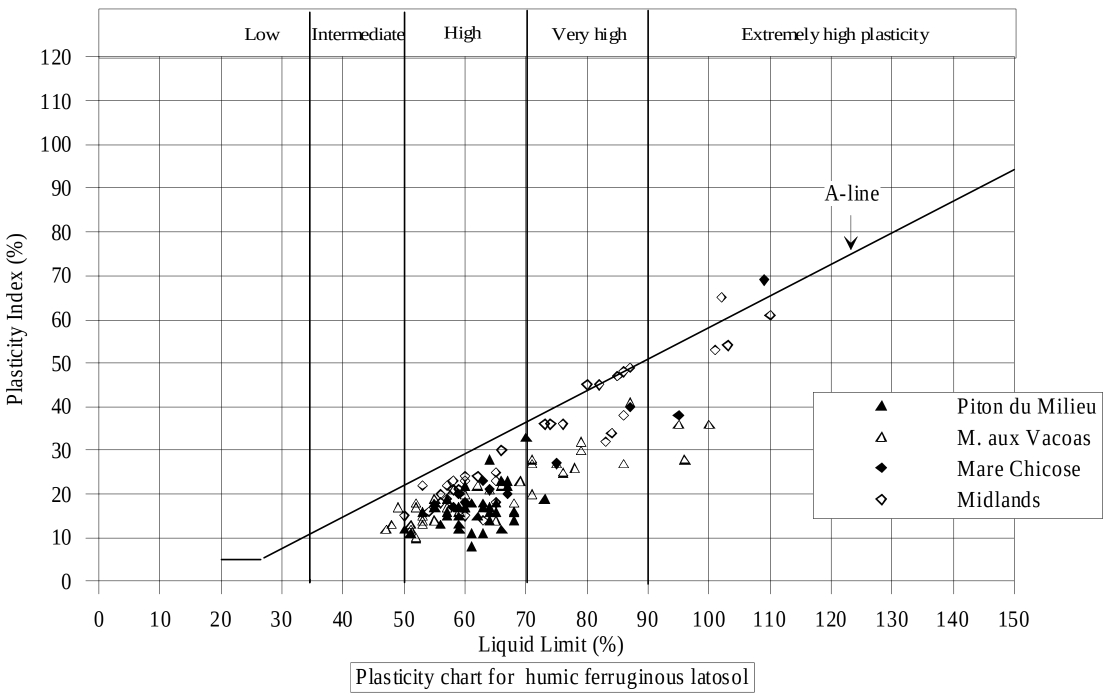


Figure 5.4 (e) Plasticity of humic ferruginous latosols

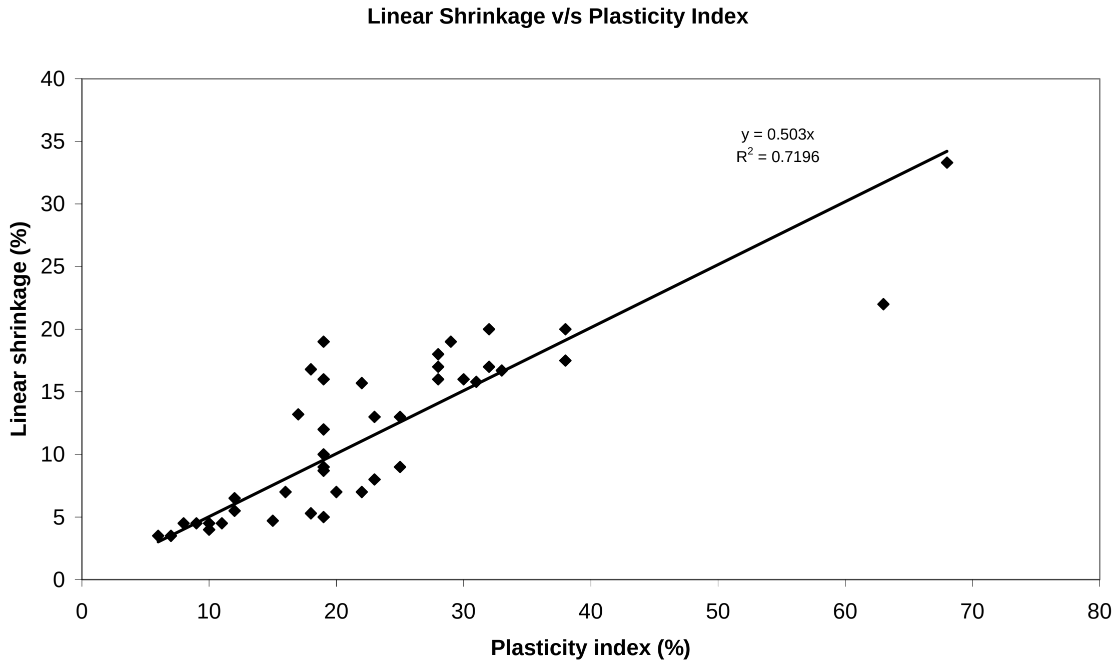


Figure 5.5 Relationship between linear shrinkage and plasticity index

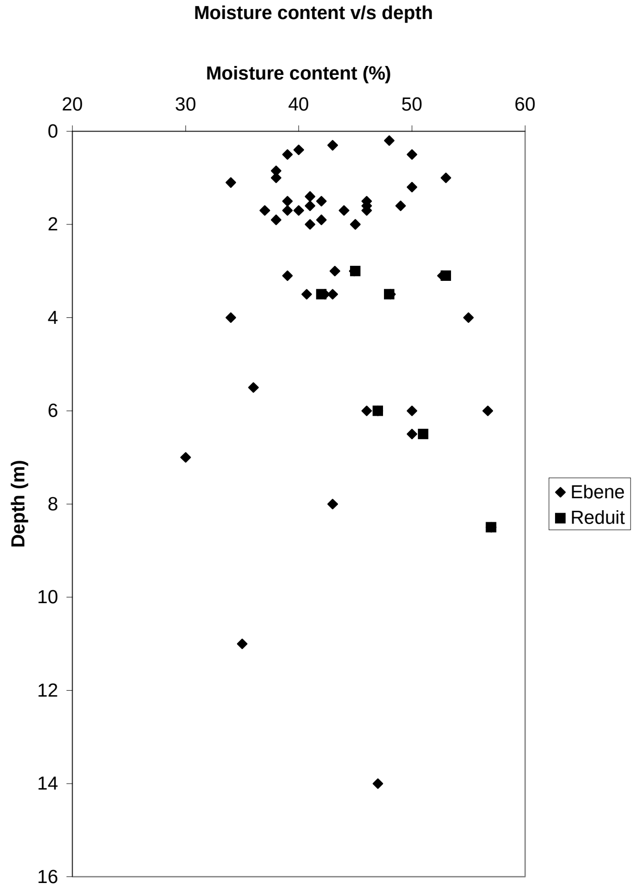


Figure 5.6 Typical variation of moisture content with depth

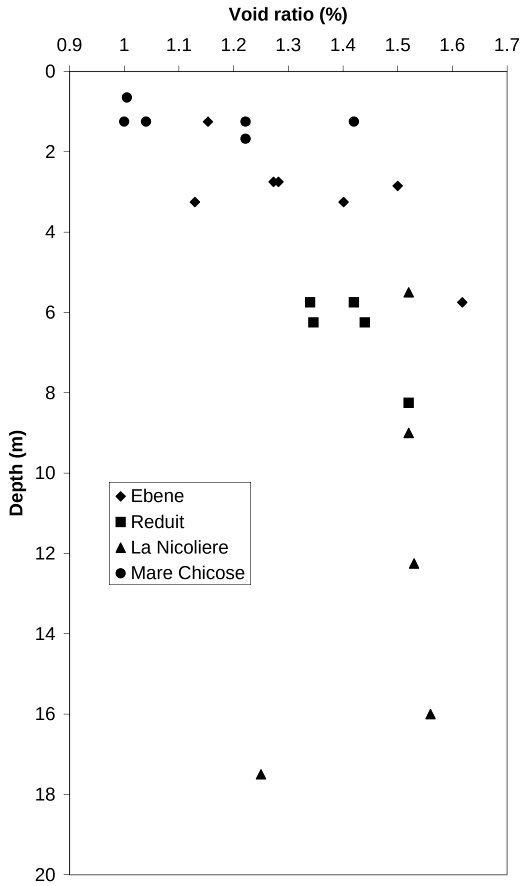


Figure 5.7 Typical variation of void ratio with depth

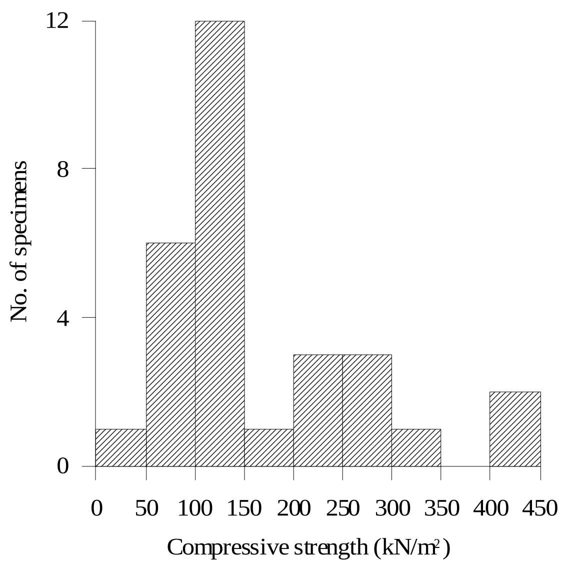


Figure 5.8 Histogram of compressive strength of residual soils

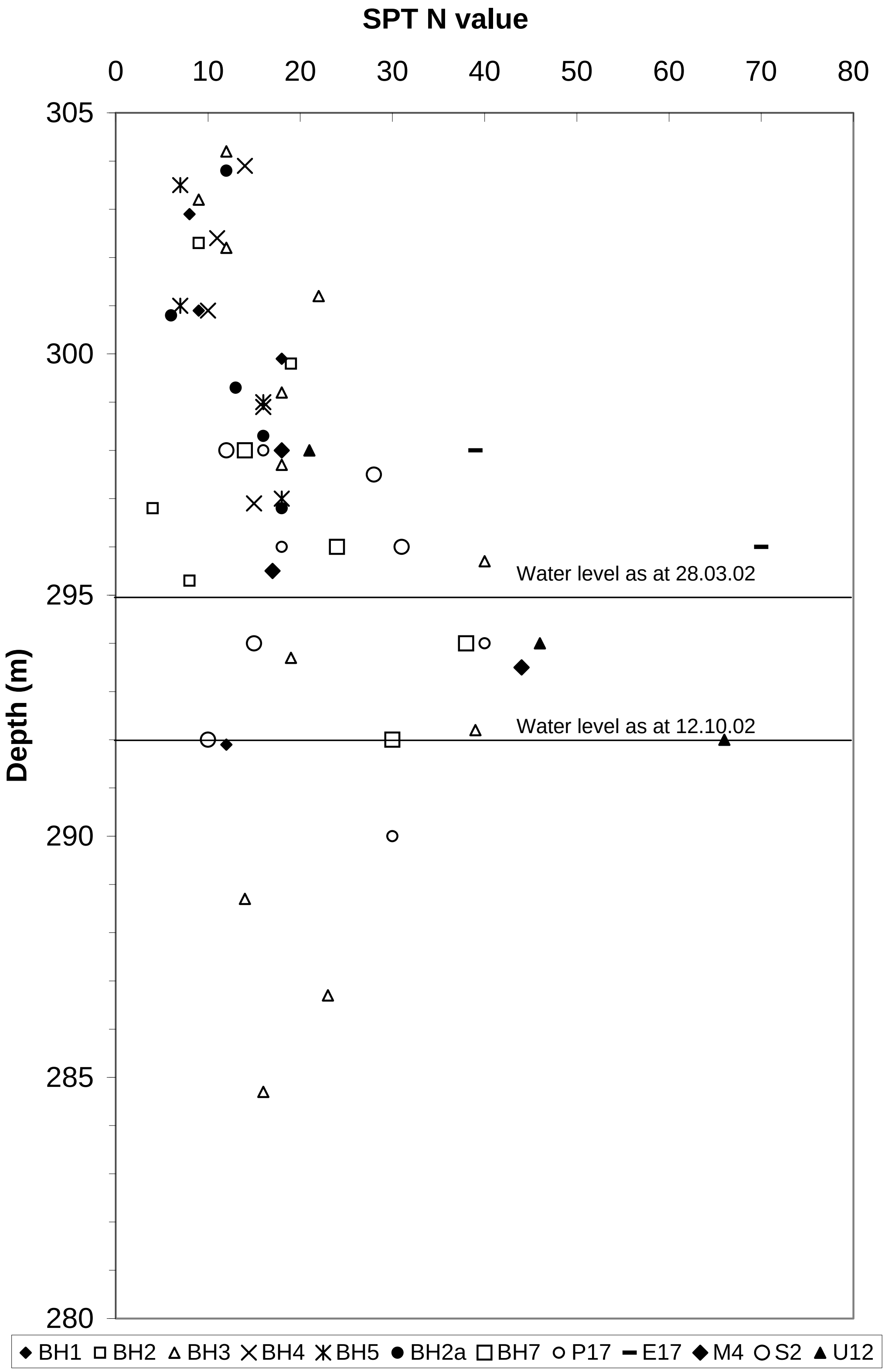


Figure 5.9 Variation of consistency of soil as measured by SPT at Ebene

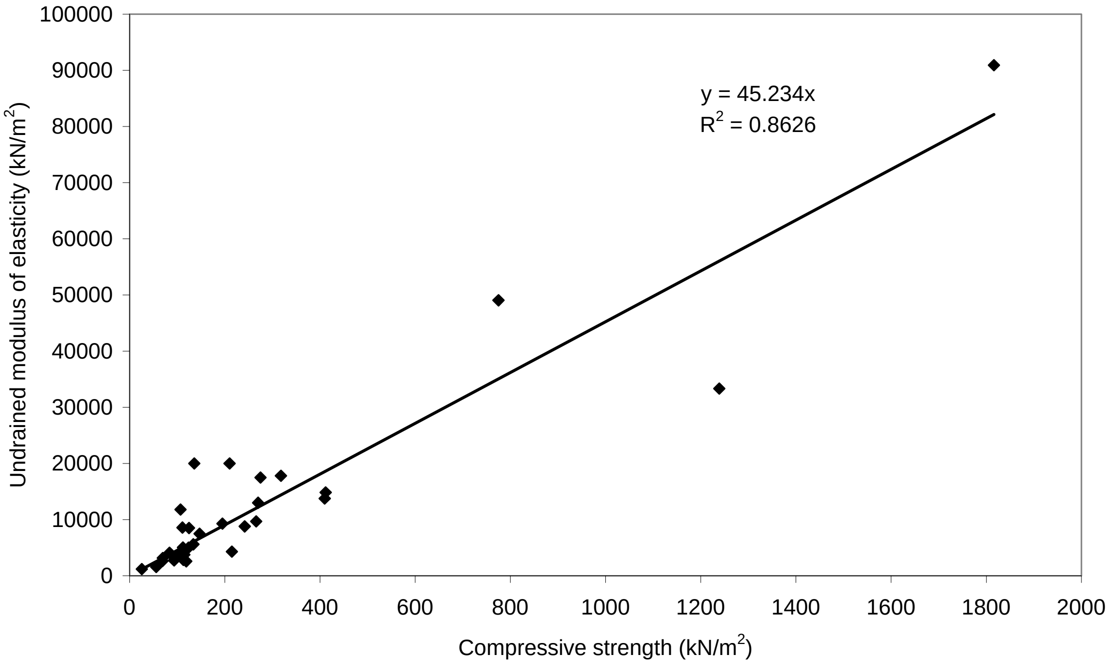


Figure 5.10 Correlation between undrained modulus of elasticity and compressive strength of residual soils

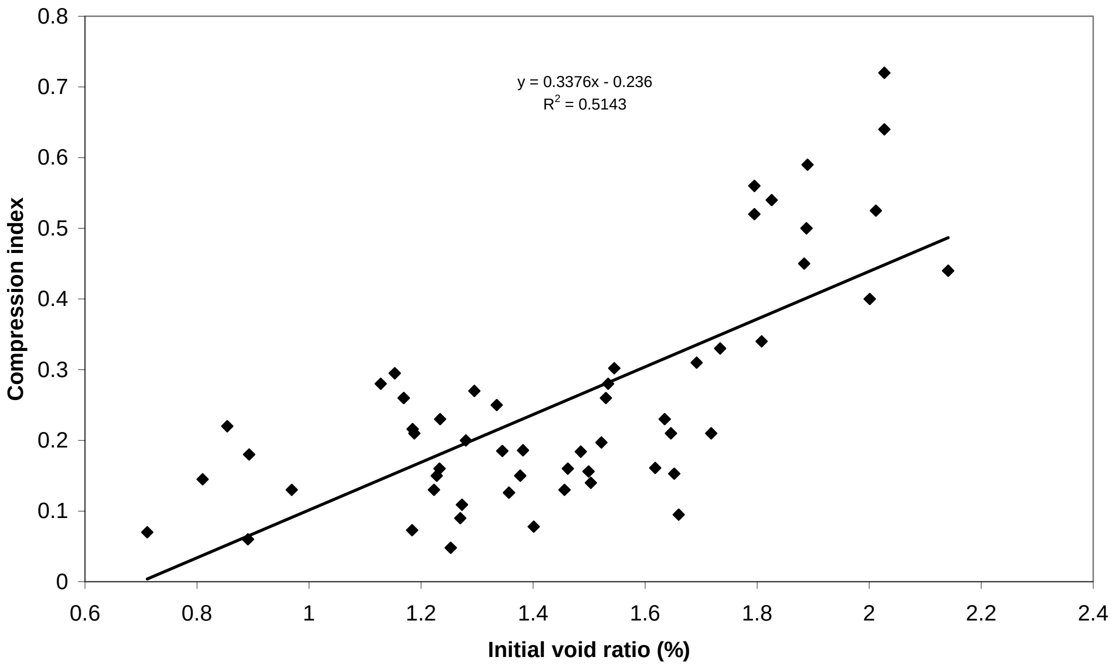


Figure 5.11 Correlation between compression index and initial void ratio of residual soils

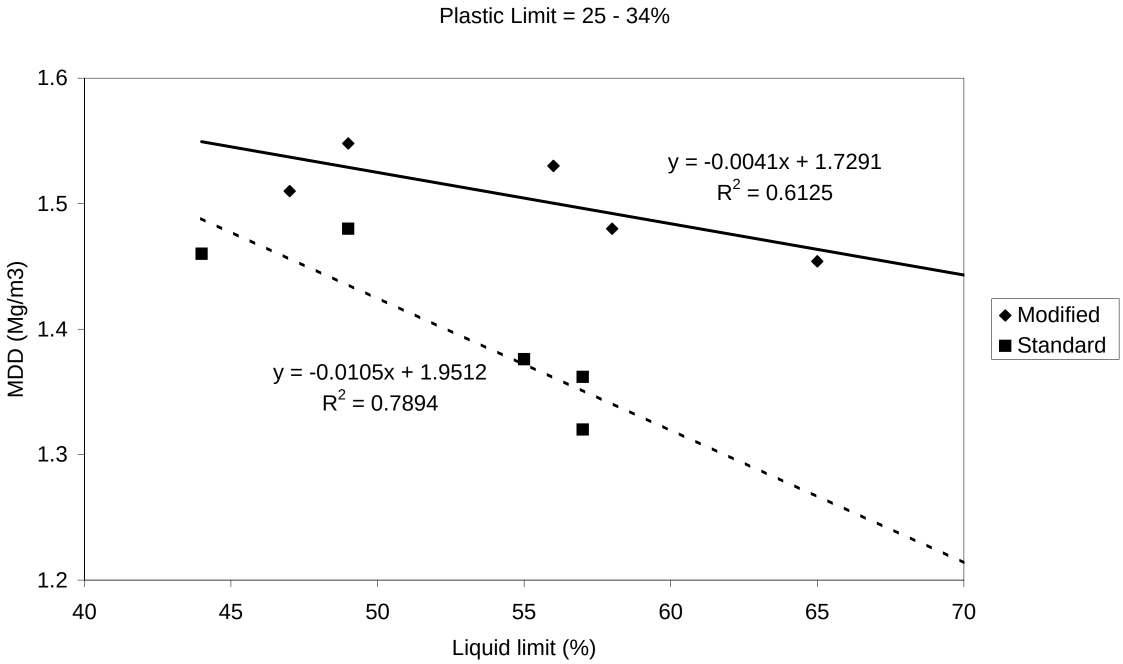


Figure 5.12 (a)

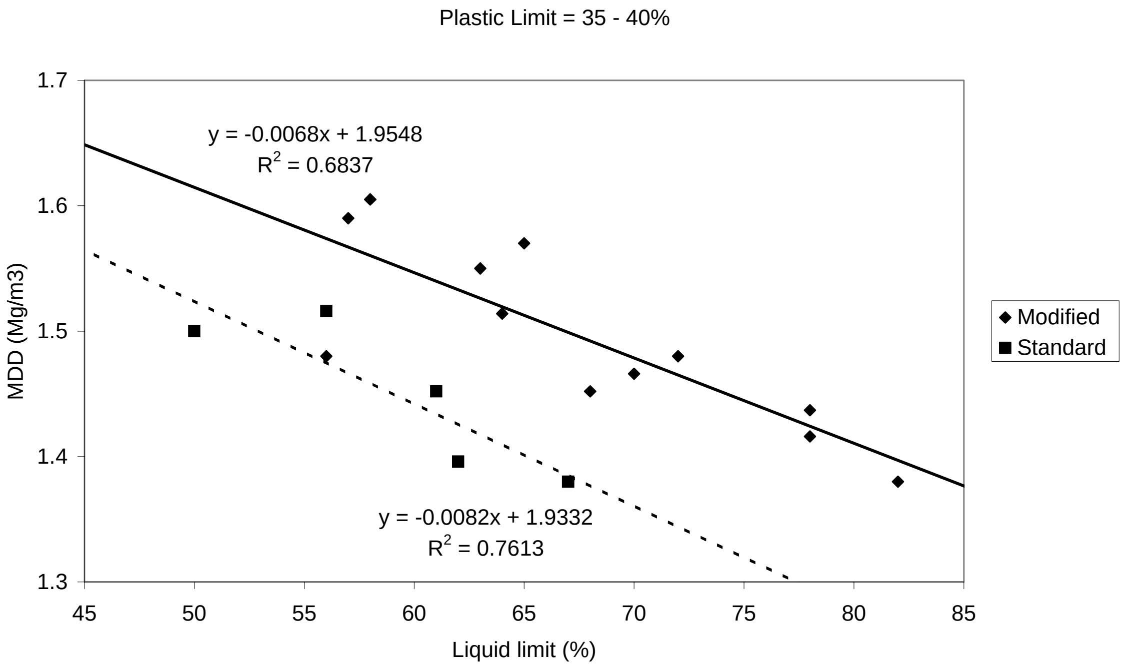
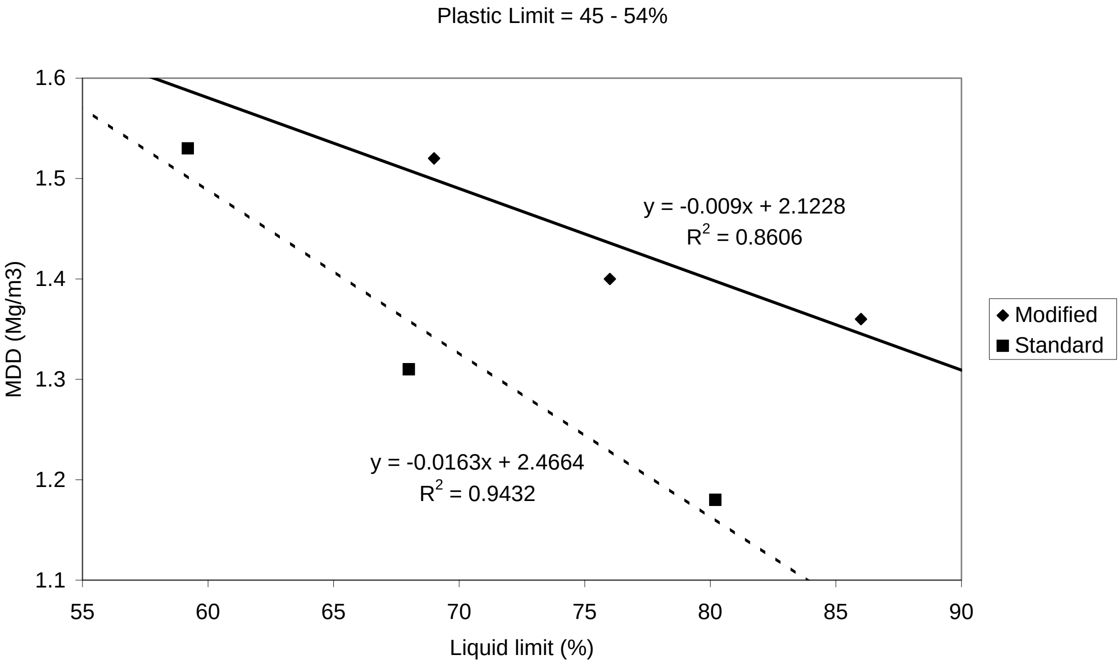


Figure 5.12 (b)



(c)

Figure 5.12 Relationship between maximum dry density and liquid limit of residual soils

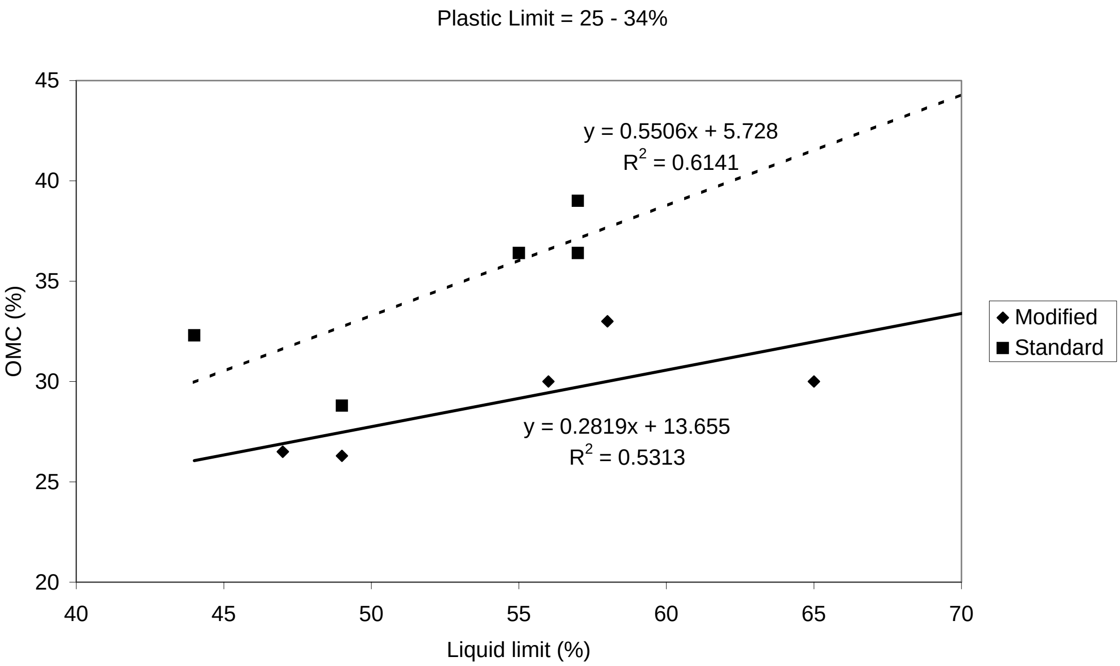


Figure 5.13 (a)

Plastic Limit = 35 - 44%

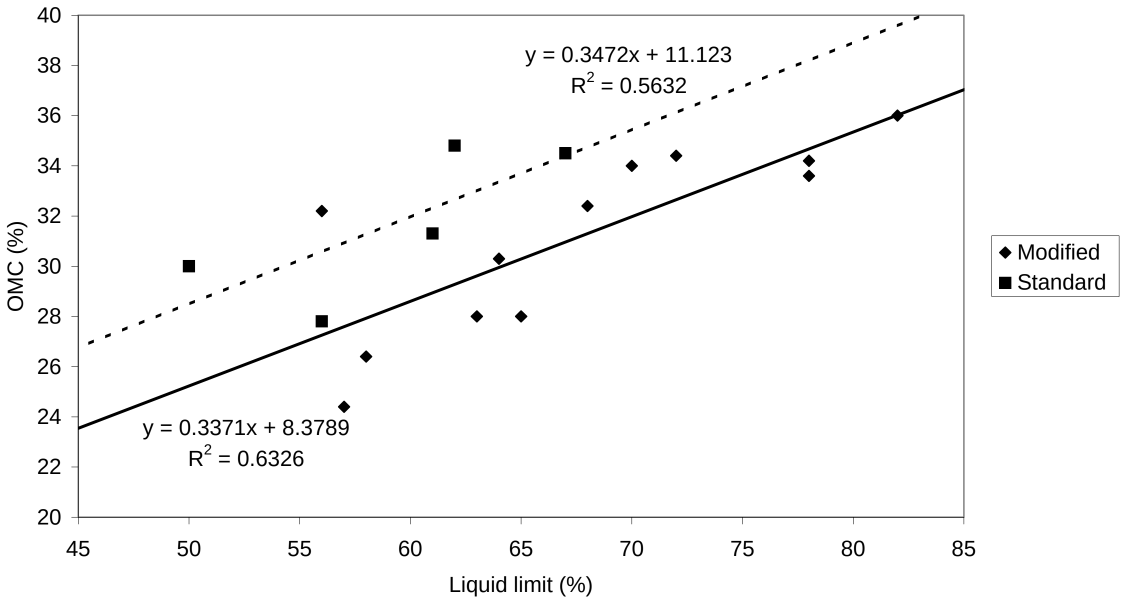
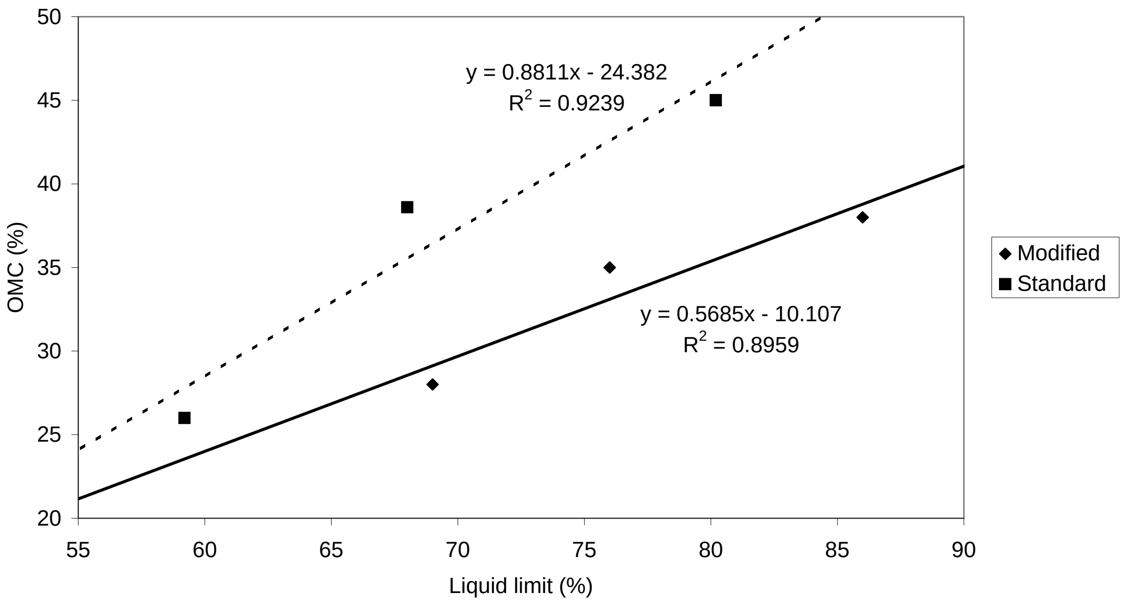


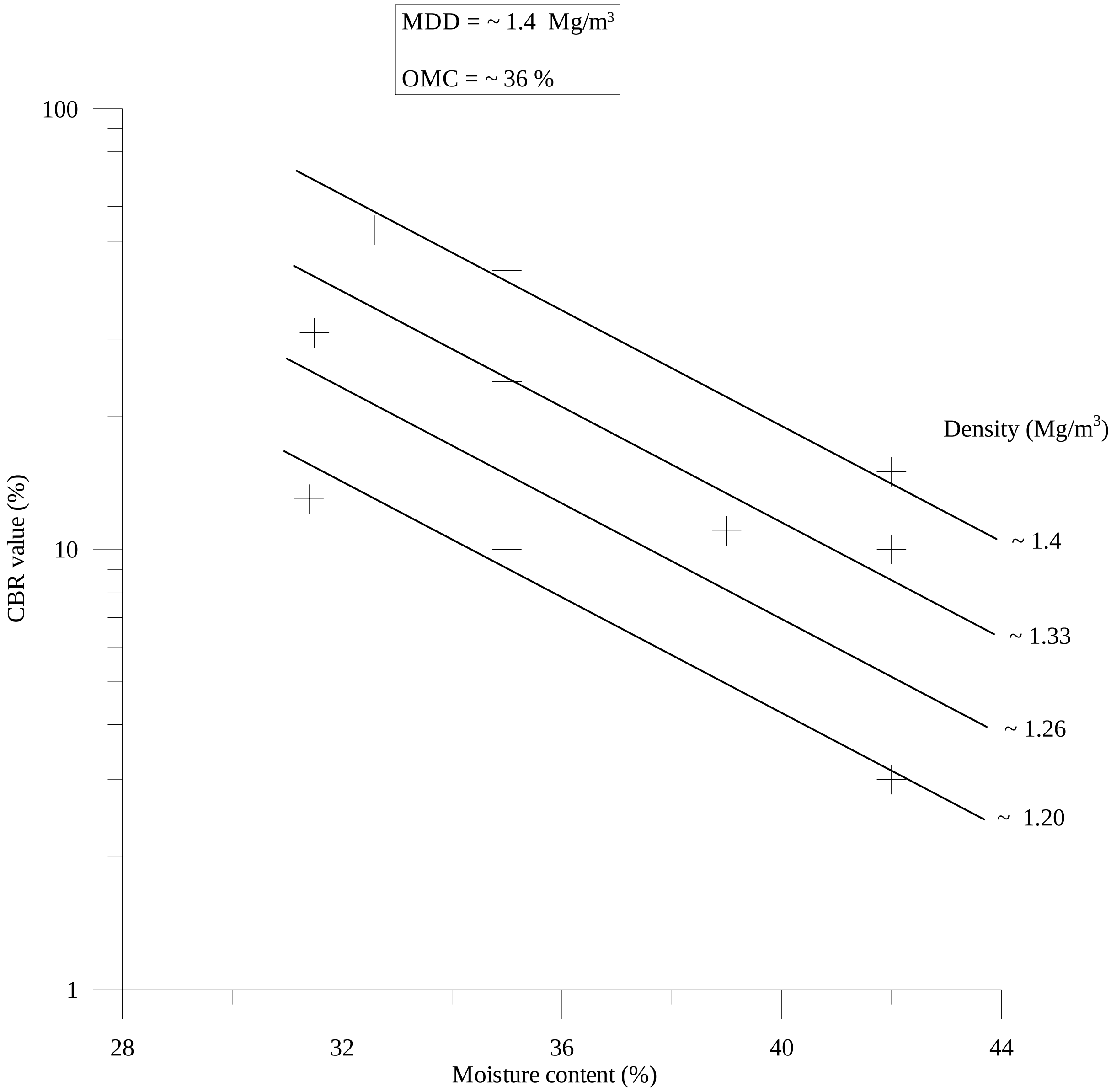
Figure 5.13 (b)

Plastic Limit = 45 - 54%



(c)

Figure 5.13 Relationship between optimum moisture content and liquid limit of residual soils



MDD = ~ 1.4 Mg/m3
OMC = ~ 36 %
100
10
1
CBR value (%)
Density (Mg/m3)
~ 1.4
~ 1.33
~ 1.26
~ 1.20
28
32
36
40
44
Moisture content (%)

Figure 5.14 (a)

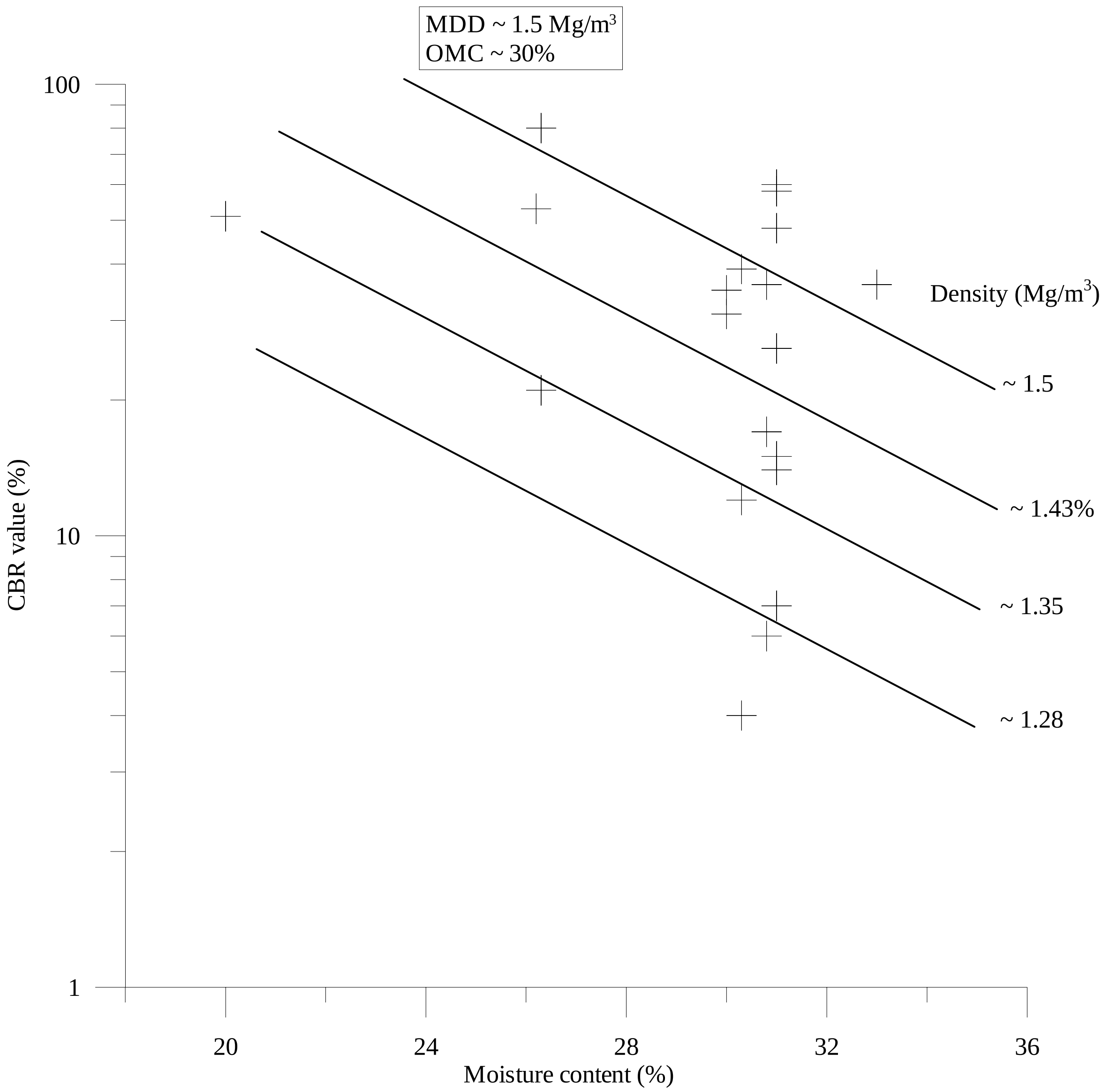
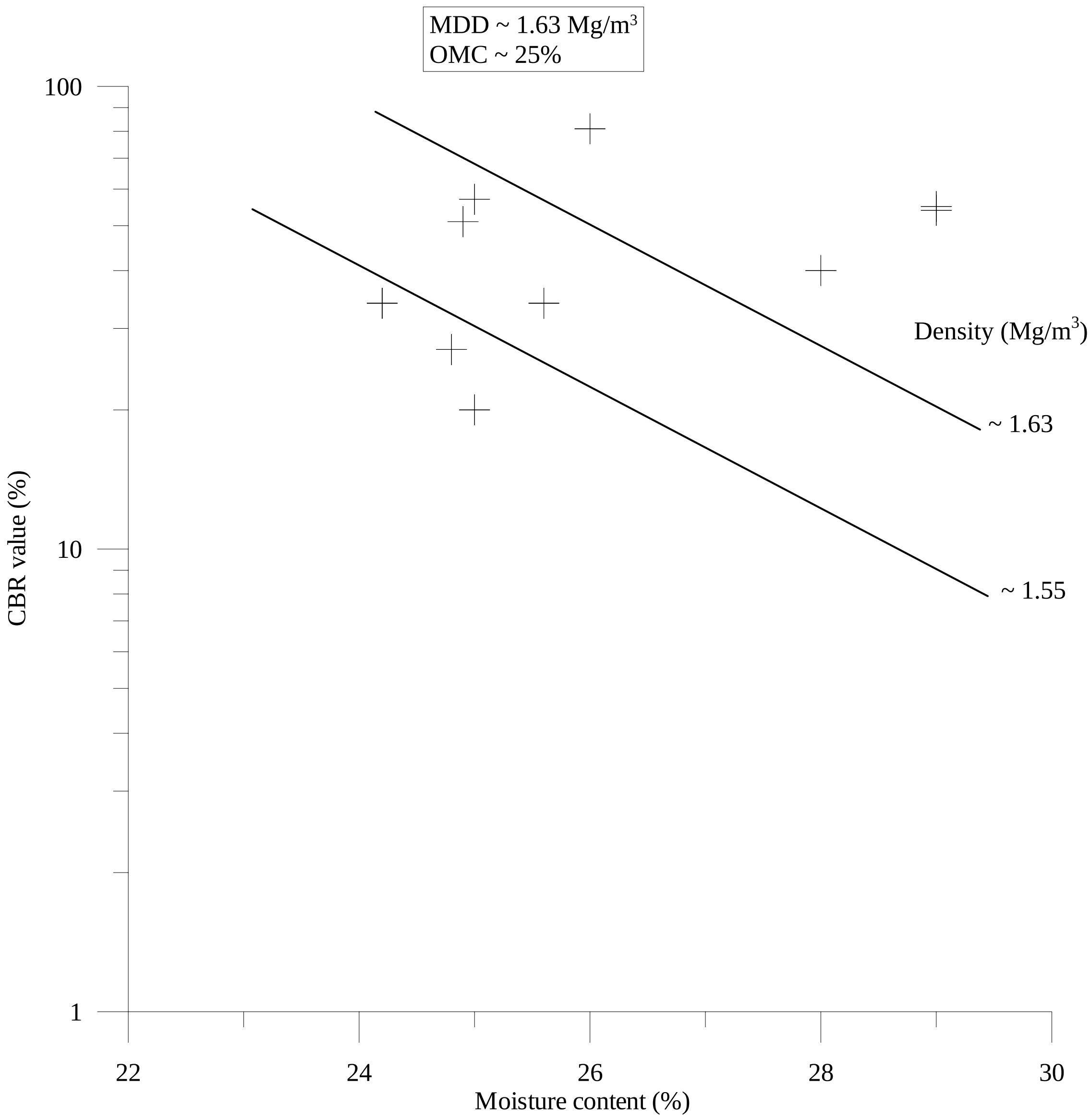

MDD ~ 1.5 Mg/m3
OMC ~ 30%
100
10
1
CBR value (%)
20
24
28
32
36
Moisture content (%)
Density (Mg/m3)
~ 1.5
~ 1.43%
~ 1.35
~ 1.28

Figure 5.14 (b)



(c)

Figure 5.14 Variation of 4-day soaked CBR of residual soils with moisture content

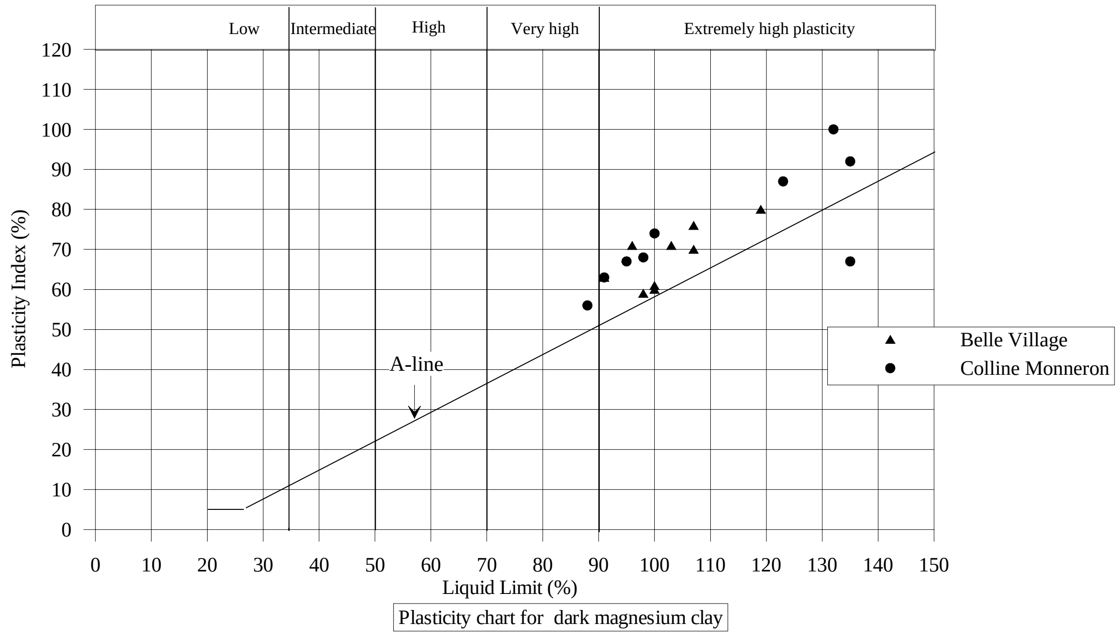


Figure 5.15 Plasticity of dark magnesium clay

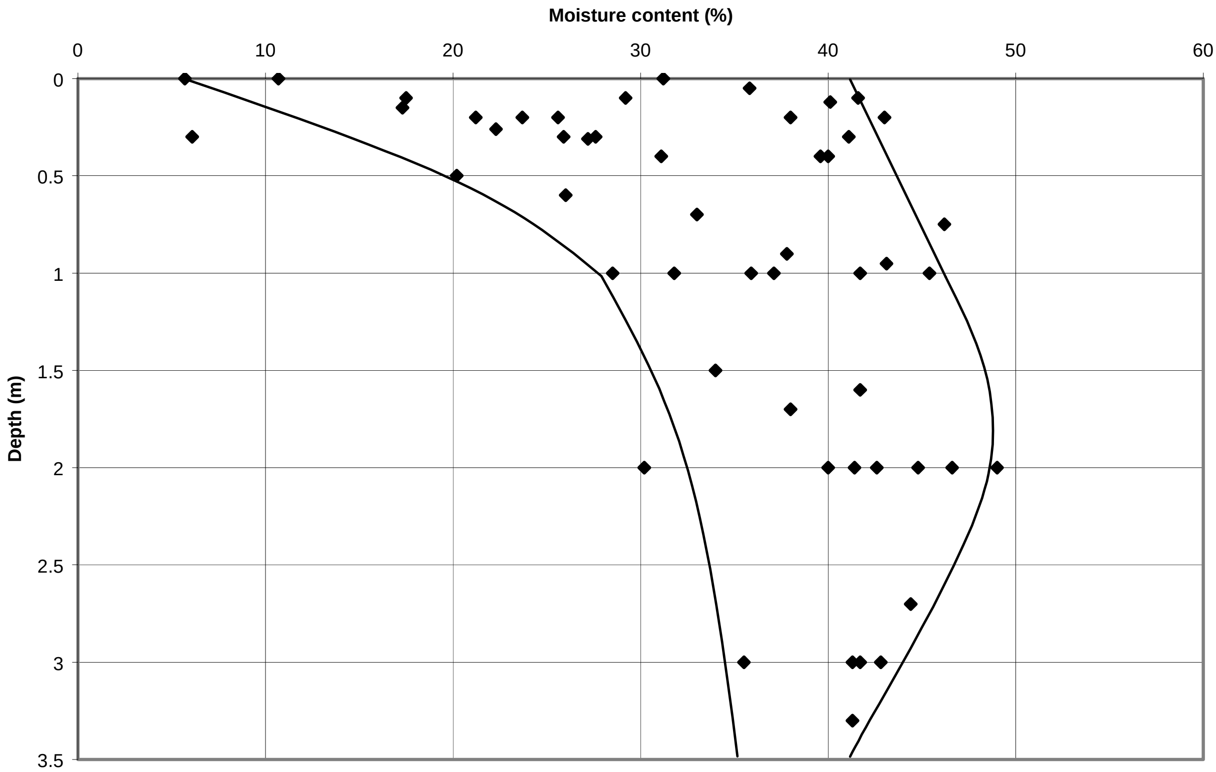


Figure 5.16 Variation of natural moisture content with depth of dark magnesium clay measured at five locations

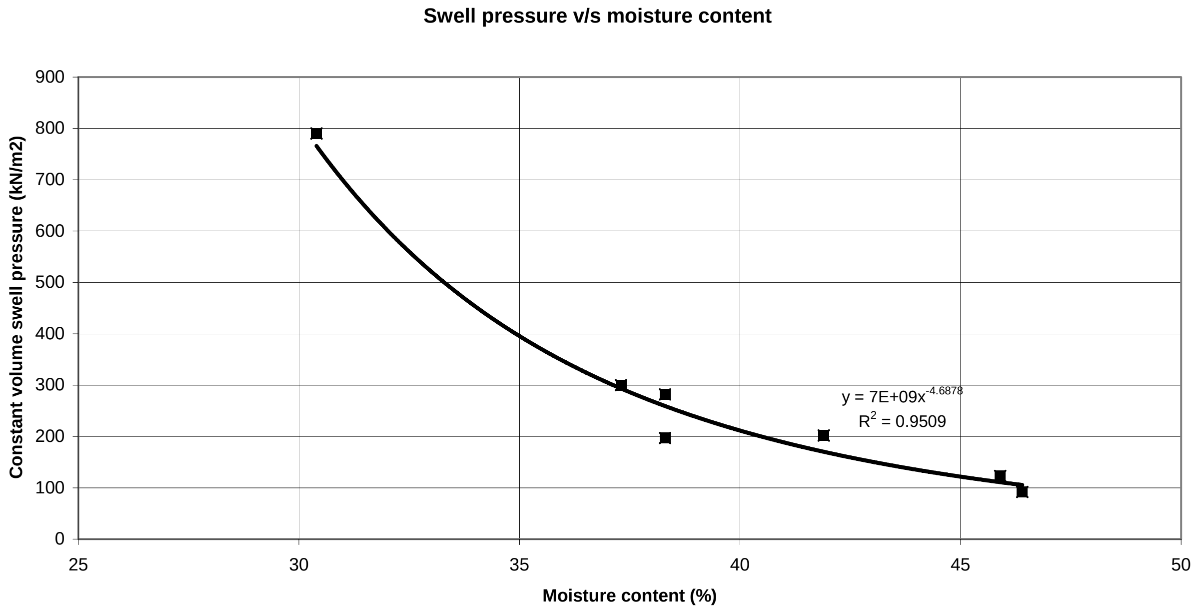


Figure 5.17 Variation of constant volume swell pressure with initial moisture content of dark magnesium clay

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

6.1 Conclusions

1. Published engineering description and classification schemes for rocks and soils were reviewed. It was found that most of them were devised for granitic rocks and/or sedimentary soils. Relevant items were extracted from the published schemes and consolidated into a scheme for local basaltic rocks and residual soils. The scheme was used to standardize the descriptive terms during the input of existing geological logs into an electronic format.
2. A total of about 400 borehole logs, with more than 200 of them located in Port-Louis, have been archived using the SID software package as data management system. The data were used to draw up geological profiles at different locations in Port-Louis. It is concluded that the geology of the latter was influenced significantly by climate which gave rise to thick deposits of alluvium and sea level changes which gave rise to thick deposits of estuarine sediments.
3. For basaltic rocks, the strength and modulus of elasticity were correlated to simple indices. Good correlations were obtained between unconfined compressive strength and porosity or bulk unit weight. On the other hand, the modulus of elasticity of basalt can be estimated from the unconfined compressive strength using a modulus ratio of 600.
4. The point load index test was found to be a simple and quick test for determining the compressive strength of rock cores. A major finding of the research work was that the correlation factor for basalt was found to be 12 instead of 24 as reported in the published literature.
5. The 1962 MSIRI Soil Map of Mauritius was used to group the soils under investigation. Index properties such as Atterberg limits, grain size distribution and specific gravity and strength parameters are given for the five main soil groups.

6. Compressibility, in terms of compression indices obtained from one-dimensional oedometer test, was correlated to initial void ratio as is generally done for sedimentary soils.
7. Design charts have been worked out so that soaked CBR values can be estimated for different soil types. These charts are useful in the design of roads.
8. High shrinkage/swelling potential is the main problem associated with the dark magnesium clay, which is a colluvial deposit. It was found that swell pressures in excess of 100 kN/m^2 could be attained. A chart is given whereby the swell pressure generated by the clay at constant volume can be estimated from its relationship with the initial moisture content.

6.2 Recommendations for future work

6.2.1 Guidelines for soil and rock logging in Mauritius

It is proposed that the guidelines presented in Chapter 2 be published in a user-friendly manual.

6.2.2 Electronic geological and geotechnical database

1. There is a need to upgrade and maintain the computer data management system. The version of the computer software package SID that was acquired could be run only on Windows 98. There is a need to find funds to upgrade the software. As new data become available, the database must be updated and this will require the necessary human resources to maintain the system.
2. With the relevant software, the present database can be integrated in a geographical information system (GIS) so that spatial information can be queried from a digital map.
3. The necessary system can be designed so that the database can be made available on the internet.

6.2.3 Properties of rocks

Further work is required to investigate the index properties of moderately weathered basalts and coral deposits with a view to allow more reliable statistical analysis.

6.2.4 Properties of soils

Data are available for coral sand and estuarine deposits and remain to be analysed.

6.2.5 Database for Rodrigues

The present work can be extended to Rodrigues for which there is a significant amount of geological and geotechnical data.

REFERENCES

Anon. 1979. Classification of rocks and soils for engineering geological mapping. Part 1 – Rock and soil materials. *Bulletin International Association Engineering Geology*, **19**, 364-371.

Bieniawski, Z.T. 1975. The point load test in geotechnical practice. *Engineering Geology*, **9**, pp 1-11.

Bissessur, L. 1995. *Engineering properties of local rock*. B. Tech. Project Report, University of Mauritius.

Bowden, A.J., Lamont-Black, J. and Ullyott, S. 1998. Point load testing of weak rocks with particular reference to chalk. *Quarterly Journal of Engineering Geology*, **31**, pp 95-103.

Brink, A.B.A. and Bruin, R.M.H. 1990. Guidelines for soil and rock logging in South Africa. *Proc., Geoterminology Workshop organized by AEG, SAICE and SAIEG,* South Africa.

British Standard Institution. 1999. *BS 5930: Code of practice for site investigations*. B.S.I, London.

Broch, E. and Franklin, J.A. 1972. The point load strength test. *International Journal Rock Mechanics and Mineral Science,* **9**, pp 669-697.

Brown, E.T. (ed.) 1981. *Rock characterization, testing and monitoring*. Pergamon, Oxford.

Chan Chim Yuk, A. 2002. Evaluation of point load index testing on the vesicular basalt of Mauritius. *Proc., 9th IAEG Congress on Engineering Geology for Developing Countries*, Durban, South Africa.

Chan Chim Yuk, A. and Dabee, B. 1997. Lime stabilisation of the swelling clays of Mauritius. *Proc., International Conference on "Ground Improvement Techniques"*, Macau.

Chan Chim Yuk, A., Armoogum, U, and Nowbuth, M. (1998). *A knowledge-based system for geotechnical site characterization for Mauritius.* Progress report submitted to the Mauritius Research Council.

Chen, F.H. 1975. Foundations on expansive soils. *Developments in Geotechnical Engineering*, **12**, Elsevier Scientific Publishing Co., Amsterdam, Netherlands.

Doobory, B.A.P. 2001. *Point load index test to predict the compressive strength of vesicular basalt rocks*. B. Eng. Project Report, University of Mauritius.

Deere, D.U. 1964. Technical description of cores for engineering purposes. *Rock Mechanics and Engineering Geology*, **1**, 18-22.

Giorgi, L., Borchiellini, S. and Delucchi, L. 1999. *La carte géologique de l'Ile Maurice au 1:50000*. Mauritius.

International Society for Rock Mechanics 1985. Suggested method for determining point load strength. *International Journal Rock Mechanics Mineral Science and Geomechanical Abstract*, **22**, pp 51-60.

Lutton, R.J. 1968. Comparison of strengths of dense and vesicular types of basalt. *Bulletin of the Association of Engineering Geologists*, **5**, pp 1-11.

Ng, C.W.W., Guan, P. and Shang, Y.J. 2001. Weathering mechanisms and indices of the igneous rocks of Hong Kong. *Quarterly Journal of Engineering Geology*, **34**, 133-151.

Parish, D.H. and Feillafe, S.M. 1965. *Notes on the 1:100,000 soil map of Mauritius*. MSIRI Occasional Paper No. 22.

Peck, R.B., Hanson, W.E. and Thornburn, T.H. 1967, *Foundation Engineering*, 2nd edn, John Wiley, New York.

Ramburrun, M. 2000. *Evaluation of point load index test*. B. Eng. Project Report, University of Mauritius.

Rampertaub, S. 1993. *Geotechnical properties of local basalts*. B. Tech. Project Report, University of Mauritius.

Seeboo, A. (1998). *Cement stabilization of a swelling clay*. B. Eng. Project Report, University of Mauritius.

Seed, H.B., Woodward, R.J. and Lundgren, R. 1962. Prediction of swelling potential for compacted clays. *Journal of the Soil Mechanics and Foundations Division, ASCE*, **88**, no. SM4, pp 107-131.

Simmons, J.V. and Blight, G.E. 1997. Profile description and sampling methods. In *Mechanics of Residual Soils*, Blight, G.E. (ed). A.A. Balkema, Rotterdam, pp 41-55.

Simpson, E.S. (1950). The geology and mineral resources of Mauritius. *Colonial Geology and Mineral Resources*, **4**, pp 217-235.

Stroud, M. A. (1989). The Standard Penetration Test – Its Applications and Interpretation. *Proc. I.C.E. Conf. On Penetration Testing*, Birmingham, UK.

Tuncer, E.R., Lohnes, R.A. and Demirel, T. 1977. desiccation of soils derived from volcanic ash. *Transportation Research Record*, **642**, National Academy of Sciences, Washington, D.C., pp 44-49.

Whitlow, R. 2001. *Basic soil mechanics*. 4th edn. Prentice Hall, UK.

Willaime, P. 1984. *Carte pedologique de l'ile Maurice*. ORSTOM/MSIRI, Mauritius.

Williams, A. A. B. and Donaldson, G. 1973. Building on expansive clays in South Africa. *Proc. 4th Int. Conf. On Expansive Soils*, Denver, pp 834 – 844.