IDENTIFICATION AND ASSESSMENT OF PROBLEMATIC EXPANSIVE SOILS

by

PHILIP ROBERT STOTT

A thesis submitted in fulfilment of the requirements for the degree
Doctorate of Engineering in Civil Engineering
in the
Department of Civil Engineering
of the
Faculty of Engineering and Information Technology
of the
Central University of Technology, Free State, South Africa

Supervisor: Prof E Theron

July 2017

PREFACE

The research contained in this thesis was completed by the candidate while based in the Discipline of Civil Engineering, Department of Engineering, Faculty of Engineering and Information Technology, Free State, South Africa. The research was financially supported by the Central University of Technology, Free State and the National Research Foundation.

The contents of this work have not been submitted in any form to another university and, except where the work of others is acknowledged in the text, the results reported are due to investigations by the candidate.

Elizabeth Theron

Signature: ________________

Date: July 2017

Bloemfontein, South Africa

DECLARATION: PLAGI

I, the undersigned, declare that the thesis hereby submitted by me for the degree *Doctorate of Engineering in Civil Engineering* at the Central University of Technology, Free State, is my own independent work and has not been submitted by me to another University and/or Faculty in order to obtain a degree. I further cede copyright of this thesis in favour of the Central University of Technology, Free State. I, Philip Robert Stott, further declare that:

(a) This thesis does not contain other persons' data, pictures, graphs or other information, unless specifically acknowledged as being sourced from other persons.

(b) This thesis does not contain other persons' writing, unless specifically acknowledged as being sourced from other researchers. Where other written sources have been quoted, then:

  (i) Their words have been re-written but the general information attributed to them has been referenced.

  (ii) Where their exact words have been used, their writing has been placed inside quotation marks or in italics, and referenced.

(c) Where I have used material for which publications followed, I have indicated in detail my role in the work.

(d) This thesis is primarily a collection of material, prepared by myself, published as journal articles or presented as oral presentations at conferences. In some cases, additional material has been included.

(e) This thesis does not contain text, graphics or tables copied and pasted from the Internet, unless specifically acknowledged, and the source being detailed in the thesis and in the References sections.

Philip Robert Stott

P.R.Stott

Signature: ________________

Date: March 2017
Bloemfontein, South Africa

DECLARATION: PUBLI S

My role in each paper is indicated. The * indicates the corresponding author.

Chapter 2

Stott, PR* and Theron, E. 2015. Some shortcomings in the standard South African testing procedures for assessing heaving clay. *Journal of the South African Institution of Civil Engineering*. Vol 57 No 2, June 2015, pp. 36–44. Pretoria: SAICE.

The candidate organised, supervised and took part in the testing programme for assessment of standard tests procedures and for possible improvements to these procedures and wrote the paper under supervision of the co-author.

Chapter 3

Stott, PR* and Theron, E. 2015. Shortcomings in the current methods of identification and assessment of heaving clays*. Proceedings of the 16th African Regional Conference on Soil Mechanics and Geotechnical Engineering.* Hammamet, Tunisia 27-30 April 2015. Tunis:ATMS.

The paper is in the form of a review of current methods common in Africa and an assessment of their adequacy. The candidate undertook the tests involved and wrote the paper under the supervision of the co-author.

Chapter 4

Badenhorst, WS., Theron, E and Stott PR*. 2015. Duplicate testing conducted on the input parameters for the estimation of potential expansiveness of clay. *Proceedings of the 16th African Regional Conference on Soil Mechanics and Geotechnical Engineering*, Hammamet, Tunisia 27-30 April 2015. Tunis: ATMS.

The candidate was co-supervisor with Dr E. Theron for the first author’s research project, for which the investigation described in this paper formed a part. The candidate collaborated with the first author in writing the paper.

Chapter 5

Stott, PR* and Theron, E. 2015. Assessment of clays by small-scale suction tests. *Proceedings of the 16th African Regional Conference on Soil Mechanics and Geotechnical Engineering*. Hammamet, Tunisia 27-30 April 2015. Tunis:ATMS.

The candidate devised and conducted the tests described and wrote the paper under the supervision of the co-author. The conference organisers announced that selected papers would be invited for elaboration and publication in the Centenary issue of *Geotechnical Engineering* in 2017. This paper was selected and the candidate expanded the paper significantly after additional research. The resultant paper is presented in Chapter 7.

Chapter 6

Stott, PR* and Theron, E. 2016. Variability in soil properties and its consequence for design. *Proceedings of the 1st Southern African Conference on Geotechnical Engineering*. Sun City 5-6 May 2016. Rotterdam: Balkema.

The candidate devised and performed the tests and wrote the paper under the supervision of the co-author.

Chapter 7

Stott, PR* and Theron, E. 2017. Estimation of Shrink/Swell Potential and Variability of Clays by Small-Scale Suction Tests. *Geotechnical Engineering ,June,* 2017 (in print)

This paper was an invited contribution for the SEAGS and AGSSEA Journal's Centenary edition and expands on methods introduced in Chapters 5 and 6. The candidate performed all tests involved in the investigation and wrote the paper under the supervision of the co-author.

Chapter 8

Stott, PR* and Theron, E. 2016. Shortcomings in the estimation of clay fraction by Hydrometer. *Journal of the South African Institution of Civil Engineering* Pretoria: SAICEVol 58 No 2 June 2016. pp.14-24.

The candidate devised the pr performed the investigations and wrote the paper under the supervision of the co-author.

Chapter 9

Stott, PR*., Jacobsz, SW and Theron, E. 2017. Particle size distribution for geotechnical purposes: traditional and automatic particle size analysis assessed by optical examination. It is ready for submission to a technical journal.

The candidate performed the tests and wrote the paper under the supervision of the co-authors.

Chapter 10

Stott, PR*., Monye, PK and Theron, E. 2016. Assessment of reliability of the hydrometer by examination of sediment. *Proceedings of the 1st Southern African Conference on Geotechnical Engineering*. Sun City 5-6 May 2016. Rotterdam: Balkema.

The candidate devised the apparatus and the test procedures, supervised the tests (performed by PK. Monye) and wrote the paper in collaboration with the second author.

Chapter 11

Bester DM., Stott, PR* and Theron, E. 2016. The movement of soil moisture under a Government subsidy house. *Proceedings of the 1st Southern African Conference on Geotechnical Engineering*. Sun City 5-6 May 2016.Rotterdam: Balkema.

The candidate conceived the project, co-supervised (with Dr E. Theron) its execution by D.M. Bester and collaborated in writing the paper.

Chapter 12

Stott ,PR* and Theron, E. 2017. Rapid generation of soil shrinkage curves using small samples and highly volatile non-polar fluids. It is ready for submission to a technical journal

The candidate devised the procedure and the apparatus and wrote the paper under the supervision of the co-author.

ABSTRACT

In the last few decades the construction of small dwellings, particularly government subsidy houses, has increased markedly in South Africa. Large numbers of these light structures have suffered severe damage and many have been demolished after being occupied for only a small fraction of their expected life-span. The research detailed in this thesis deals with a search for reasons for these widespread failures and investigations into economically feasible solutions. The investigation began with an assessment of the current methods of identification and assessment of potentially problematic expansive soils. This revealed serious shortcomings in a number of the standard testing procedures. These shortcomings may have consequences for a wider range of engineering projects than just housing development. Progress towards more reliable testing methods has led to an appreciation of the importance of variability of soil properties and the necessity of taking this into account in engineering design. Investigation of clay fraction determination by the hydrometer has led to the realization that the unreliability of this procedure has been a cause of failure in subsidy housing projects as well as other light structure developments. Assessment of expansive potential cannot safely rely on hydrometer determination of clay fraction, which is a principal parameter for a number of popular design procedures. It was also established that attractive alternatives in the form of automatic particle sizing equipment (designed for the analysis of industrial powders) are not suitable for natural soils containing the clays commonly found in South African soils. Alternative procedures, particularly small-scale suction testing, have been found to provide more reliable insights into design for expansive clays. The investigations suggest that some form of Reliability Based Design, which takes account of specific variability in soil properties, may be the only way to achieve rational design for foundations on expansive clay.

ACKNOWLEDGEMENTS

A number of special acknowledgements deserve specific mention:

(a) The Rectorate and relevant functionaries from the Central University of Technology, Free State, for the opportunity of completing this research;

(b) The various agencies for funding and in particular the Central University of Technology, Free State and National Research Foundation;

(c) Prof E Theron of Central University of Technology, Free State, whose help, advice, encouragement and exceptionally competent administrative input have been invaluable;

(d) Prof Derek Sparks of University of Cape Town, whose help was gratefully accepted until his retirement at a very advanced age led to my feeling unable to trouble him further

(e) Prof G Blight, a giant in the field. His advice and help was cut short when he succumbed to cancer in 2012. His passing was a great loss to all in the geotechnical field and to South Africa in general;

(f) Prof P Day of University of Stellenbosch and director of Jones Wagner Pty.Ltd. His willingness to help, advise and encourage all involved in geotechnical engineering has been pure inspiration to me;

(g) Prof SW Jacobsz of Pretoria University for guidance and support given;

(h) Staff and Students of the Civil Engineering department of Central University of Technology who helped with many hours of experimental work: Prof E. Theron, D. Bester, J. Koller, E. Burger, M. Jele, W. Bandenhost, E. Burger, CVD. Westthuizen and J. Honeyball;

(i) Undergraduate students, keenly and voluntarily involved in the Central University of Technology's Soil Mechanics Research Group, who assisted with some of the more advanced tests in particular Z. Smith and L.J. Coffee; and

(j) My family and colleagues, for their patience and understanding throughout this research.

Above all I would like to acknowledge the Lord Jesus Christ, without whom we can do nothing, but through whom we can do all things.

TABLE OF CONTENTS

Page

LIST OF TABLES

Page

LIST OF FIGURES

Page

LIST OF ABBREVIATIONS

CEC	Cation Exchange Capacity
CLSM	Continuous Logging Soil Moisture
COV	Coefficient of Variation
g/s	Gramme per Second
g	Gramme
kN	Kilonewton
kPa	Kilopascal (or kN/m^2)
LL	Liquid Limit
LS	Linear Shrinkage
LS	Laser Scattering
m	Metre
MBV	Methylene Blue Value
MB	Methylene Blue
MBI	Methylene Blue Index
mg	Milligram
min	Minute
ml	Millilitre
mm	Millimetre
Mpa	Megapascal (or N/mm^2)
PI	Plasticity Index
PL	Plastic Limit
PSD	Particle Size Distribution
RBD	Reliability Based Design
s	Second
SEM	Scanning Electron Microscope
SSA	Specific Surface Area
SWCC	Soil Water Characteristic Curve
TMH	Technical Methods for Highways
W	Moisture content

LIST OF ACRONYMS

AFNOR	Association Française de Normalisation
ASTM	American Society for Testing and Material
BS	British Standards
CSRI	Council for Scientific and Industrial Research
CUT	Central University of Technology
GSH	Government Subsidy House
IS	Indian Standards
ISO	International Organisation for Standardisation
NHBRC	National Home Builders Registration Council
SABS	South African Bureau of Standards
SAICE	South African Institution of Civil Engineering
SANAS	South African National Accreditation System
SANS	South African National Standards
UFS	University of Free State

CHAPTER 1: INTRODUCTION

1.1 Backgrounds to the Problem to be Investigated

The problem of founding structures on expansive soils began to receive considerable attention in South Africa in the late 1950's. The major mining houses were then experiencing heave problems in accommodation provided for their work-force, particularly at their mines in the Free State. The mining houses provided funding for geotechnical researchers to investigate the problem. Some of the most respected soil mechanics experts in the country, including Professor J.E. Jennings and Professor K. Knight, were involved in this research. Since that time mining in South Africa has become less profitable and funds for such investigations have largely dried up. Little research has been done on this problem since the 1980s.

By far the most commonly used method for assessing heave potential for light buildings remains that devised in 1963 by Van der Merwe (1964), as part of the quest for solutions to work-force housing problems for gold-mining companies in the Free State. This widespread acceptance is somewhat surprising since the method was based on examination of just four houses and two oedometer tests. Of the four houses, two showed very good agreement between predicted and observed heave, one showed acceptable agreement and one showed very poor agreement (predicted heave 120mm and observed heave 220mm).

In the rest of the world the problem of heaving foundations began to receive attention at about the same time as in South Africa. However, formal instruction in Soil Mechanics courses lagged behind research in this field throughout the world and despite a growing number of conferences covering research into this topic, very few practicing engineers found themselves equipped to deal with the problems associated with expansive clays. In the 1990s the National Science Foundation in America sought to address this problem by sponsoring "a document which can serve both as a reference guide for the practicing engineer and a textbook for a course on foundations and pavements on expansive soils." The result was the book "Expansive Soils: Problems and Practice in Foundation and Pavement Engineering." (Nelson and Miller, 1992), which remains the standard text on this subject. The topic of expansive clays has been expanded into the discipline of “Unsaturated Soil Mechanics”. The former series of four-yearly “International Conference on Expansive Clay Soils” gave way in 1995 to the four yearly “International Conference on Unsaturated Soil Mechanics.” Attention of the research community in this field has tended to shift away from expansive clays to more tractable and profitable fields, particularly environmental protection, where unsaturated soil mechanics has proved to be particularly successful.

The question of expansive soils has again become a prominent problem in South Africa since the new political dispensation came into being in 1994. A rapidly emerging black middle class with aspirations of a home of their own led to burgeoning construction of single-storey houses. Attempts by the government to provide low cost housing for the poorest section of the community led to the construction of millions of small, simple, light houses. This significant increase in light structure construction has coincided with progressive contraction of the economy. Mining companies are no-longer in a position to subsidise research. Public service institutions like the Council for Scientific and Industrial Research have inadequate resources to undertake significant research. The public service is now staffed largely by management personnel with little or no engineering knowledge and it is perhaps not surprising that constructive attempts to solve the problem of damage to large numbers of light structures due to expansive clay problems appear to be lacking. Large numbers of failures continue to be common. Large numbers of houses are being demolished and replaced and in many cases these replacements seem to have little prospect of a longer useful life than the ones they replace.

1.2 The Beginning of this Research Investigation

As a practicing Civil Engineer the candidate became involved with failures due to heaving clay problems in 2009. The National Home Builders Registration Council (NHBRC), which acts as an insurance agent for newly constructed dwellings, commissioned investigations in which the candidate was called upon to examine a number of light structures. Most of these were in either emerging middle-class developments or government subsidy housing projects. The aim of the investigations was to establish the reasons for the houses concerned having cracked to the point of becoming structurally unsound. These investigations usually showed that normal accepted standards and procedures had been followed, yet the foundations had been unable to provide sufficient resistance against shrink/swell of the underlying clay to prevent structural damage. The candidate began a study of available literature on the subject of foundation design for expansive clay. This led to correspondence and meetings with Professor Derek Sparks of Cape Town University who appeared to be the only academic in South Africa dealing with expansive clays. This association and the literature search led to the realisation that satisfactory answers were not to be readily found in the literature. They also led to the realisation that soils testing, focussed on the problems exposed by these informal investigations, were needed. Such testing could not be done at Professor Sparks' laboratory in Cape Town since it had been demolished to make way for a new one. Dr E. Theron at the Central University of Technology was approached in 2010 for possible involvement of the CUT soil mechanics laboratory. Dr Theron suggested that this would be best handled as a Doctoral research project. A team of interested staff and students were prepared to help with the investigations and the CUT Soil Mechanic Research Group was formed in 2011 specifically to examine problems of foundations of light structures on expansive clays. Professor G.E. Blight, a former colleague from Witwatersrand University, kindly agreed to give advice with the project. His assistance was cut short when he unfortunately succumbed to cancer in 2012. This thesis deals with progress made towards understanding why current procedures appear to be inadequate, and finding possible solutions to the problem.

1.3 Currently Available Procedures

1.3.1 Atterberg limits

Many methods (Van der Merwe, 1964; Savage, 2007; Weston, 1980; Sparks, 2010 etc.) for estimating swell potential rely on the Atterberg limits. These tests have the advantage of being simple and inexpensive, and they are offered by almost all soils laboratories. They do not give a direct measurement of expansive potential. The Plasticity Index (PI) (Liquid Limit (LL) – Plastic Limit (PL)) gives an indication of the amount of water which can be absorbed when mechanical mixing effort is applied, but gives no indication of how strongly water can be drawn into a sample against pressure and hence cause heave damage.

1.3.2 Linear shrinkage

Linear Shrinkage (LS), which is normally measured together with Atterberg Limits, is also well established and employs very simple apparatus. It is one of the standard "Foundation Indicator Tests" offered by most South African soils laboratories. It can give a very good indication of how much volume change is possible in a sample if performed in a suitable manner, but does not give an indication of the forces which can be mobilised in this volume change.

1.3.3 Hydrometer and pipette determination of clay fraction

Clay content is one of the input parameters for many heave prediction methods (Skempton, 1953; Van der Merwe, 1964; Nayak and Christensen, 1974 etc.). Determination of clay and silt fractions by sedimentation has been standard procedure world-wide for many years and is the standard method of fine fraction determination in most countries. It is a relatively quick, easy, and inexpensive test. There have been questions raised about its reliability e.g. (Rolfe *et al.*, 1960); (Nettleship *et al.*,1997); (Savage, 2007) and there have been investigations into possible replacement by automated optical methods. No alternative has gained acceptance and sedimentation by hydrometer or pipette remains the accepted standard throughout the world. It was not anticipated that this would be a major problem for the investigation to examine.

1.3.4 Oedometer testing

The oedometer has the potential to indicate both volume change and the forces involved in volume change, and to do so for relatively undisturbed samples. It was formerly favoured for expansive clay analysis because theoretically is would appear that it should give meaningful results. But sample preparation has proved to be difficult and any lack of fit between sample and confining apparatus leads to invalid results as voids are filled by expansion of the sample without expansion against applied load. The method proposed by Schreiner and Habte (2008) using compacted, slurried material may solve this, but the potential advantage of testing substantially undisturbed samples is lost. Nelson and Miller (1992) deal with the odometer in detail, but conclude that it does not provide reliable indication of heave potential. Tests performed by the candidate on very expansive plastic clays showed that from adding water to the sample until cessation of expansion against low pressure could take several weeks, making the test unattractive for practical purposes, and probably unfeasible for low cost housing.

1.3.5 Clod test

The clod test is often used in America and is indicated by Nelson and Miller to be probably the best heave indicator test available. The test uses Saran resin to form a semi-permeable coating to samples. The resin is permeable to water vapour, allowing moisture change, but effectively impermeable to short term immersion in liquid water, allowing volume determination by Archimedes principle. Saran resin is not available in Africa. It is prepared by dissolving in methyl ethyl ketone, which is hazardous for health and needs special handling. Alternative non-toxic coatings have been proposed (Krosley *et al.*, 2003). The authors claimed that testing time was approximately the same as for Saran Resin. The candidate tested several alternative coatings similar to those used by Krosley *et al.* Time to produce the shrinkage curve was more than two months, which is again unreasonably long for the problem at hand.

1.3.6 X-Ray diffraction

Mineral composition has very significant influence on swelling potential. Mineral composition can be indicated by X-Ray diffraction. The equipment for this analysis is very expensive and few South African institutions possess such apparatus. While involved with soils testing for this project in co-operation with the soil science laboratory at the University of the Free State, the candidate delivered samples for

X-Ray analysis. The samples remained in a queue for more than one year by which time it was concluded that such delays ensure that this method cannot be considered a possible solution for the problem at hand. Recently apparatus has become available which gives results in less time and with less skilled input, but such apparatus was not available for testing.

1.3.7 Cation exchange capacity

Cation Exchange Capacity (CEC) gives an indication of mineral composition. CEC is related to clay mineralogy and gives an indication of surface activity. The higher the CEC, the higher the surface activity, and the greater the probable heave potential. The test for CEC is not normally offered by engineering soils laboratories, but is commonly offered by agricultural laboratories. It is complex and involves exchanging the cations naturally occurring in a soil sample with cations of a known species. The process involves detailed and precise procedures, expensive spectral analysis apparatus for identification of exchanged cations and takes considerably more time than most common engineering soils tests.

1.3.8 Methylene blue value

The Methylene blue value (MBV) test is a quick alternative to the CEC test. It is widely used in France. Methylene blue preferentially displaces cations from clay surfaces. It gives an estimate of the CEC without attempting to identify the cations involved. It appears to be rarely used in South Africa, but tests at the CUT soils laboratory indicate that it is likely to be a good indicator of heave potential and is quick and inexpensive to perform. It was also found that Methylene Blue can be very useful for labelling clays and it played a significant role in the investigations detailed in Chapters 8, 9 and 10.

1.4 The Unsaturated Soil Mechanics Approach to Expansive Clay Problems

On the wider front of world-wide advance in unsaturated soil mechanics, the main thrust has been in applying the principles of continuum mechanics to geotechnical problems. This appears to promise a more rational and reliable approach than empirical methods like those of Van der Merwe (1964) and Savage (2007) etc. noted above. Partial differential equations are set up to model the general processes involved; boundary conditions are determined corresponding to the particular problem on hand and mathematical solution packages are then used to solve the differential equations. A major problem for this approach has been the determination of suitable soil parameters to use in the equations. This approach to unsaturated soil mechanics requires soil property functions rather than simple soil properties as in the case of saturated soil mechanics. Determining these soil property functions requires lengthy laboratory investigations. Techniques have been developed for estimating the required functions from saturated soil properties, but these estimations require the soil water characteristic curve (SWCC), also known as the soil suction curve, or the soil water retention curve, which represents the relationship between suction and water content for the soil from saturation to desiccation. The determination of the SWCC is a protracted and expensive process. In spite of widespread investigations into techniques for speeding up SWCC determination it remains a lengthy process for soils with significant clay content.

In some areas of geotechnical engineering this continuum mechanics approach has had considerable success – particularly in the field of providing soil covers for the protection of the environment from undesirable wastes and pollutants. But in the field of expansive clays it has proved particularly unsuccessful. This was pointed out by DG. Fredlund, one of the leaders in the field, (Fredlund, 2009) when he remarked in his presentation of the 8th C.W. Lovel lecture at Perdue*, I started in unsaturated soil mechanics by looking at swelling clay problems, where if you get involved you usually lose money, it is not a good problem.* This certainly does not suggest that the problem of foundations on swelling clay is solved or that the unsaturated soil mechanics approach of continuum mechanics has immediate prospects of delivering a solution.

1.5 Research problem to be addressed by this thesis

The research problem addressed by this thesis is (a) the identification of the weaknesses in current methods of identifying and assessing expansive clays which lead to widespread foundation failures in South Africa and (b) the search for alternative methods which will assess such clays more accurately and reliably.

1.6 Proposed Methodology for Solving the Problem

It was proposed first to conduct sensitivity analysis for some of the empirical methods (Van der Merwe, 1964; Savage, 2007; Sparks, 2010 and Weston, 1980 etc.) to determine their sensitivity to precision attainable for the readily available inputs - Atterberg Limits etc. Multiple testing would then be performed to compare precision achievable with that required and assess the impact on the potential reliability of these empirical methods. If necessary, new methods would be sought to give better precision for these parameters. Figure 1.1 shows sensitivity of heave prediction to variation in PI for Savage's method. It can be seen that for LL = 55, inaccuracy of 2 either side of the assumed correct PI 28 leads to difference in predicted heave from 35mm to 67mm per m depth.

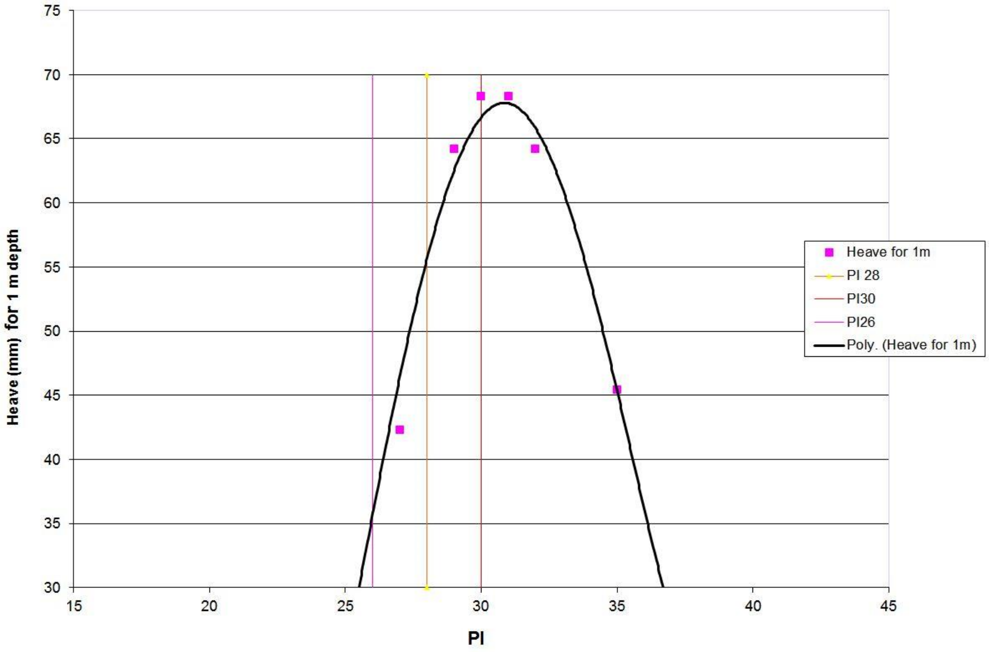


Figure 1.1: Sensitivity of heave prediction (using Savage's method 2007) to variation in PI for LL = 55.

Finite element methods would be developed using these input parameters to model interaction between raft foundations typical of those used for light structures in general and government subsidy housing in particular. Such models would depend on mound shapes which developed under the foundations. The initial assumption would be guided by results from ground cover tests (de Bruin, 1974); (Pidgeon and Pellissier,1987); (Miller *et al*.,1995) and (Fityus *et al*. 2004). These tests had monitored heave under simulated foundations of either plastic sheets or raft-like concrete slabs. It was proposed that a member of the CUT Soil Mechanics Research Group would undertake a Masters project under the candidate's supervision to instrument at least one real government subsidy house to compare moisture distribution under a real house with the results from simulated foundations. Inputs to the finite element analysis could be adjusted accordingly.

The second proposed stage was to seek quicker ways of producing the inputs required for the continuum mechanics approach currently favoured by the leaders in the field of unsaturated soil mechanics. This approach uses differential equations suitable for the process at hand – in this case moisture movement and consequent volume change under a foundation. Boundary conditions are applied depending on the foundation shape and form of the surrounding ground. Solution of the equations is achieved by dedicated software (the FlexPDE package was tried and found to be suitable). The expense and time required to produce the input functions – particularly the SWCC and the shrinkage curve – were a barrier to such procedures being feasible for the problems being addressed. Only if quicker alternatives had been found would this method have been feasible.

1.7 Progress Towards Achieving Expected Goals

1.7.1 Assessment of standard tests

A range of expansive soils from government subsidy housing projects were collected and tested by members of the CUT research team under the candidate's supervision using the Technical Methods for Highways (TMH) procedures then standard for engineering soils testing in South Africa. Results were compared with those from commercial laboratories. Discrepancies were far larger than expected. The candidate undertook an investigation into the TMH procedures and also into the SANS 3001 procedures which had recently been introduced with the intention of replacing TMH. It was found that both were seriously flawed. Among other problems both contained alternative procedure without clear guidance on the use of the alternatives. No guidance was given as to who is responsible for choosing between discretionary alternatives even though major time and cost factors are dependent on these decisions. Investigations at commercial laboratories concluded that the quickest, cheapest procedures are always followed, however unsuitable they may be for the particular soil at hand. A paper (Chapter 2) dealing with the investigation, its findings, and proposals for possible changes was published in the journal of the South African Institution of Civil Engineering in June 2015.

1.7.2 Finite element approach

Progress was made in developing finite element models to examine structural interaction with heaving foundations. Figure 1.2 shows a finite element model of a government subsidy house on a raft foundation subjected to heave beneath the foundation. The figure illustrates that the rigidity of the walls of the house tends to concentrate stress in the beams of the raft directly under door opening. Such models assumed the kind of heave mound generally accepted following ground cover tests referred to in section 1.4. Further development was put on hold until results from the house being instrumented (introduced in section 1.4) could give a distorted shape for the foundation more likely to be a true reflection of reality.

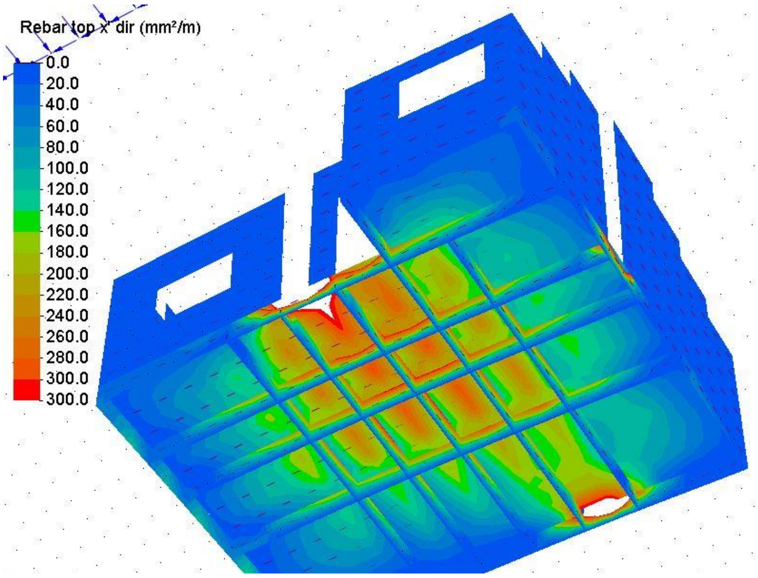


Figure 1.2: Finite element model.

1.7.3 Unsaturated soil mechanics approach

Progress was made with rapid procedures for determining essential unsaturated soils inputs. Accurate shrinkage curves for a very wide range of soils could be produced within 8 hours, as opposed to several weeks using the standard procedure of Saran resin coating. Two very widely different soils are shown in Figures 1.3 and Figure 1.4.

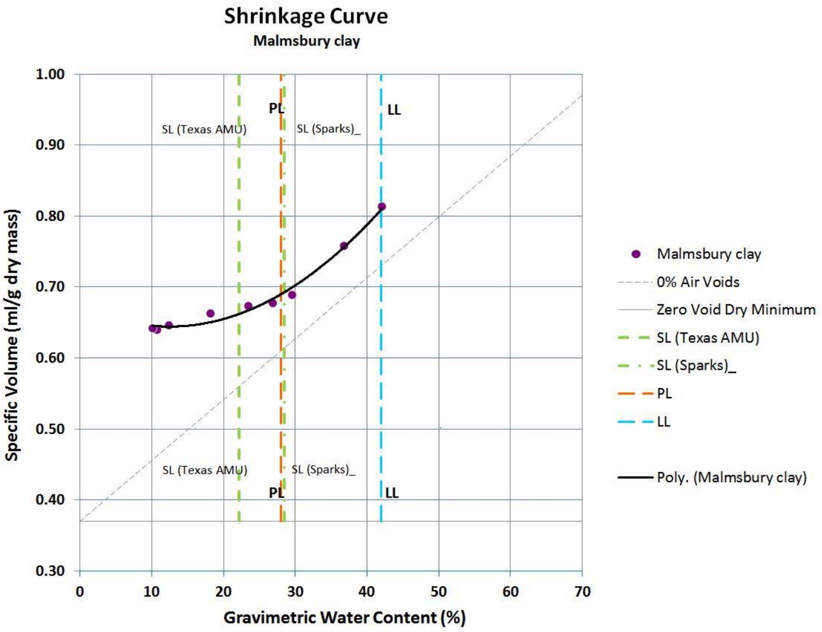


Figure 1.3: Shrinkage curve for a kaolinitic clay.

Figure 1.3 show the shrinkage curve from Liquid Limit to Shrinkage Limit for a kaolinitic clay from the Western Cape, whilst figure 1.4 shows the shrinkage curve from Liquid Limit to suction in excess of 300 MPa for a very expansive clay from the Free State. It appears that this clay may not reach its shrinkage limit even in severe drought conditions. Chapter 12 deals with details of some of the procedures developed in this aspect of the research project.

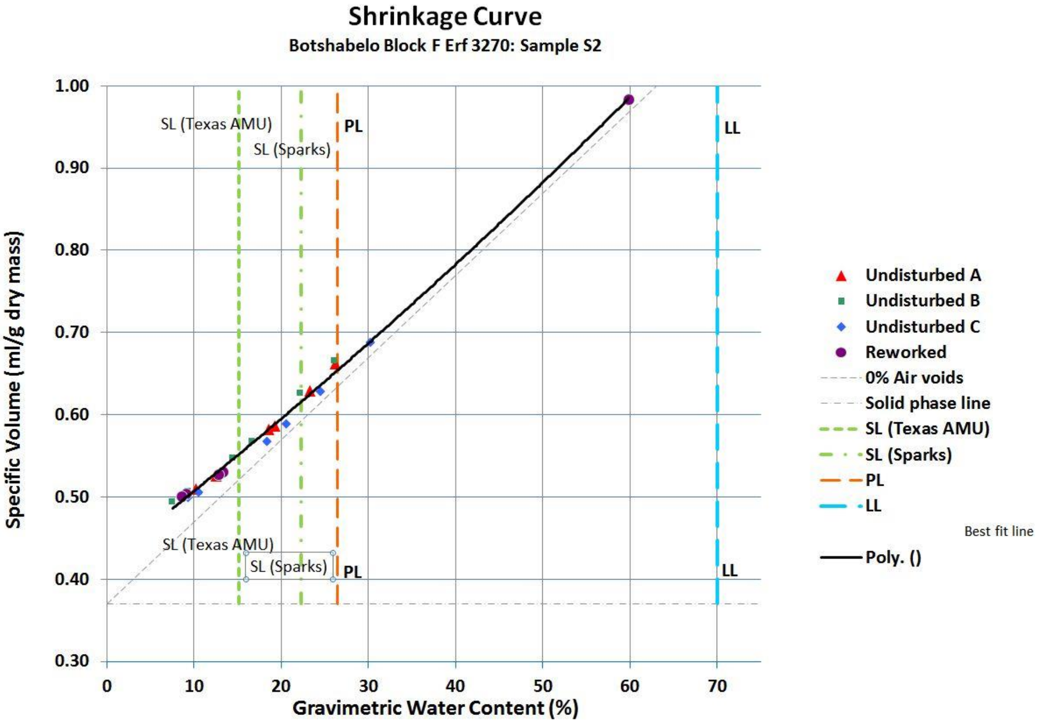


Figure 1.4: Shrinkage curve for a very expansive clay.

1.8 New Avenues of Investigation

Perhaps the most interesting aspect of research is that unexpected avenues of investigation open up and invite attention. Two such openings led to a change in focus towards two aspects of soil mechanics which appear to have been somewhat overlooked. These two aspects – variability of soil properties and clay fraction determination - have such a profound influence on the problem under investigation – damage to light structures due to expansive clays – that they have become the prime focus of attention in this research project.

1.9 Small Scale Suction Tests

Soil Mechanics became a science when Karl Terzaghi introduced the concept of effective stress. Saturated soil mechanics deals with soil solids and soil water. Effective stress is total stress minus soil water pressure. Unsaturated soil mechanics is more complex, since it deals with soil solids, soil water and soil air. Unsaturated soil pioneers – Fredlund, Bishop, Blight and others established that soil suction is as fundamental to unsaturated soil mechanics as effective stress is to saturated soil mechanic.

Suction in unsaturated soils is caused by two distinct factors. The first is capillary suction due to the meniscus of pore water bridging the gap between individual soil particles. In the case of clay, the individual particles are very small, the capillary diameter between particles is extremely small, and the capillary suction is consequently high. The second factor causing soil suction is far more complex and involves electrostatic forces in clay mineral structure.

Capillary suction is not usually associated with major damage to structures such as dwelling houses or even very light structures such as roads. Electrostatic suction, however, can be very powerful and can cause severe damage to low-rise buildings, roads, pipelines, canals, drains and other items of infrastructure.

It would therefore appear to be of more value to assess suction potential than to assess the quantity of clay in a soil when attempting to predict its potential for causing damage due to heave. A clay like Kaolinite, which has a very stable molecular structure, may show a high PI (indicating the ability to absorb a considerable amount of water), but this water is mostly in the form of relatively low-suction capillary water and is unlikely to cause serious heave problems even if it makes up a large fraction of the soil. On the other hand, Montmorillonite, which can draw in water electrostatically into its physical structure against considerable pressure, may cause serious heave problems even if it makes up only a very small fraction of the soil (Rogers *et al.*, 2010).

Unsaturated soil mechanics places great emphasis on the value of soil suction and its relationship with water content, but it remains challenging to measure soil suction reliably and economically. Direct measurement of soil suction, usually involving the use of manometers, is very slow and inconvenient and is restricted to such low suctions that it has limited value for engineering purposes. “Axis translation” techniques, which assess suction potential by applying pneumatic pressure to expel water from a sample, require considerable expenditure on equipment, careful maintenance and several weeks of testing for each sample of significantly plastic clay. The method cannot deal with suctions above 1500 kPa and to reach this value can be quite challenging. Instruments specifically designed to measure soil suction are becoming more reliable and affordable, but tend to be able to measure a relatively small range of suction. Several samples were tested at the soil science laboratory of the University of the Free State as part of this investigation. It was found that the time and expense of testing highly plastic clays by these methods will ensure that they will never be commercially viable for any but the largest engineering projects. An inexpensive and versatile method which can cope with a wide range of suctions uses high-quality filter paper to measure suction (Gourley and Schreiner 1995; Bulut *et al.*, 2001). This is a popular method in practical field investigations (Mavroulidou *et al.* 2013; Mendes and Toll ,2013), but the filter paper method takes from one to several weeks to yield a suction value for expansive clay and requires careful laboratory technique.

Blight used the well-established principle of bringing samples to equilibrium with saturated solutions of various salts, or various molarities of certain chemicals, to determine the suction potential of soils (Blight, 2013). His experiments suffered from the problem of a long (90 day) equilibration period, which made his procedure unattractive for practical engineering use. In the quest for rapid methods of determining suction potential of soils, the candidate conducted tests comparing water content retained by soils and Whatman’s 42 filter paper for which suction calibration curves are readily available (Gourley and Schreiner, 1995). Comparisons were done using very small samples which allowed many samples to be tested at once (over standard solutions in air-tight containers or in a climate chamber). Somewhat surprisingly it was found that while the filter paper was very consistent in its water content retained against various applied suctions, some of the soil samples were not consistent. In fact, some showed very large differences in water retention between individual samples of the same soil. A long and detailed series of tests was undertaken on many hundreds of samples which led to the conclusion that coefficient of variability in suction potential may be a definite property of active clays, and that the range of this variability across different clays is very large. Since suction is directly linked to heave potential, this implies that heave potential may vary widely across different samples of one soil also. PI is often used as an indication of heave potential. Multiple duplicate testing at a number of commercial laboratories gave coefficient of variation in PI quite comparable to those given by small scale suction tests. These small scale suction tests give not only a good indication of variability but also a good indication of heave potential in a very reasonable time. The procedures for small-scale suction tests were presented at the 16th African Regional Conference on Soil Mechanics and Geotechnical Engineering in 2015 (Chapter 5). Findings from investigations into variability using this technique were presented at the 1st Southern African Conference on Geotechnical Engineering in 2016 (Chapter 6). The candidate was invited to expand the paper from the 16th African Regional Conference for publication in the centenary edition of *Geotechnical Engineering.* This paper (Chapter 7) deals comprehensively with the estimation of both heave potential and variability using the procedures introduced in chapters 5 and 6.

The results of the above investigations throw doubt on the value of some of the highly mathematical procedures being considered in unsaturated soil mechanics. Fredlund has long emphasized the need for parametric functions such as the SWCC in unsaturated soil mechanics (Fredlund and Rehardjo, 1993; Fredlund, Rehardjo and Fredlund, 2012). But for an expansive soil with a large coefficient of variation a range of SWCCs can be produced from a range of individual samples. Each SWCC costs in the region of $10 000 to produce according to Fredlund (2009), and to gain a fair idea of variability at least 10 such SWCCs would be needed. Also to make sense of this variability something comparable to a Monte-Carlo analysis might be required at each step of the solution of the simultaneous differential equations when dealing with such soils. This might explain Fredlund's comment, noted above in section 1.3, in which he inferred that heave problems remain intractable to unsaturated soil mechanics, whereas many other soils problems can be dealt with successfully.

The existence of significant variability over a small spatial range also suggests plausible reasons for some rather surprising occurrences. For example, Miller *et al.* (1995) found a very strange heave pattern under the ground cover they were studying at an active clay site in Texas. The cover was intended to simulate a foundation. Figure 1.5 shows the heave pattern observed. Miller *et al*. suggested that this strange pattern might be due to the fact that the strata underlying the site were not level and this could have allowed moisture to penetrate under the simulated foundation in such a way as to allow a very uneven moisture-content pattern to develop in the underlying clay.

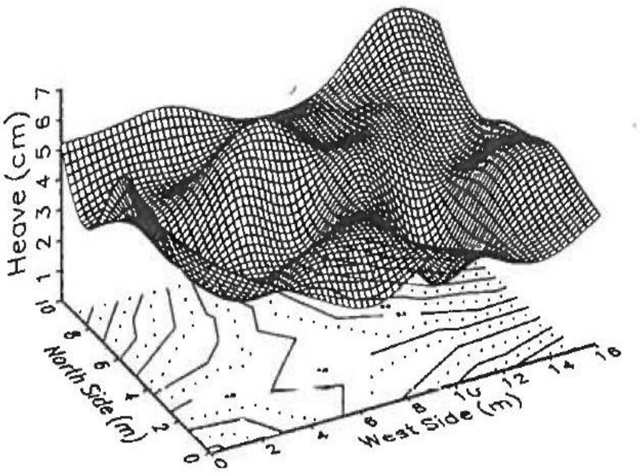


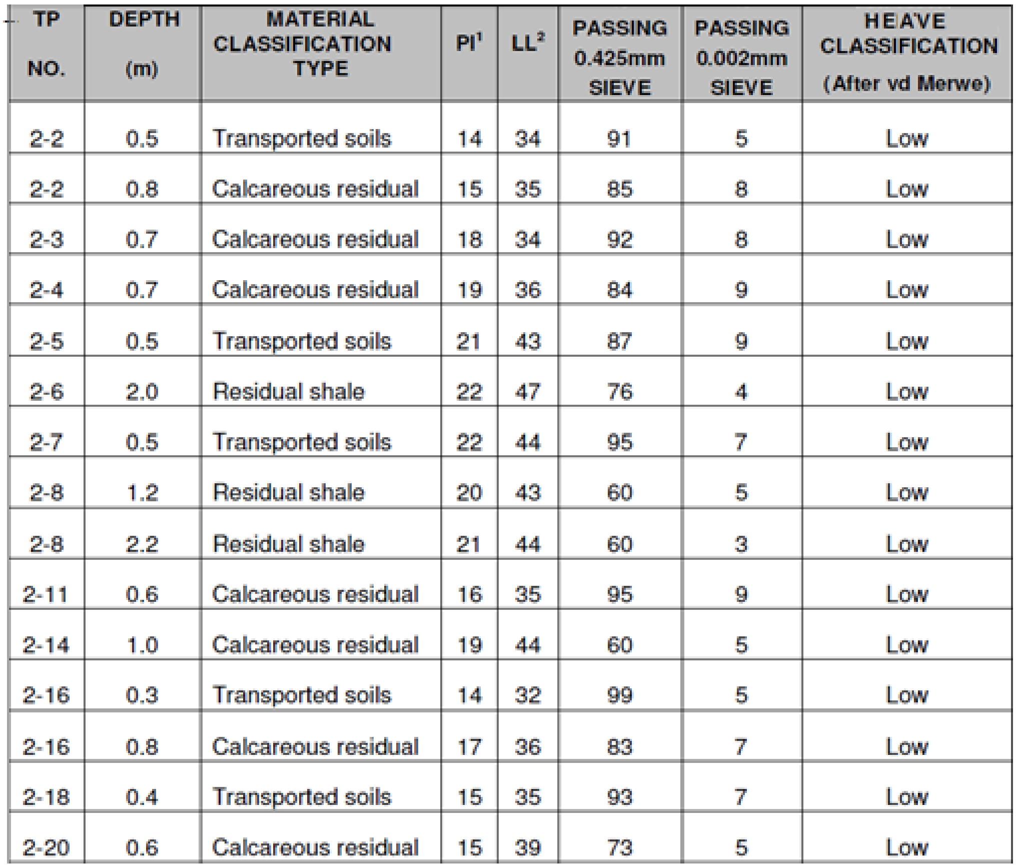
Figure 1.5: Heave pattern observed by Miller *et al.* (1995).

In the light of the prevalence of large variability over small spatial ranges found in the above suction tests, it seems that a more likely explanation could be significant variability of heave potential in the expansive clay beneath the cover. This might also explain the not uncommon situation shown in Figure 1.6. Clayey soil from an adjoining housing development to the right of the road shown in the picture was tested and found to have a substantial coefficient of variation for heave potential.

Figure 1.6: Undulations in a road.

1.10 Clay Fraction Determination

Due to interest in the work of the CUT Soils Research Group, invitations were received to visit housing developments where severe damage by heaving clay had led to a large proportion of houses being demolished. In addition, the group was invited to comment on a new development (Lerato Park) of a higher standard than normal for government subsidy housing. The candidate noticed that no movement joints had been provided in any of the houses, some of which were two storeys and many consisted of semi-detached “duets”. A study of the geotechnical report revealed that extremely low clay content characterized the entire site. Figure 1.7 is an extract from the geotechnical report indicating the test results from phase 2.



TP NO.	DEPTH (m)	MATERIAL CLASSIFICATION TYPE	PI[1]	LL[2]	PASSING 0.425mm SIEVE	PASSING 0.002mm SIEVE	HEAVE CLASSIFICATION (After vd Merwe)
2-2	0.5	Transported soils	14	34	91	5	Low
2-2	0.8	Calcareous residual	15	35	85	8	Low
2-3	0.7	Calcareous residual	18	34	92	8	Low
2-4	0.7	Calcareous residual	19	36	84	9	Low
2-5	0.5	Transported soils	21	43	87	9	Low
2-6	2.0	Residual shale	22	47	76	4	Low
2-7	0.5	Transported soils	22	44	95	7	Low
2-8	1.2	Residual shale	20	43	60	5	Low
2-8	2.2	Residual shale	21	44	60	3	Low
2-11	0.6	Calcareous residual	16	35	95	9	Low
2-14	1.0	Calcareous residual	19	44	60	5	Low
2-16	0.3	Transported soils	14	32	99	5	Low
2-16	0.8	Calcareous residual	17	36	83	7	Low
2-18	0.4	Transported soils	15	35	93	7	Low
2-20	0.6	Calcareous residual	15	39	73	5	Low

Figure 1.7: Extract from the geotechnical report.

However, cracks were visible at the ground surface as is shown in Figure 1.8. Such cracks do not normally develop unless the soil has clay content of at least 20%. Samples were taken and analysed at the CUT Soils Research laboratory. Suction tests indicated that heave potential was significant, but the geotechnical report, which had used Van der Merwe’s method to predict heave throughout the site, indicated 0% heave because the clay content was everywhere less than 12%. The sample taken for examination at the CUT laboratory was close to test pit 2-14.

Figure 1.8: Shrinkage cracks at the ground surface.

Van der Merwe’s method indicates zero heave for any soil with less than 12% clay (Figure 1.9). Significant heave damaged soon became apparent. This led the candidate to undertake detailed examination of soil samples taken from the site.

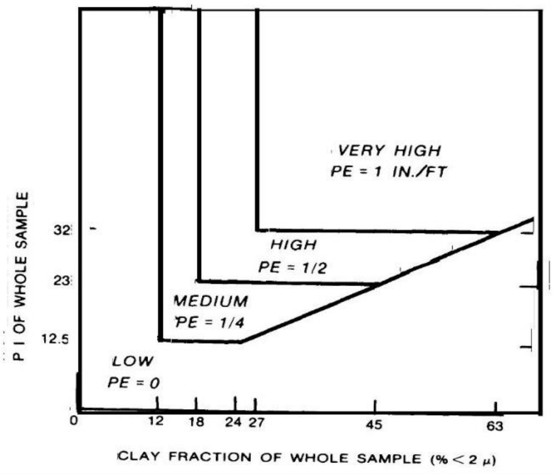


Figure 1.9: Chart for heave prediction by Van der Merwe (1964).

This chart is often used in a form slightly modified by incorporating two extra Skempton activity lines as shown in figure 1.10. All soils with clay content less than 12% are taken to have heave potential of 0 on both charts.

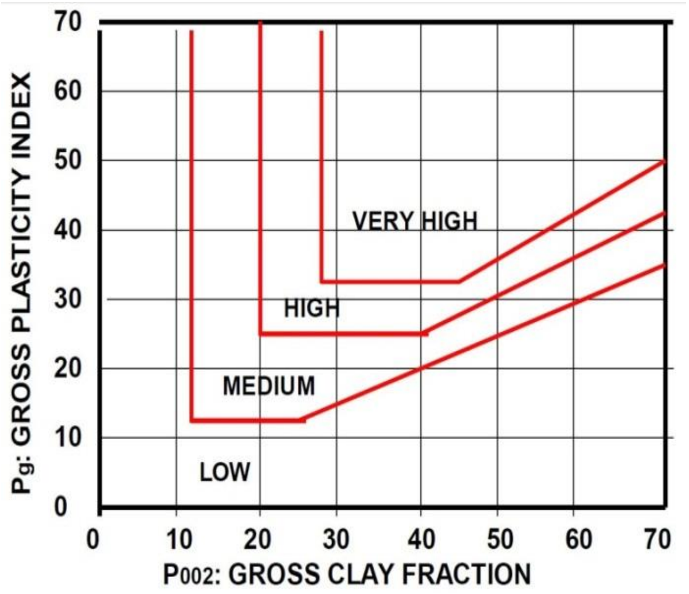


Figure 1.10: Modified Van der Merwe's chart (Williams and Donaldson 1980)

Visual examination of clay is commonly done using scanning electron microscopes. These have the advantage of providing the high magnification necessary to identify clay particles, which may have sizes less than 0.1 micron. Preparation of samples for scanning electron microscope (SEM) examination involves processes like freeze drying and gold plating. Unfortunately, this makes it very unlikely that any conclusions can be drawn from SEM images about the probable behaviour of clay particles in the hydrometer. In the hydrometer all particles are fully hydrated and suspended in water. Suspensions of clay cannot be examined by SEM, but they can be examined by light microscopes. Clay is not normally examined using the light microscope, apparently for five main reasons:

- On drying clay binds together into masses whose structure is opaque and not easy to discern.
- Particles of most types of clay range from about 2 microns down to 0.1 micron in size and the lower part of this range is normally considered beyond the range of the optical microscope.
- Suspended clay particles smaller than 1 micron are subject to vigorous Brownian motion and focussing on them is difficult or impossible.
- The high-powered objective lenses necessary to resolve particles as small as clay have very small depth of focus making observation of suspended particles difficult.

- Even under high magnification it is difficult to distinguish between clay particles and silt particles of very small size.

These problems were addressed by using a digital camera mounted on the microscope and by using methylene blue ($C_{16}H_{18}N_3SCl$) to label the clays for positive identification. The digital camera allows digital magnification leading to a meaningful total magnification while using an objective of only moderate power and more suitable depth of focus. Methylene Blue (MB) molecules exchange places with exchangeable cations in clays but are not attracted to non-clay minerals. High Cation Exchange Capacity (CEC) clays become stained dark blue with a small addition of the dye. Low CEC clays require far more MB to become visibly stained and usually appear light blue by the time sufficient MB has been added to turn high CEC clay particles very dark and opaque.

In addition to the Lerato park sample, a number of soils from the Free State, Limpopo and the Western Cape were examined. It was found that the procedures of SANS 3001 GR3 are inadequate in all of the cases examined. In all cases the dispersion of clay particles was not complete. Sand and silt particles often showed coatings of clay of both high and low cation exchange capacity. Agglomerations of clay particles were common, agglomerations of clay and silt particles were common. In some cases, the dispersed clay particles, which hydrometer analysis would take to constitute the clay fraction, were significantly outnumbered by the clay particles adhering to larger particles, which would probably not be assessed by the hydrometer as clay.

Tests were performed to examine the effect of increasing the time of soaking in dispersant, increasing the concentration of dispersant, increasing the quantity of dispersant used for soaking and increasing the time of mechanical agitation. It was found that these measures made little or no visible difference to the results. It was concluded that the hydrometer is likely to under-estimate clay fraction in almost all cases, and in many cases that under-estimation is likely to be substantial.

The sample from the Lerato Park housing project was one of the samples which showed very poor dispersion in spite of chemical and mechanical treatment.

Suction test results for this soil also indicated that heave potential was significant. The geotechnical report for the housing project therefore appeared to be very misleading. The housing authority was advised of these findings but no action was apparently taken.

Within two years of completion of the project there were public calls for the whole housing development to be declared unsafe and to be demolished. Large-scale repair and re-building was undertaken. This expensive disaster could be blamed on the misleading results produced by hydrometer analysis. It is likely that many other failures have been caused, and will be caused in future, by relying on hydrometer analysis together with popular methods to estimate heave potential. The findings of the microscopic investigation were published in the Journal of the South African Institution of Civil Engineering in 2016 and are presented in Chapter 8.

The above investigation had shown that clay particles form agglomerations among themselves and with silt particles in spite of prolonged chemical and mechanical dispersion, but it was not obvious how these agglomerations would settle in the hydrometer. It seemed likely that agglomerations might settle more slowly than single particles of the same size due to their uneven profile producing increased drag. In addition, water held between individual soil particles might lead to a lower aggregate density. To investigate this question P.K. Monye undertook to examine it as a Master's project under the candidate's supervision. The results of the first part of her investigation (using the dispersant and mechanical agitation specified in SANS 3001-GR3) was presented at the 1st. Southern African Conference on Geotechnical Engineering in 2016. The findings tend to confirm that agglomerations settle more slowly than individual particles of the same size, that much of the clay will be counted by hydrometer analysis as silt, and consequent under-estimation of clay fraction will vary significantly from one soil to another.

The findings of Chapter 8 also led to an invitation from Professor S.W. Jacobsz to assess the effectiveness of laser scattering particle-sizing at the Soil Mechanics laboratory of Pretoria University. The findings of this investigation are presented as Chapter 9.

1.11 Summary of Research Problem and Progress Towards its Solution

The problem investigated by this thesis entails a) identifying deficiencies in current procedures for assessing expansive clays, since they are evidently not reliable enough to enable rational design to provide economically feasible foundations with an acceptable probability of an acceptable lifespan, and b) the possibility of finding reliable alternative procedures for this purpose.

Currently the only procedure widely considered able to give reasonable probability of an acceptable lifespan is the removal of clay and replacement by inert fill (Lawson, 2006). This is not an economically feasible alternative for many projects. The papers presented in chapters 2 to 12 of this thesis demonstrate that two major features appear to play a very significant role in the inadequacy of current procedures. These are a) inability of current particle size determination procedures to correctly measure clay fraction and b) variability in heave potential of clay soils over spatial ranges of apparently fractal proportions. The papers presented show that the errors in clay fraction determination can be very large and the coefficient of variation can be extremely large. A solution to the problem of particle size distribution for soils which contain a significant clay fraction has not yet been found, and until such a solution is forthcoming the only answer offered is to avoid methods of assessment which rely on PSD.

A solution to the problem of large COV of expansive potential has been suggested, i.e. the use of Reliability Based Design. It appears that the multiple small-scale suction potential measurement procedure described in Chapter 7 can quickly and easily give suitable inputs for Reliability Based Design, which should be able to provide an acceptable probability of failure for light structures on expansive clays.

CHAPTER 2: SOME SHORTCOMINGS IN THE STANDARD SOUTH AFRICAN TESTING PROCEDURES FOR ASSESSING HEAVING CLAYS

2.1 Abstract

Design of foundations for most light structures in South Africa, and in particular for low cost housing, relies heavily on particle size analysis and the determination of Atterberg limits. The tests for these properties are currently performed in commercial materials testing laboratories using the procedures of CSIR's *Technical Methods for Highways Part 1 (TMH1)*. SANS 3001 is being phased in to replace TMH1. Both are primarily concerned with road construction. Investigations at the Central University of Technology indicate serious shortcomings in both of these norms in the context of foundation design for light structures. Highly plastic material is not usually used as road construction material, and these methods may be adequate to simply identify material so plastic that they should not be used in road construction. Structural foundations, particularly for low-cost housing, do not usually have this option; it is essential to estimate the actual heave potential. This investigation suggests that some of the changes proposed in SANS 3001 may be beneficial for heave assessment but the most likely application of SANS 3001 could be unsatisfactory in many cases.

Keywords: Expansive clay, TMH methods, SANS 3001, Liquid Limit, Plasticity Index, Linear Shrinkage

2.2 Introduction

Semi-arid and dry sub-humid areas in South Africa and many other parts of the world are noted for heaving foundation problems. Intense seasonal rainfall and lengthy intervals of drought lead to soils alternating between desiccation and saturation. Some types of clay minerals, particularly the smectite group, change volume powerfully with change in moisture content.

The behaviour of clay depends on its physical and chemical make-up. Detailed analysis of the structure and composition of clay is time-consuming and depends on sophisticated apparatus and highly skilled personnel. Experience has shown however that valuable insights can usually be gained by simple tests performed with inexpensive equipment. The tests on which most engineers in South Africa make their assessments are detailed in CSIR's *Technical Methods for Highways No. 1* (CSIR, 1986). SANS 3001 *Civil engineering test methods* (SABS 2011) is being phased in to replace the TMH1 methods. The SANS 3001 procedures are similar to the TMH1 methods with a number of modifications. There is considerable resistance to the introduction of SANS 3001, since some of its procedures are commercially unattractive.

The methods examined here are:

(1) Liquid Limit - TMH1 Method A2 (SANS 3001 GR10, GR11, GR12)
(2) Plastic Limit and Plasticity Index – TMH1 Method A3 (SANS 3001 GR 10, GR11, GR12)
(3) Linear Shrinkage - TMH1 Method A4 (SANS 3001 GR 10).

Note: Linear Shrinkage determination was removed from TMH1 in the latest edition, but it has remained as one of the “foundation indicators” (TMH1 (1986) A1 – A6) offered by commercial laboratories. Its re-introduction in SANS 3001 GR10 is stated to be for assessing the shrinkage product of wearing course gravels. The methods of preparing samples on which these test are performed (TMH1 A1 (a), SANS 3001 GR1, GR2) play a vital role in the effectiveness of these tests and some aspects of their influence are considered in *Preparation of material*.

It appears possible that the above methods (in TMH1 form), if strictly followed, may give acceptable results in many cases for highway applications. Highly plastic clay is usually rejected as a construction material in highway construction. The TMH1 and SANS 3001 tests may be adequate for simply identifying materials so plastic that they must be rejected or given special treatment.

Light structures in general, and low-cost houses in particular, have limited prospects for removal of problem material. Removal is too expensive and the engineer must usually design foundations which can cope with heaving clays, rather than avoid them. To make matters worse, very little of the budget is dedicated to soils testing and the most economical procedures are favoured.

These considerations are particularly relevant to Housing Subsidy Scheme projects, where many houses have suffered severe distress due to the activity of heaving clays. Poor assessment of heave potential due to shortcomings of current testing procedures may be one of the reasons for this. It should be noted that it is possible to have soils tests performed by academic institutions and specialized geotechnical laboratories. These services appear to be rarely, if ever, sought in the case of economic housing projects since they come at a premium and the allocation for geotechnical investigation is extremely small. Only commercial materials testing laboratory services are considered in this investigation.

2.3 Reasons for the Current Investigation

Concern over a large number of failures in subsidy houses led to a series of tests being carried out on clay soils from a development project at Botshabelo in the Free State. Samples of a raw, un-dried, un-sieved soil (extracted from a test pit in a very moist condition) consistently gave a Liquid Limit (LL) of 67 and Plasticity Index (PI) of 43 with scatter in results of ± 1,5. The same soil, tested in an accredited materials laboratory using their normal preparation method (stated to be TMH1 A1 (a)) gave LL 50 and PI 25. When the commercial values were used in the heave prediction methods of Van der Merwe (1964) and Savage (2007) the estimated heave was approximately half that given by the “raw sample” values. Reasons for the apparent anomaly were sought. One possible source of error is the Casagrande apparatus specified by both TMH1 and SANS 3001. It is widely considered to be susceptible to operator bias, and has been replaced by falling cone apparatus in the standards of many countries. The possibility of using this apparatus in South Africa has been examined by Sampson and Netterburg (1984), but their conclusion was not very attractive. Correlations with TMH1 and standards from elsewhere were not always straightforward, preparation of samples for testing had certain disadvantages and it was not preferred by testers. The fall cone was not adopted in SANS 3001. Its suitability for South African use is currently being re-investigated at the Central University of Technology (CUT) in Bloemfontein.

To gain an approximate measure of operator bias and other possible sources of error in the current procedures a team of six testers tested a range of clay soils at the geotechnical laboratory of CUT. The number of testers was reduced to four after the general magnitude of operator influence had been established. These tests suggest that there are, indeed, deficiencies in the usual implementation of the TMH1 procedures, but operator error cannot account for their magnitude. Some of the deficiencies may have been addressed in the SANS 3001 methods, others not. It is proposed here to examine the more familiar TMH1 Methods first and then consider the implications of the SANS 3001 modifications.

2.4 Preparation of Material

Preparation of material for grading and indicator tests is specified in TMH1 Method A1 and SANS 3001 GR1, GR2, and GR5. Since the commercial “foundation indicator tests” employ Method A4 from TMH1 (1986) this is the version here considered. Method A1(a) includes steps 3.4 *Boiling and Washing* and 3.5 *Drying and disintegration of fines*. Personal discussions with laboratory personnel and comparison of results from commercial laboratories with those obtained at CUT and at the Soil Science Department of the University of the Free State suggest that these two steps may be omitted in many cases. These steps consume time, laboratory space and energy. This investigation found that in many cases this will lead to only small differences in the Atterberg Limits. In other cases, however, the omission of these steps may lead to their under-estimation as well as under-estimation of the clay fraction, see *Influence of preparation procedures*. Jacobsz and Day (2008) raised concerns about the reliability of results from commercial laboratories, and noted at least one laboratory estimating linear shrinkage by dividing the PI by two. If short-cuts are made in the preparation of samples then other properties could also be unreliable.

SANS 3001 describes three different methods of preparation; GR1 for wet preparation, GR2 for dry preparation and GR5 for wet preparation at low temperature. GR5 is recognition that something may be unsatisfactory with the normal preparation procedures, but its implementation is optional *"when it is expected that heating of the fines will significantly alter their properties"*. No mention is made of who is responsible for the decision to employ this procedure. While one would expect the engineer to take this responsibility, when a commercial laboratory manager was asked about this he reported that he is often told to make all relevant decisions since he is a testing specialist.

The GR5 procedure is very time consuming. In the case of high plasticity clays, the procedure took typically 3 days in the oven at 45 degrees, one week or more out of the oven. While the oven is set at 45 degrees it cannot be used for its normal purposes and basins of suspended fines take up a large amount of space. This method of preparation would therefore involve higher costs than GR1 or GR2, leading to the following considerations: - i) It is unlikely that the testing technician would be given the responsibility of choosing to do such a protracted test. ii) The laboratory manager could incur the displeasure of his client if he presented a bill more expensive than expected. iii) To prove that the procedure was necessary, it would be necessary also to prepare and test samples using method GR1 and demonstrate a significant difference. iv) Few clients are likely to be in a position to assess heat sensitivity and actually specify the procedure. v) Most clients are very concerned about keeping costs low. Such considerations suggest that method GR5 will not be performed by the majority of South African commercial laboratories – certainly not for low-cost housing projects.

2.5 Atterberg Limits

2.5.1 Liquid limit

The LL is determined by TMH1 Method A2, *Determination of the Liquid Limit of soils by means of the flow curve method.* The body of this method describes the determination of the flow curve by three tests with the Casagrande apparatus.

There is a note at the end of the method. *"It has been found that the Liquid Limit of certain materials is influenced by the mixing time. Hence it was considered necessary to stipulate a mixing time and a period of ten minutes was decided upon."* This key point will be addressed in *Testing of samples*.

Following the specification of the flow curve method is clause 5.2 *One-point method* in which it is noted that the Liquid Limit may also be determined using only one test. There is the provision in clause 5.6 *"In the case of dispute the flow curve method shall be the referee method"*.

Commercial laboratories strive to make their services competitive and affordable to their clients; they therefore almost exclusively opt for the one-point procedure. They also interpret the note concerning the mixing time of 10 minutes to mean that the total time, from adding the first drop of water to the fines powder until testing in the Casagrande cup, is to be exactly ten minutes.

SANS 3001 details three different tests for Liquid Limit determination, a one-point method (GR10), a two-point method (GR11) and the three-point flow curve method (GR12). There is close similarity to the TMH1 procedures and there is even stronger emphasis on the mixing time limit of 10 minutes. It is recommended in the introductions to both GR10 and GR11 that the three-point flow curve should be used where a PI greater than 20 is expected; but it is not specified who makes this expectation, nor is it specified as the referee method. Similar considerations to those which make the use of GR5 very unlikely apply to GR12. Most laboratories will probably continue to perform the one-point method in all cases. It should be noted that the most common method of estimating heave is that of Van der Merwe (1964). That method relies on the PI, which at the time the method was devised was always deduced from the 3-point flow curve and there was no ten-minute time limit.

2.5.2 Plastic limit and plasticity index

TMH1 Method A3 and SANS 3001 GR10 specify similar procedures for determining the PL and PI. Both specify the use of wet material remaining after LL determination and after filling of the shrinkage trough. This makes it likely that for expansive material the PL will be determined at least thirty minutes after saturation. This has important consequences as addressed in *Results and observations* and *Testing of samples* below.

2.5.3 Linear shrinkage

TMH1 Method A4 and SANS 3001 GR10 both specify placing the trough in the oven after filling. GR10 specifies that this should be done *immediately* after filling (section 6.2.2.1) and it emphasizes that there should be no air drying before placing in the oven. This is addressed in *Results and observations* and *discussion and comments*.

2.6 Tests Performed to Assess the Reliability and Consistency of these Specified Procedures

Samples were prepared according to TMH1 Methods A1(a) with and without steps 3.4 and 3.5 i.e. boiling, washing, drying and disintegration (corresponding to SANS 3001 GR1 and GR2). The samples were prepared from oven dried (105°-110°C) material passing the 0.425 mm sieve. Since comparison with commercial results, and with tests performed by the soil science department of Free State University, suggested that steps 3.4 and 3.5 were probably usually omitted in the commercial procedures, comparisons with commercial values shown here are from tests on samples where these two steps were omitted unless specifically noted. Tests performed on samples whose preparation included these two steps usually showed only small differences in Atterberg Limits but sometimes large differences in clay fraction. A range of soils, mostly from the central Free State, was used. These samples were drawn from one major geotechnical investigation and several smaller investigations in the central Free State and a handful of samples from the Western and Northern Cape and Gauteng. Screening tests on about sixty of the samples showed acceptable agreement with commercial laboratory Atterberg Limits for predominantly sandy soils while clayey soils gave less good agreement. From approximately 200 available samples, thirty predominantly clay samples covering a wide range of plasticity, location and sampling depth were selected for detailed testing.

Five samples were used for comparison of the one point and the flow curve methods. In the case of very plastic clays the discrepancy was considerable. It was deduced that at least part of this difference could be due to time dependence. The flow curve method provides more time for the sample to absorb water than the one-point method does. Tests were performed to monitor this time dependence for a range of 22 clayey soils - 20 from the central Free State, one from the Northern Cape, one from the Western Cape.

The procedure involved rapidly adding an amount of water commensurate with the commercial value of LL at the start of the test. This ensured that all of the dry fines came into contact with water at the start of the mixing process. In the commercial procedure water is added very slowly and in the case of very plastic material some of the fines remain dry far into the mixing process. Mixing was performed (with the addition of more water as needed) until the slurry was suitable for testing in the Casagrande apparatus. The time of adding the first water and the times of testing were recorded. Each tester determined the Liquid Limit four, five or six times. The first result was obtained as quickly as possible; the remainder were spaced over about one hour.

In the case of highly plastic material it was noted that water was initially absorbed by the fines very quickly. In transferring material from the mixing bowl to the Casagrande cup for the first test of each series it was not uncommon for the slurry to have stiffened beyond the point where a successful test could be made. It needed considerable practice to achieve a result in less than 5 minutes. After Liquid Limit tests the remainder of the paste was used for Linear Shrinkage and Plastic Limit tests.

2.7 Results and Observations

Figure 2.1 shows results of three point tests for a firm grey-olive residual clay from the Central Free State compared to one-point results from a commercial laboratory. The 3-point results give LL 71 ± 2 and PI 42 ± 1 as against the commercial laboratory value of LL50, PI 24.

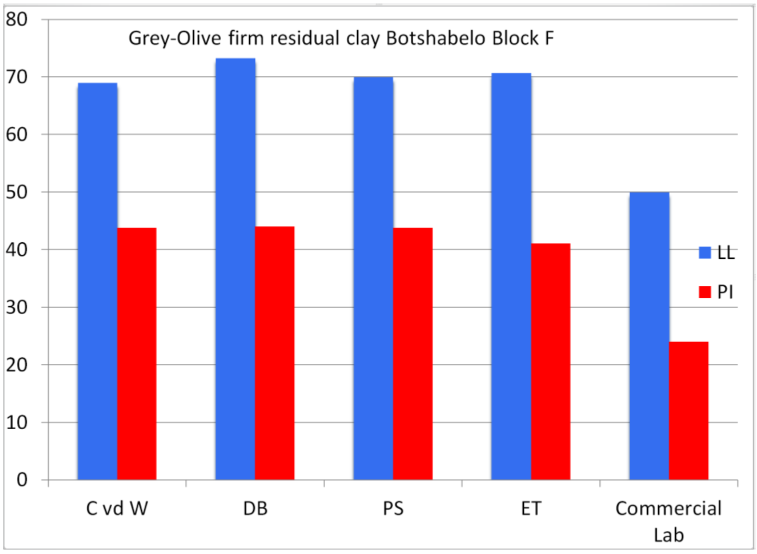


Figure 2.1: Three point tests compared to one-point results.

Table 2.1 summarizes the results for the five clays tested.

Table 2.1: PI: commercial one point and measured flow-curve values

Description	PI commercial	PI 3-point	Difference (3pt – c)/c
Black stiff transported clay	25	36 ± 2	44%
Grey-olive firm residual clay	24	42 ± 1	75%
Black firm transported silty clay	22	32 ± 2	45%
Orange-banded yellowish-grey stiff sandy clay	18	27 ± 2	50%
Dark brown firm transported sandy clay	17	22 ± 1	29%

Figure 2.2 shows typical plots of PI against time for four clays from the central Free State. Trend lines and upper and lower envelopes have been added. The trends of the envelopes and the best-fit (logarithmic) curves suggest an initial rapid increase in PI slowing noticeably after about 20 minutes. Rapidly changing PI appears to be accompanied by increased scatter. Results from individual testers suggest that values may not change smoothly with time.

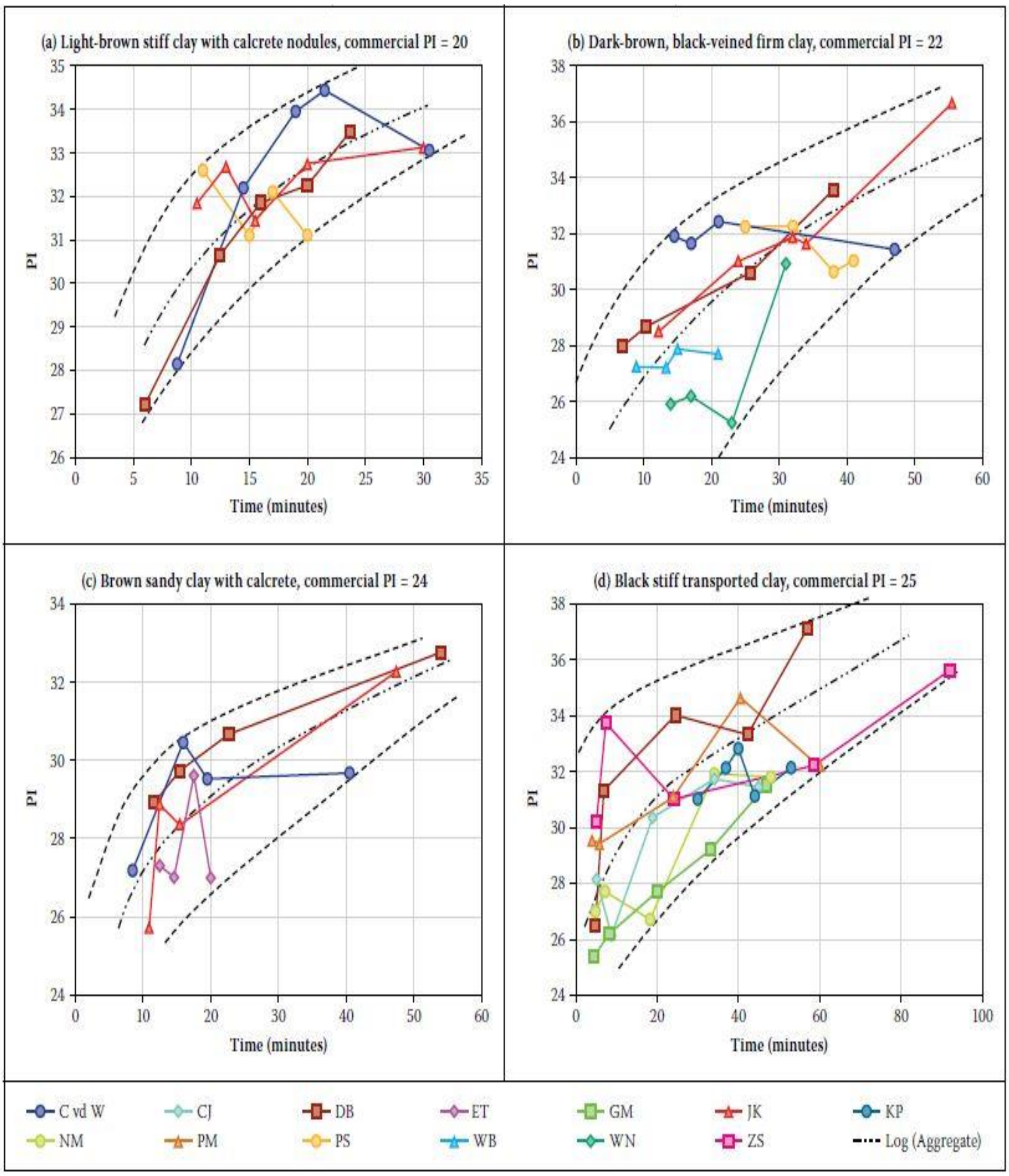


Figure 2.2: Plots of variation of PI against time for four clay soils.

Less plastic clays were found to give generally less time dependence, and also less scatter in results. Results from a non-expansive, low CEC, low suction-potential clay from the Western Cape can be seen in Figure 2.3. The small scatter allows the possibility of insight into operator bias. No attempt was made to rigorously assess this factor, but it appears to be too small to account for the observed discrepancies.

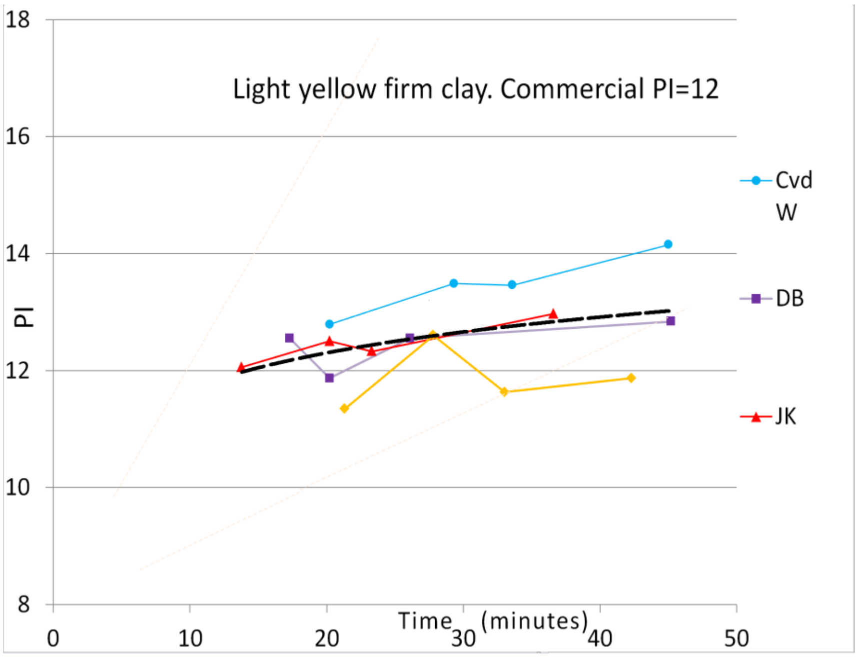


Figure 2.3: Variation of PI with wetting time for a non-expansive clay.

Little time dependence is evident in figure 2.3 and scatter of results is small; the values determined by individual testers suggest a range of operator bias in the region of ± 1. For the 22 samples tested the apparent increase in PI was less than 5 in 8 cases, between 5 and 10 in 9 cases and greater than 10 in 5 cases. Plastic Limits were found to correspond quite well with those from commercial laboratories. Saturated material remaining after completion of the LL procedure is used for PL determination; the time involved is probably sufficient to approach a stable moisture condition in most cases. It was found that Linear Shrinkage of highly plastic clays was poorly assessed using the stated procedure of placing the trough in the oven immediately after filling. Such clays tend to shatter and arch to such an extent that reconstructing the sample for final measurement is difficult.

Widely different values of LS were given by different specimens of the same clay. When samples were left to air dry for a period these problems reduce significantly. Samples of the same soil left for 24 hours, 48 hours and 76 hours before oven drying showed that the Linear Shrinkage value and the consistency of results increase with time of air drying. Samples left to air dry until shrinkage stopped rarely showed pronounced arching and sometimes reached Linear Shrinkage almost double the value of immediately oven-dried specimens. Air drying probably models shrinkage in the field far better than oven drying. Less plastic soils were less seriously affected by immediate oven drying. While the current LS procedures may be satisfactory for assessing the shrinkage product of wearing course gravels they are not at all suitable for the foundation indicator tests offered by commercial laboratories. Figures 2.4 and 2.5 show the effect of drying of the same firm grey-olive residual clay that was used in the LL and PI tests (Figure 2.1).

Figure 2.4: Oven-dried sample of. firm grey-olive residual clay

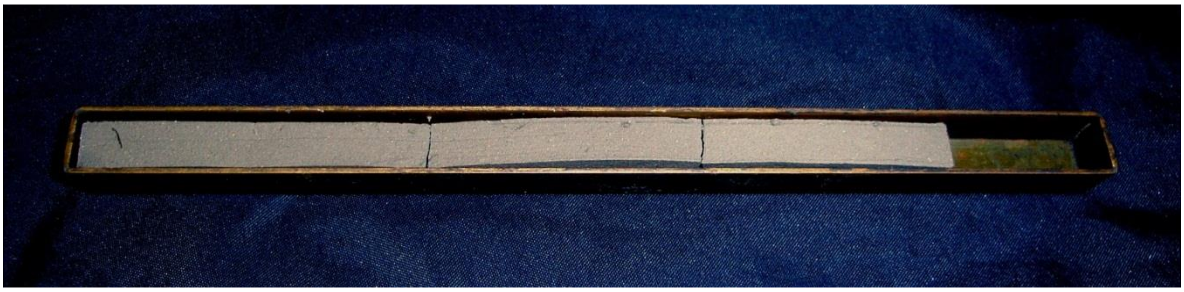


Figure 2.5: Air-dried sample of the same clay.

Figure 2.6 shows the linear shrinkage (air dried) of two clay soils. Free swell, CEC and suction tests suggest that the upper clay soil (commercial PI = 12) is non-expansive and the lower clay soil (commercial PI = 25) is very expansive. The Linear Shrinkage gives graphic confirmation.

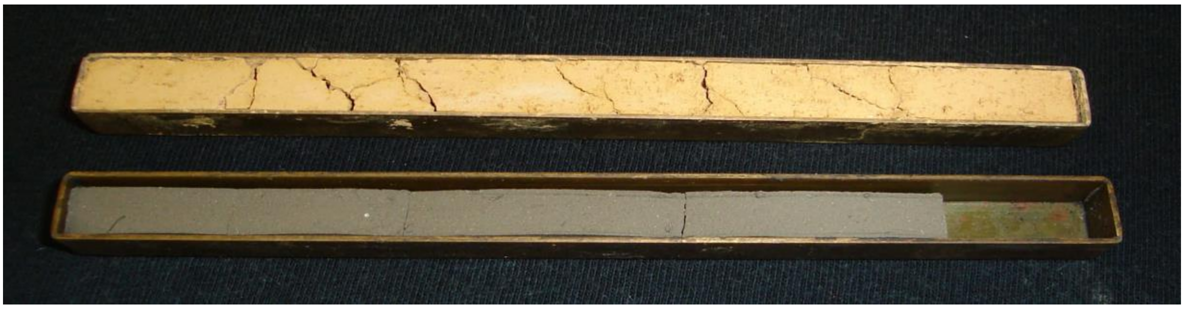


Figure 2.6: Contrasting high and low linear shrinkage.

Figure 2.7 air-dried samples of some of the soils used in this investigation – the range of linear shrinkage shown here is from less than 5% to just over 20%; the red clay second from left is a raw (un-sieved, undried) sample from Gauteng which received no preparation other than mixing with de-ionised water (raw PI = 42); the samples on either side of it were extracted by wet preparation from two Free State soils and were not added back to the dry-sieved fines (PI = 49 and PI = 46); raw samples typically show more fracturing than fines-only samples; linear shrinkage of air-dried samples corresponded well with several heave indicators not dealt with in the TMH1 and SANS 3001 procedures.

Figure 2.7: Shrinkage troughs with air dried samples.

2.8 Influence of Preparation Procedures

In many cases particularly for granular soils it was found that omitting steps 3.4 *Boiling and Washing* and 3.5 *Drying and disintegration of fines* from the TMH1 A1 (a) procedure (equivalent to using SANS 3001 GR2 instead of GR1) made little difference to the Atterberg Limits. In the cases where it did make a difference the effect on both Atterberg limits and clay fraction could be very significant. Investigation of these cases usually revealed a fraction of extremely active clays, having very high suction potential and able to bind fine material to form grains of high strength (tensile strengths greater than 7 MPa were measured in some cases). Grinding of these grains with a rubber-tipped pestle failed to reduce them from gravel to powder in a reasonable time. Up to 70% of some soils appeared to be gravel, but with wet preparation much of this material passed the 0.425 mm sieve. One such sample (Photo 5) was shown to a commercial laboratory manager. He assessed it as mudstone gravel not suitable for PI testing. In some cases, clay separated out during wet preparation was found to have very high LL and PI (LL>80, PI>40). This may be significant in view of the warning by Rogers *et al*. that the behaviour of any type of soil may be controlled by swelling clay if it contains more than 5% clay by weight. Figure 2.8 shows material retained on the 0.425mm sieve after prolonged grinding in the mortar; despite appearing to be hard gravel, more than 80% of this material passed the 0.425mm sieve with wet preparation.

Figure 2.8: Material retained on the 0.425mm sieve.

It was found that plastic materials which show time dependence after oven drying at 105° to 110°C showed little time dependence after GR5 preparation. This appears to confirm that the time dependence may be due to removal of water from the structure of the clay during oven drying. Procedure GR5 may be essential to obtain realistic LL and PI for highly plastic clay when the current ten-minute testing time is used, but allowing adequate time for water re-absorption may make GR5 unnecessary. The possibility that oven drying may permanently alter properties of some clays are currently being investigated.

2.9 Discussion and Comment

Commercial laboratories need to deliver competitive services. Clients are often under both time and economic pressures. Commercial procedures must therefore be as simple, quick and economical as possible. This investigation suggests that in many cases – particularly for sandy soils - procedures being commonly performed commercially may give acceptable results. In the case of highly plastic material they do not. The SANS 3001 procedures, if followed strictly, would possibly give satisfactory results in most cases. Discussions with commercial laboratory personnel suggest that it is unlikely that these procedures will be followed strictly because of time and cost implications and the discretionary nature of some procedures.

2.9.1 Preparation of samples

Certain materials will only give meaningful Atterberg Limits when wet preparation is performed. Wet preparation is time, space and energy consuming, and it appears that it may be frequently omitted in many laboratories. In some cases, omitting wet preparation may have minor consequences for the Atterberg limits and it may be primarily for high plasticity materials that it is essential. Its omission has drastic consequences for clay fraction determination in some cases.

Whether samples are prepared according to TMH1 A1 (a), SANS 3001 Gr1 or GR2, the specified one-point LL procedures give poor results for highly plastic clays. The use of GR5 (wet preparation at low temperature) appears to have the potential to solve this problem. However, the economic consequences of the method are such that it is almost certain that GR5 will not be used commercially. Small changes to the normal testing procedure may render GR5 unnecessary.

2.9.2 Testing of sample

2.9.2.1 Liquid limit

Some deficiencies apparent in the tests for Atterberg limits seem to be specifically dependent on the 10-minute mixing time. This limit may be linked to the findings of

Kleyn *et al*. (2009). Their paper deals with the assessment of decomposed dolerite gravel for use in road construction. It deals with materials of very low plasticity. They found an increase in PI from 2 to 7 with extended mixing. They express the opinion that prolonged mixing of the gravel may break down the grains and expose more of the dolerite's montmorillonite component to water and thus allow the mineral to exert its swelling potential. They give a graph of measured PI against mixing time which shows great similarity (though in a much lower range) to some of the curves found in this investigation for active clays.

Figure 2.9 shows the variation of PI as determined by one tester; (a) five data points; mixing time: 10–30 min for a decomposed dolerite gravel (Kleyn *et al.* 2009), and (b) for a plastic clay (tester C vd W from Figure 2.2 (a)).

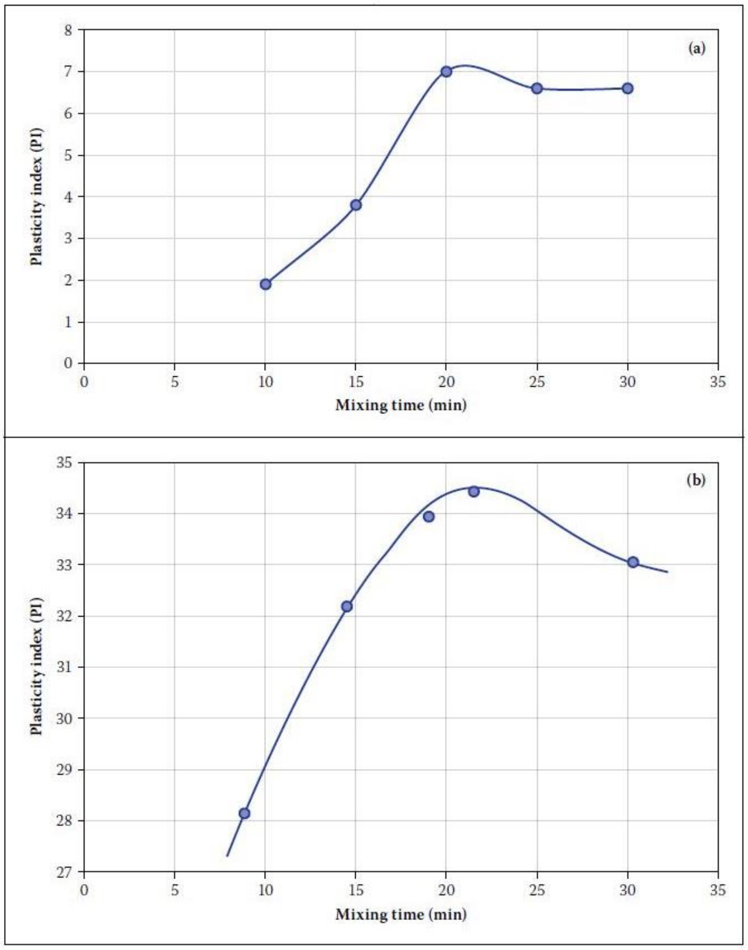


Figure 2.9: Time variation of PI for dolerite gravel and plastic clay

The mechanism proposed by Kleyn *et al*. (2009) is unlikely to be the correct explanation for the time dependence investigated here. Raw, un-sieved wet clay samples which required the addition of little water and little mixing time, gave values of LL and PI much higher than those given by a commercial laboratory which observes the 10-minute mixing time. In the timed tests LL and PI values were usually above the commercial laboratory’s values even when the mixing time was less than 5 minutes (Figure 2.2). This investigation suggests that the problem is not that of breaking down soil particles, but of not allowing sufficient time for reintegration of water into the structure of the dried clay before performing the test. The similarity of the Kleyn *et al.* (2009) curve to some of those in this investigation suggest that the change in PI of their samples could be at least partly due to the montmorillonite fraction being given time to absorb water, and that such material might be less suitable for road construction without treatment than the ten-minute test suggests. Even if Kleyn *et al*. (2009) are correct that longer mixing breaks some of the particles, tests at CUT have found decomposed dolerite to be porous. The montmorillonite fraction will therefore eventually gain access to water and swell in any case.

2.9.2.2 Plastic limit

The PI is the difference between the LL and the PL. The PL is determined using material left over from the LL test after the filling of the shrinkage trough. The procedure makes it almost certain that the sample will have been fully saturated for a considerable period before the PL can be obtained. This investigation found acceptable agreement with commercial laboratory PL values whether LL-time-dependence was present or not. As a consequence, the differences in numerical values of PI and LL are comparable; but the percentage differences are far greater. For the clay mentioned above, the commercial laboratory values were LL50, PL25, PI 25, whereas the values for the un-dried sample were LL67, PL24, PI 43. The PL differs by only 1, the LL differs by 17 (i.e. 34% above the commercial lab value) and the PI differs by 18 (i.e. 72% above the commercial lab value). The PI is the most commonly used indicator of potential heave, and this discrepancy is very serious.

2.9.2.3 Proposed amendment to liquid limit procedure

A simple procedure is proposed to address this problem; a procedure which meets the 10-minute mixing requirement of TMH1 and SANS 3001. It is suggested that it could be used until an alternative can be standardized. It is very easy for an experienced tester to recognize a sample which will have a high LL; it is usually obvious with the first addition of water to the oven-dry fines. While not all soils with high LL show time dependence for water intake, all of the clayey soils which this investigation found to be markedly time-dependent do have a high LL. It is proposed that as soon as a tester suspects a high LL, then sufficient water to bring the sample close to the LL be added in the first 5 minutes of mixing. The sample should then be covered and set aside for at least 30 minutes (allowing the testing of another sample), after which the mixing procedure should be resumed for a further 5 minutes and the remainder of the test continued as normal. Section 6.1.10 of SANS 3001 GR10 should be ignored. The economic implications of this proposal would be small. Probably only a small percentage of samples would need this treatment, which is itself minor. It is far more likely to be embraced by commercial laboratories than the blanket application of SANS 3001 to all samples. Simple procedures for identifying soils which need special treatment, and inexpensive procedures for dealing with them reliably and effectively are being sought. It is hoped that they can be incorporated in an economically viable revision to SANS 3001.

2.9.2.4 Linear shrinkage

Soils containing very expansive clays contract so violently under oven drying conditions that they often bend, shatter and produce very unreliable results in the Linear Shrinkage trough. A trough without a base is specified for the case of materials expected to have PI greater than 20. This type of trough reduces bending and shattering and may give better results for highly plastic soils. The “expectation” leading to its use would indicate a flow curve LL - with the same considerations as in *PREPARATION OF MATERIAL* above. Even if it were actually used in the case of plastic clays, immediate placement in the oven at 105°C would guarantee a poor indication of shrinkage potential.

To be of value for the assessment of clays the trough should not be put into the oven until the shrinkage limit has been almost reached. With this procedure the LS gives a consistent indication of shrinkage from Liquid Limit to Shrinkage Limit and probably models field conditions reasonably well. Linear Shrinkage appears to have potentially more value for heave prediction than is currently being utilised. Page Green and Ventura (1999) demonstrated that the Linear Shrinkage test can give valuable insights for low plasticity road-building materials. Cerato and Lutenegger (2006) demonstrated that it can give a fair approximation to the full shrinkage curve. Investigations in progress at CUT suggest that it can indicate cases where current methods fail to show true heave potential and give confirmation when they do.

2.10 Comparisons with Standards from Elsewhere and Previous South African Findings

The New York State Department of Transportation's specification for liquid limit determination, Geotechnical Engineering Bureau (2007) recommends that where possible clays should be tested in their natural, un-dried state (section 4. Preparation of test samples). Where this is not possible section 6.2, Note 1 states *"Allow ample time for mixing and curing since variation can cause erroneous test results. Some soils are slow to absorb water. Therefore, it is possible to add the increments of water so fast that a false liquid limit value is obtained. This is particularly true when the liquid limit of clay soil is obtained from one determination as in the one-point method*."

There is recognition of time dependence without warning that it is precisely the most problematic clays which are at issue. There is also no warning that the "false liquid limits" are always under-estimates and that they may be very severe under-estimates. There is no guidance on how much time might be "ample time", but there certainly is a clear warning about the inadequacy of the one-point method for clays. Rather than suggesting that extended mixing time affects the result by breaking down soil particles the suggestion is that the time required for absorbing water into the clay is the important factor.

Professor JE Jennings, one of South Africa's most notable soils experts, habitually advised his students to allow "ample time" for mixing and curing - also without specifying just how much time is "ample".

Indian Standard (IS 2720 (Part 5) 1985 - Reaffirmed 1995) does give specific guidance. Clause 3.4.1 states "*In the case of clayey soils, the soil paste shall be left to stand for a sufficient time (24 hours) so as to ensure uniform distribution of moisture throughout the soil mass."*

This again suggests that the time for which the sample is left in contact with water is important, rather than that mixing may break down soil particles. While this specification recognizes the possibility of time dependence and may be a safe way to proceed, there may be few South African soils for which such lengthy curing is essential. Storage of slurried samples for 24 hours needs additional containers, labelling, record-keeping and storage space. S.A. standards frequently follow British standards quite closely. BS 1377 also specifies a far more reasonable time frame than TMH1 and SANS 3001.

Blight and Cheong (2012) note that it is well known that drying (even air-drying) of some soils affects their properties and can lead to underestimate of LL and PI. They recommend that Atterberg limits should therefore be performed without any drying wherever practicable. This requires testing soil in a state as close to its natural condition as possible, with only minimal processing for removal of large particles. They point out that extended mixing (of soil in its natural condition) can lead to increased LL and PI because of breaking down of cemented bonds between clay clusters (a well-known feature of tropical and volcanic soils). This series of tests found that where clay samples can be tested in the raw state (with only removal of large particles where necessary) PI for the whole sample could be considerably greater than that of the fines alone in the standard test.

2.11 Some Theoretical Considerations

Non-expansive clays, typically Kaolinite, consist of alternating Silicon-based tetrahedral sheets and Aluminium-based octahedral sheets. They form a structure in which hydrogen bonds hold the pairs of sheets in stacks of the order of 1000 layers thick. When a soil containing such clay absorbs moisture, the water fills the spaces between the individual soil crumbs, but does not penetrate between the individual sheets of the clay structure itself. The clay does expand somewhat (despite being called “non-expansive”) since the small size of the clay particles (<0.002mm) leads to capillary suction which draws water into the tiny pore spaces quite strongly. This suction is, however, not usually severe enough to cause expansion against the pressure under the foundation of a building.

Expansive clays (e.g. smectites), on the other hand, consist of silicon-based tetrahedral sheets and aluminium-based octahedral sheets which have suffered so much corruption to the ideal structure (e.g. by substitution of aluminium^{3+} for silicon^{4+} in the tetrahedral sheets and magnesium^{2+} for aluminium^{3+} in the octahedral sheets) that the pattern of hydrogen bonds is disrupted and layers become attached largely by electrostatic forces. The number of layers in the stacks of these sheets may be very small, giving a thin, plate-like structure to the clay particles. When such clay is allowed to take in moisture, not only the pores between individual grains are filled, but water is drawn in between the individual layers, which move apart to admit water-borne cations attracted by the unbalanced electrostatic forces on the sheets. The spacing of layers is of molecular dimensions. The suction is extremely high, so the expansion can take place against a considerable pressure, but the spacing is so small that an appreciable amount of time may be needed for water to be drawn in. Such considerations lead to the question of how long it takes for water to be drawn into the structure of expansive clays, and whether the procedures of TMH1 and SANS 3001 allow sufficient time for this hydration to take place.

2.12 Conclusions

The TMH1 methods have long been standard in commercial laboratories throughout South Africa. The way these methods are currently being applied is convenient and economical but unsatisfactory for assessing clays under foundations of light structures. The introduction of SANS 3001 to replace the TMH methods might address shortcomings in this regard if all of its procedures were strictly employed. The discretionary nature of critical procedures and their cost implications, however, makes their employment unlikely. Introduction of procedures to address the cases where the TMH methods are not satisfactory is suggested as an alternative at least for the “foundation indicator” tests. It appears that relatively small modifications to the current methods may be sufficient in many cases to provide more reliable values for these indicators for active clays. These minor changes would have relatively minor economic implications and would be more attractive to commercial laboratories and their clients than SANS 3001 in its current form.

Two further points are offered for consideration:

(i) Kitcher (1980) showed that records of damage to small houses on heaving clays suggest R 2150 spent on geotechnical investigation per house would be economically justified to eliminate future repair costs. The figure currently allocated for subsidy housing is approximately R104 per house. The cost of heave damage at one 500-unit housing project visited by the CUT Soil Mechanics Research Group in 2013 was more than R14 000 000.

(ii) If geotechnical investigations are to be entrusted to engineers with sufficient skill and experience to make valid assessments of what tests must be performed, what procedures must be followed, and what conclusions can be drawn from the results, then a realistic allocation of funds will be needed.

CHAPTER 3: SHORTCOMINGS IN CURRENT METHODS OF IDENTIFICATION AND ASSESSMENT OF EXPANSIVE CLAYS.

3.1 Abstract

The problem of damage to buildings caused by heaving foundations remains a challenge in Africa. Recent developments in unsaturated soil mechanics have led to theoretically convincing tools for dealing with most unsaturated soils problems, but the parameters required to apply these methods involve tests so lengthy and expensive that they are not feasible at this stage for economic housing projects. Widely used empirical methods rely on simple, economical tests, but procedures usually involved in these tests are primarily designed for road-building rather than housing foundation purposes. There is a need for simple and economic procedures tailored to the specific needs of light structure foundations.

3.2 Introduction

The problem of providing economic housing for lower-income communities is widespread throughout Africa. In South Africa the government is attempting to provide small subsidised houses for the very poor, while the economically emerging see building a home for themselves as a high priority. Much of Africa has semi-arid and sub-humid conditions which lead to generally shallow residual soils subject to seasonal alternation of desiccation and saturation. Such conditions are known throughout the world for giving expansive foundation problems. The necessity of reserving easily worked, fertile soils for farming, and of avoiding problem soils for essential and expensive services like roads, and airports may result in problematic soils being allocated to housing by default. In the 1960s and 70s a great deal of work was done towards understanding and designing for heaving clays. Much of the development in this field was funded by large mining companies seeking solutions to problems with the housing they were providing for their work force. Economic housing does not usually have the financial backing of such companies today.

At the 7th Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Pidgeon (1980) noted 14 methods of foundation design from America, Australia and Africa and presented a comparison of the five most promising among them. Since that time great strides have been made in the whole discipline of unsaturated soil mechanics, of which expansive soils have traditionally featured as a major part. It would be expected that in the 35 years since that time the problems associated with foundations on heaving clay would have been solved. Lawson (2006) makes it clear that this is not the case. He observed that even among experience senior engineers who spend most of their time working on expansive soil problems the only design solution which is considered reliable is removal of the expansive soil and replacement with inert fill.

The situation was even more clearly highlighted by DG Fredlund (2009), perhaps the world's leading authority on unsaturated soil mechanics, who noted: "I started in unsaturated soil mechanics by looking at swelling clay problems, where if you get involved you usually lose money, it is not a good problem." He then suggested that geotechnical engineers turn to more profitable problems like environmental protection. This suggests that the problem of heaving foundations is not even close to being solved.

It is therefore not surprising that many economic houses are still being destroyed by expansive clays. The authors have visited one housing project where almost 30 percent of the houses had to be demolished after becoming unserviceable due to heaving foundations. At another project a house suffered such serious heave that it lost its structural integrity before construction was finished. It had to be demolished not many days after the first brick was laid.

Although this may not be financially a good problem to get involved with, it would not be ethical to simply turn our attention to more profitable geotechnical problems. There is a need to find a solution which can lead to acceptable foundations in at least a large majority of cases. While the authors' experience in this is primarily in South Africa, their findings may be relevant to other countries since similar conditions are quite widespread in Africa.

3.3 Shortcomings in Available Methods for Dealing with the Problem

The most advanced methods available are mathematical. One need only to specify the governing partial differential equations, the boundary conditions of the system, the relevant parameters for the equations and readily available software will deliver a solution. Unfortunately, as Fredlund (2009) has pointed out, the soil water characteristic curve (SWCC) is the pivotal input parameter, for which he indicated a typical cost of US $10 000. While approximations can be made to the SWCC for preliminary design, Blight (2013) noted that if it is needed at all, then it must be measured, since estimated values can only serve as very preliminary guides. For some of the seriously expansive clays tested by the authors, more than six weeks in the pressure plate apparatus was needed to measure just the first (0-900 kPa) section of the SWCC. Such a test is not feasible for small housing projects from both cost and time perspectives. Other parameters besides the SWCC must also be determined. They also require considerable time and expense.

Alternatives to a fully mathematical unsaturated soil mechanics analysis are considered in Nelson and Miller’s (1992) standard text on expansive soils, which is widely considered the most comprehensive and authoritative guide available for expansive foundation problems. Nelson and Miller (1992) consider three general approaches to heave prediction: the use of the oedometer, the use of shrinkage / suction curves, and the use of empirical methods.

Several procedures for estimating heave potential by means of the oedometer have been devised. But there are well known shortcomings even for relatively easily-handled soils. It is difficult to get undisturbed samples to completely fill the odometer ring and imperfections in the cutting of samples lead to spurious results. Tests by the authors have shown that expansive clays may take up to two months to stop expanding after inundation under nominal loading. A meaningful oedometer test on such clays is not feasible for a low-cost housing project. Nelson and Miller’s (1992) examination of the odometer is lengthy, but the conclusion is that it is not the best tool for solving the heave problem.

Nelson and Miller (1992) indicate that the CLOD test, which entails direct measurement of the shrinkage curve, may be the best method available. Unfortunately, it involves coating samples with Saran resin, which is not widely available in Africa; it is dangerous from a health point of view, and involves tests spanning six weeks or more. Less toxic substitutes have been proposed to replace the Saran resin, but the time for the test remains prohibitive. It has not been widely adopted in the design of foundations in Africa.

Alternative methods for determining the shrinkage curve have been tried. In Morocco, Bensallam *et al.* (2012) devised a method using direct measurement of un-coated soils. No indication was given of the total time for a test, but they did indicate six hourly intervals between readings and a final stage of 3 days' oven drying of the whole apparatus. This suggests a possible time-frame of the order one or two weeks. This may be more feasible than the clod test, but the method is skills, labour and equipment intensive, it does not appear to have become popular and does not seem likely to be adopted by commercial laboratories.

It is therefore not surprising that the procedures widely used in practice depend on empirical methods rather than on direct measurement of heave-specific parameters. This approach bases estimates of heave potential on simple, quick, economic tests, particularly the Atterberg limits and particle size analysis. The most popular method in South Africa, and the only one dealt with in detail in Nelson and Miller's (1992) summary of empirical methods, is that devised by Van der Merwe in 1963. Two improvements have been proposed - by Williams (1980) and by Savage (2007) - but the method remains basically the same. After fifty years of major technological progress no new method appears to have been devised which has demonstrated its superiority to the point of convincing a majority of engineers to use it. Paige Green (2004) noted that it is “somewhat worrying” that little development has taken place in this regard since the 1980s. Despite half a century of experience with these empirical methods there are still large numbers of houses suffering heave damage, becoming structurally unsound within a small fraction of their design lifespan.

Investigations by the authors suggest that a major contributing factor may be that the procedures for determining the basic parameters are seriously flawed. The soils testing procedures currently in use in commercial engineering materials testing laboratories are primarily designed for road construction. Road construction is concerned primarily with coarse-grained material; clay is usually rejected or is given stabilization treatment of some kind. Sample preparation and testing procedures which are suitable for granular soils are used for all samples, even for those not intended for road construction.

3.4 Currently Used Parameters for Foundation Design

The parameters most frequently used currently in empirical methods of heave estimation are the Liquid Limit (LL), the Plastic Limit (PL), the Plasticity Index (PI = LL-PL) the Bar Linear Shrinkage (LS) and the percentage clay fraction. The present investigation has found the current procedures for preparation of materials and laboratory testing to be usually adequate for assessing typical road construction materials, but very inadequate for assessment of heave potential in clays.

3.4.1 Liquid limit, plastic limit and plasticity index

Sample preparation involves oven drying of soils, crushing, sieving and testing of the fines fraction (the fraction passing the 425-micron sieve). The LL is found using a one-point determination with a mixing time, from oven dry to testing in the Casagrande cup of 10 minutes. Material remaining is used to fill the linear shrinkage trough and to perform the PL test. Moisture contents are determined by oven drying at 105°C to 110°C until constant mass. The PI of the fines fraction is determined by subtracting the PL from the LL. This value is reduced by multiplying by the fines fraction to give the gross PI of the whole sample.

Road construction involves re-working material, which is commonly transported and re-compacted in a new position. Current sample preparation methods may be a reasonable representation of such conditions. Economic housing does not allow for large-scale disturbance of the soil under foundations. The only disturbed and transported material is normally a layer of selected fill under floor slabs.

There have been warnings (Blight, 2012), that inappropriate preparation procedures may significantly reduce PI, particularly for the residual clays which are common in semi-arid and sub-humid regions in Africa. Results of tests prepared using the common standard procedures may therefore be relevant for road construction purposes, but not for foundations of houses. Several samples of clays from a number of different subsidy housing sites were tested using “raw”, un-dried, un-sieved samples to which the only treatment was removal of visible large particles and the addition of sufficient water to enable a LL determination. In each case the “raw” PI for the whole sample was significantly greater than the PI of the fines alone from the corresponding sample prepared using the standard procedure.

Further tests showed that when dried fines are allowed to absorb water for longer than the allotted time for the test, the measured PI increases significantly, initially at a rapid rate. Since economic houses are almost always founded on natural ground with minimal surface preparation, and since they are subject to wet-season rains which may last for days, rather than minutes, it seems likely that the PI given after minimum preparation on an un-dried “raw” sample is likely to be a better indication of the heave that will actually occur than that given by the standard laboratory procedure.

3.4.2 Linear shrinkage

The linear shrinkage test currently in use in South African laboratories uses material left over from Liquid Limit determination. The shrinkage trough is filled and immediately put in the oven at 105˚C-110˚C. This appears to be satisfactory for the granular materials common in road building. Clay soils, however, tend to arch and shatter when dried rapidly, making shrinkage measurement very inaccurate. The same soil left to air-dry usually arches and shatters to a far smaller degree; the linear shrinkage is often about double that of the oven-dried sample. Figure 3.1 shows eight linear shrinkage troughs all filled from one sample of raw, un-dried, un-sieved clay (LL 63, PI 40) which had given closure after 22 taps in the Casagrande cup. Four were put into the drying oven at 105˚C-110˚C, the remaining four were left to air dry for five days. The oven dried specimens are so shattered that the linear shrinkage values (4.39, 12.04, 10.17, 7.16) are meaningless. The air-dried values (15.14, 15.78, 15.05, 14.79) are all within 0.6 of the average value, (15.19) and give a consistent indication of high heave potential. Figure 3.1 shows the LS of a plastic clay when air dried (top) and oven dried (bottom).

Figure 3.1: Linear Shrinkage of a plastic clay, air – and oven dried.

Clays under houses experience changes in moisture content slowly over a long period. Oven drying gives a very poor indication of the shrinkage/swell likely to occur in practice. A far more realistic indication would be given by a sample consisting of the entire soil allowed to air-dry at normal temperature. Current test methods for linear shrinkage systematically underestimate the heave potential of expansive soils.

3.4.3 Clay fraction

Clay fraction is usually determined by hydrometer analysis. The unsatisfactory nature of the hydrometer has been recognized for some time. Savage (2007) noted that its unreliability could be due to the fact that clay particles may be plate-like and not spherical, as assumed in the analysis, and also that clay particles may be carried down rapidly by larger particles. He suggested that estimation of clay fraction by Skempton’s activity equation may be more reliable. The authors have found that there are often very significant differences between clay fraction estimated by this method and the hydrometer results given by commercial laboratories. Precipitation tests have been performed on a number of clay samples from economic housing sites. Soil was allowed to precipitate in an apparatus with dimensions similar to a hydrometer. Significant clay content has been found in all but the coarsest fraction precipitated. Estimates based on these observations suggest that the hydrometer test may significantly underestimate clay content in some soils, which in turn may lead to significant underestimation of heave potential.

3.5 General Considerations of the Standard Methods

The only one of the commonly performed simple tests with the potential to be a direct indication of shrink/swell potential is the linear shrinkage test. When performed on whole samples and allowed to air dry there is a possibility that it may give a direct indication of the behaviour of the in-situ material under a foundation. The Atterberg limits and clay fraction may often give a useful indirect indication of shrink/swell potential, but they do not give a direct value. Sridharan and Prakash (2000) have shown that Atterberg limits can give values suggesting very low heave potential for some expansive soils and indications of expansiveness in some soils which actually have low heave potential. If the laboratory techniques currently employed do not give reliable results for the heave potential of clays, then it is not surprising that failure of foundations for economic housing schemes remains at an unacceptably high level.

3.6 Conclusion

The design of foundations for small buildings on expansive soils is theoretically a very difficult problem, it is far from solved, and a theoretically-rigorous approach is impractical at this stage due to the cost and time required for determining the necessary parameters. Removal of a significant amount of expansive material and replacement by inert fill is also not feasible for most small structures, since the cost may be greater than the budget will allow. There seems to be little alternative but to use empirical methods of design based on parameters which are quick and economical to measure. These parameters should be directly relevant to expansiveness of soils and should be determined using procedures suitable for economic housing foundations rather than for road construction materials. Currently used tests do not adequately satisfy these criteria.

CHAPTER 4: DUPLICATE TESTING CONDUCTED ON THE INPUT PARAMETERS FOR THE ESTIMATION OF POTENTIAL EXPANSIVENESS OF CLAY.

4.1 Abstract

This paper presents a study conducted on three active clay soils analysed at seven commercial laboratories in South Africa. Commercial test results are often used as input parameters for prediction models used to estimate potential heave expected from active clays, especially when designing foundations for light structures. This paper briefly looks at the typical results obtained from such laboratories and comments on the correlation achieved.

4.2 Introduction

The American Society of Civil Engineers estimates that of all homes in the United States, a quarter has had some damage caused by expansive soils. The financial loss to property exceeds that of earthquakes, floods, hurricanes and tornadoes combined. In Africa there exists a widespread problem of providing economic housing for lower income communities. In South Africa the government is attempting to provide small subsidised houses for the very poor. Most of South Africa has semi-arid and sub-humid conditions (Weinert, 1980) which lead to generally shallow residual soils subject to seasonal saturation and aridity. Such conditions are known for giving expansive foundation problems.

Advanced mathematical solutions are available for heave prediction, but unfortunately they rely on the input of the soil water characteristic curve (SWCC) for which the typical cost is US $10 000 (Fredlund, 2009) - more than the cost of a low cost house. Besides the cost, the time needed to develop the SWCC takes in excess of 6 weeks. In practice almost all low-cost housing relies on empirical methods – estimation of heave potential based on quick, simple and economical tests. The most popular method in South Africa (Van der Merwe,1964), uses Atterberg limits and particle size distribution These tests are also known as

"Foundation Indicators". Predictions of heave are only as reliable as the input parameters. This paper focuses on the reliability of these input parameters in the form of test results received from seven premier commercial laboratories.

4.3 Particle Size Analysis of Soil Samples

Particle size analysis of soil samples is a combination of grading analysis using sieves with different size apertures and hydrometer analysis which uses settlement and Stoke's law to determine the finer fractions of a soil sample.

4.3.1 Grading analysis

Figure 4.1 shows results from a sample that was divided and sent to seven leading commercial laboratories. The results show an acceptable variance. Results are closely grouped from 13.2mm to 0.425mm. Sample preparation might have played a role in divergence of results from 0.425mm to 0.075mm. The authors have noted that details of the preparation of fines samples are critical and have a profound influence on the end result obtained.

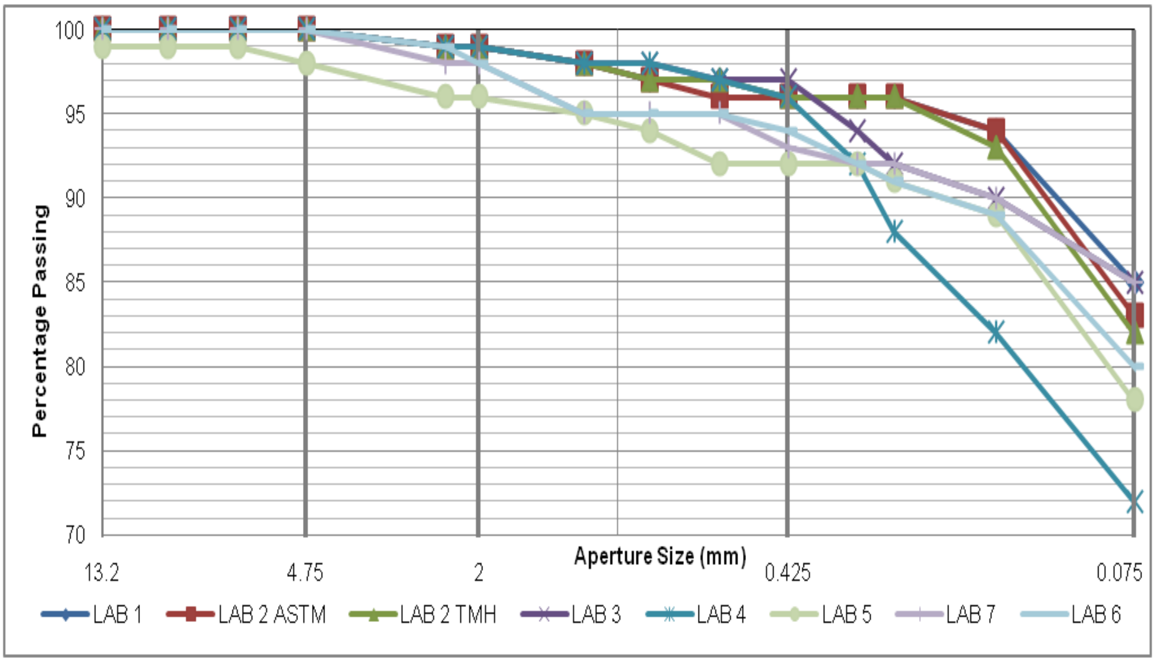


Figure 4.1: Grading analysis of Brandwag sample.

Six of the laboratories used in this study are accredited with SANAS and conform to the international ISO: 17025 standards, suggesting that the quality is well managed and ensured. For each of the three samples analysed, the grading analyses were reasonably consistent.

4.3.2 Hydrometer analysis

Hydrometer analysis is used to determine the various fractions of fines in a sample. The method is based on Stoke's law and is flawed due to several of its assumption being dubious. Clay particles may be flaky and may have very large specific surface areas (Whitlow, 2001). Hydrometer accuracy is doubtful for four reasons (Savage, 2007), Stoke's law assumes all particles are spherical, de-flocculation may not be complete at the time of testing, clay particles are partially carried down by larger particles and a relative density of 2.65 is assumed for all particles, which may not be true. This is addressed further in section 4.3.3.

Figures 4.2 (a) and Figures 4.2 (b) indicate the variance obtained from different commercial laboratories. *Note that the values vary between 63% and 18% on the Steelpoort sample (Figure 4.2 (b)), a difference of 45%.*

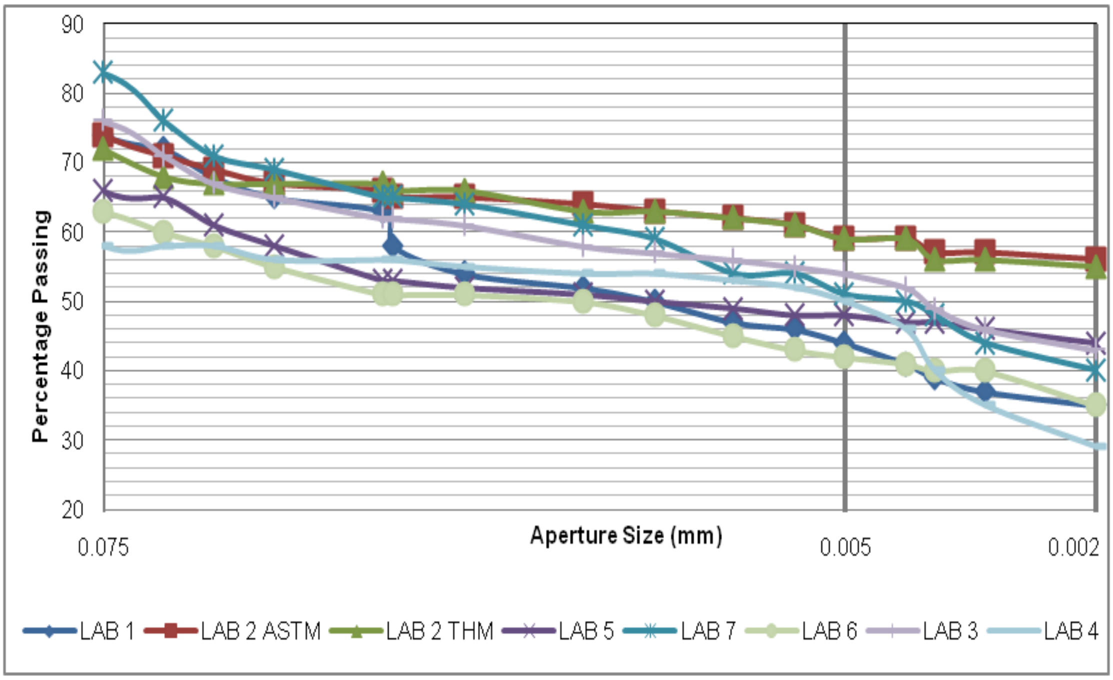


Figure 4.2 (a): Hydrometer analysis of Botshabelo Sample.

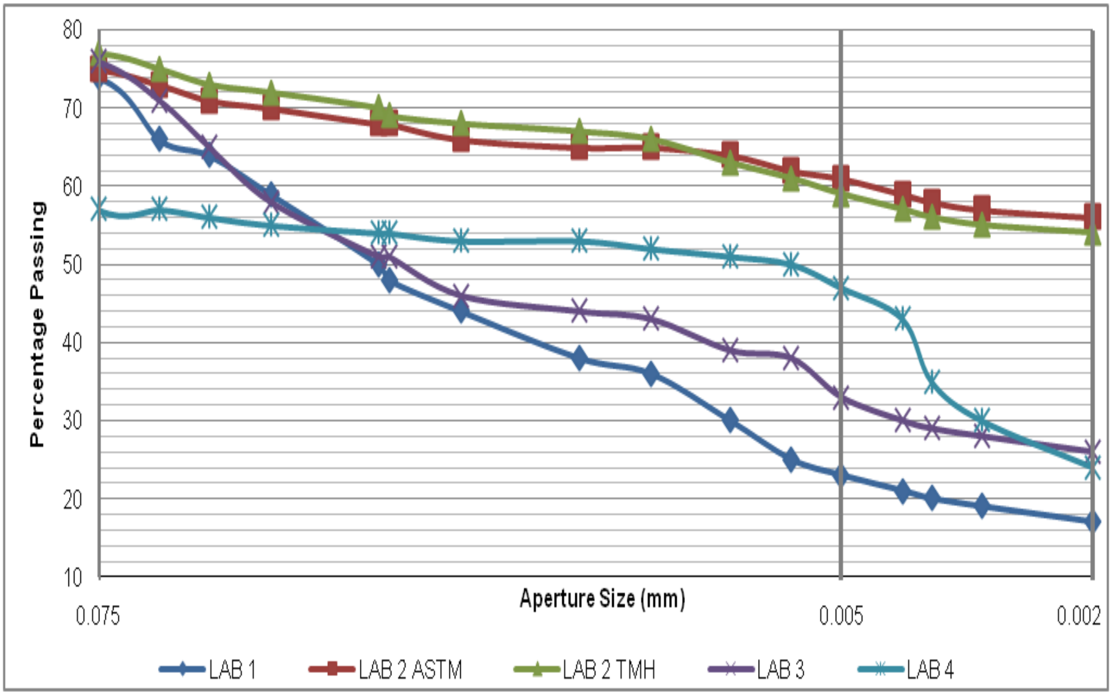


Figure 4.2 (b): Hydrometer analysis of Steelpoort Sample.

4.3.3 Savage's method to determine the 0.002mm fraction

Savage (2007) suggested using Skempton's activity equation to relate activity to the ratio (R) of the liquid limit (LL) to the plastic limit (PL) of a soil sample. Savage's analysis is as follows:

$$\text{Activity} = PI / P_{0.002} \qquad (4.1)$$

Where $P_{0.002}$ refers to the percentage of material smaller than 0.002mm

Savage found an exponential relationship between the ratio LL / PL (R) from a table of the activity values for Montmorillonite, Illite and Kaolinite published by Cornell University in 1951.

$$\text{Activity} = 0.16R^{2.13} \qquad (4.2)$$

Using equations (4.1) and (4.2) the clay content can be established empirically as:

$$P_{0.002} = PI / \text{Activity} = PI / 0.16R^{2.13} = 6.25\ PI.R^{-2.13} \qquad (4.3)$$

The $P_{0.002}$ value obtained is based on the PI that was tested, typically at $P_{0.425}$, and does not represent the whole sample. The equation is adjusted to reflect the entire sample (PI_{Gross}):

$$PI_{0.425} \times P_{0.425} = PI_{Gross} \qquad (4.4)$$

$$P_{0.002} = 6.25\, PI_{Gross}.R^{-2.13} \qquad (4.5)$$

Table 4.1: draws a comparison between the hydrometer values for clay fraction and those derived from Savage's equation (4.5). In Table 4.1 it can be seen that some of the results compare favourably, while those in *italics* are unacceptable. Savage's method does not consistently predict lower or higher values and the values do not correlate well.

Table 4.1: Hydrometer $P_{0.002}$ compared to Savage $P_{0.002}$.

Sample	Lab1	Lab2	Lab3	Lab4	Lab5	Lab6	Lab7
Steelpoort							
Hydrometer	*17*	56	*24*	*26*			
Savage	*68*	44	*11*	*49*			
Brandwag							
Hydrometer	49	58	*33*	44	56	50	47
Savage	51	*27*	*18*	44	40	43	36
Botshabelo							
Hydrometer	35	*56*	29	43	44	40	35
Savage	31	*26*	22	30	31	37	34

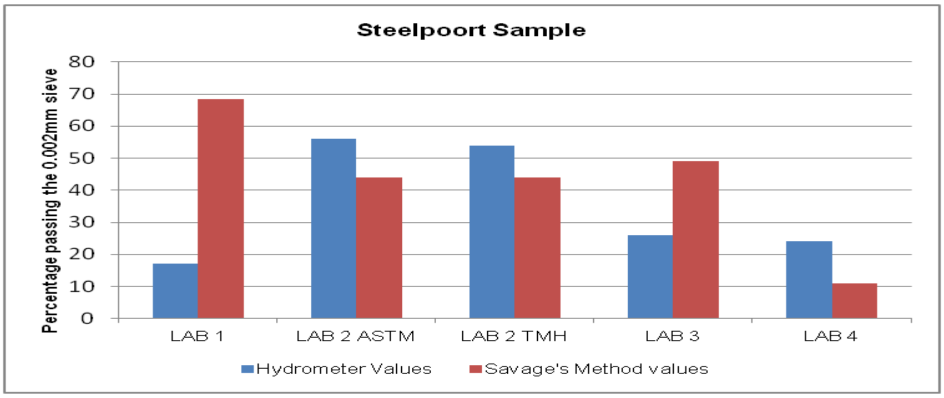


Figure 4.3: Steelpoort sample: Hydrometer $P_{0.002}$ compared to Savage $P_{0.002}$.

4.3.4 Atterberg limits

Atterberg limits include the Liquid Limit (LL), Plastic Limit (PL), Plasticity Index (PI = LL- PL), the Shrinkage Limit (SL) and Linear Shrinkage (LS).

Commercial laboratories in South Africa make use of the Casagrande – cup apparatus and typically perform the LL test according to TMH1: 1986 Method A2, which uses the same apparatus as the British Standard BS 1377-2. The authors have found that sample preparation has a significant bearing on the results, which may be responsible for the variance between the different commercial laboratories. Figure 4.4 illustrates typical results obtained for one of the three different active clay samples.

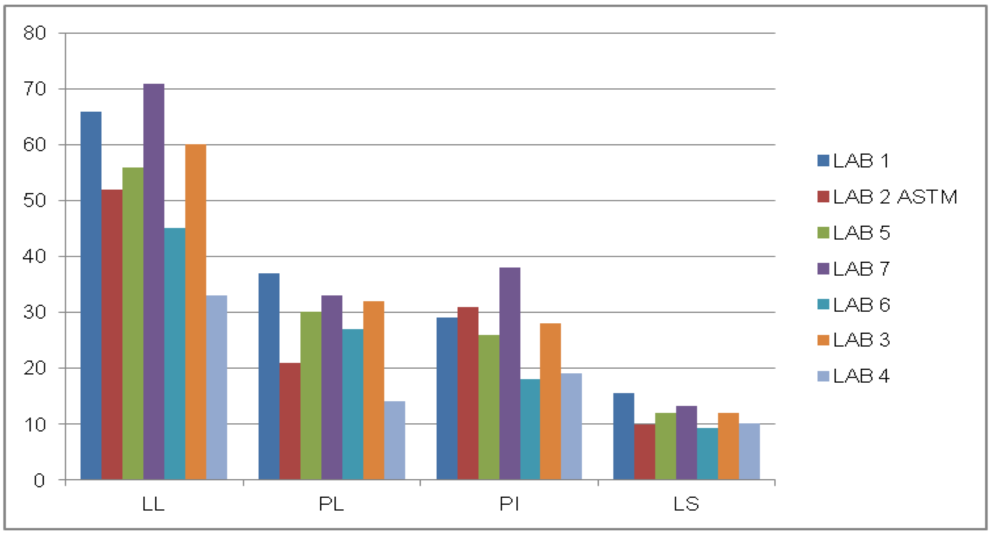


Figure 4.4: Brandwag sample results.

Based on Figure 4.4, it is clear that the LS values are relatively consistent, with most laboratories within two per-cent of the average. The LL, PL and PI values show a large variance, although there is a grouping of LAB2, LAB3, LAB5 and LAB6 which seem to compare well.

Casagrande's plasticity chart, as used by the Unified Soils Classification System can be derived from the relationship between the LL and PI of a soil sample.

Figure 4.5 shows the relationship between the liquid limit and plasticity index for the Steelpoort sample.

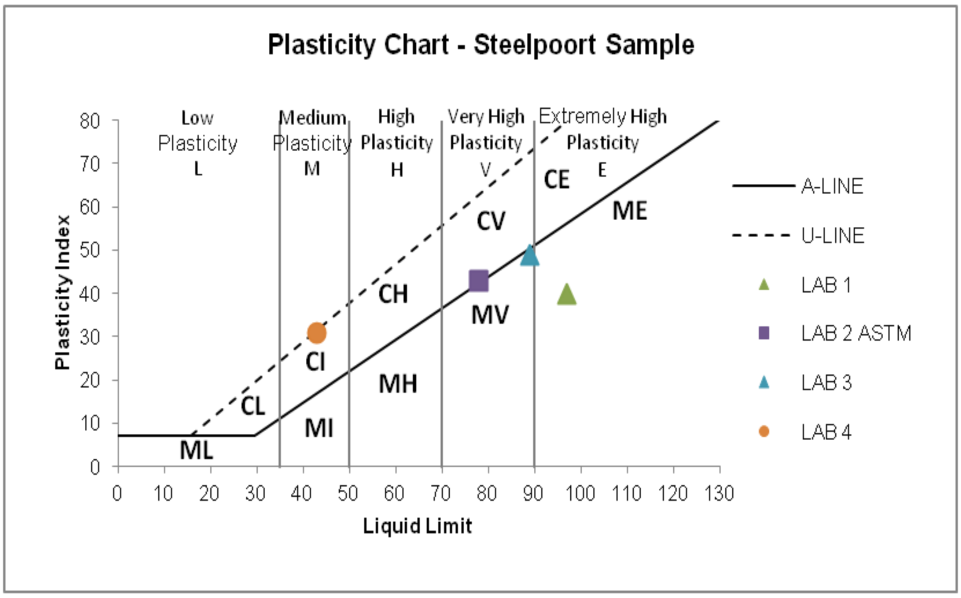


Figure 4.5: Casagrande's plasticity chart, Steelpoort sample.

4.4 Van der Merwe's Empirical Method for the Estimation of Potential Heave

Van der Merwe's method relies on the PI of the gross sample and the $P_{0.002}$ fines fraction, which according to the British Classification is deemed to be the clay fraction of a soil sample. For Van der Merwe's method to give valuable output, valuable input is required. This paper does not focus on the prediction model itself, but rather the input parameters. For better understanding of the method, a simplified example of the process follows:

PI_{Gross} is plotted on the y-axis and $P_{0.002}$ on the x-axis as the "classification of heave potential" curve, after (Van der Merwe, 1964). Figure 4.6 shows an example of such a plot with values obtained from various laboratories using Hydrometer analysis to obtain the $P_{0.002}$ fraction. Table 4.2 and Figure 4.6 compare the resultant heave potential to that obtained using Savage's equation to determine the $P_{0.002}$ fraction.

Skempton (1953) suggested using the relationship between the $P_{0.002}$ fraction and the Plasticity Index to give an indication of the heave potential of soils. He proposed using slopes of less than 0.75 to refer to inactive clays, slopes of more than 1.4 would suggest active clays and everything in between would be referred to as normal clays. In Figure 4.6 those designations have been adjusted to reflect slopes of 2.0, 1.0, 0.7, 0.6 and 0.5, with anything less than 0.5 considered inactive.

Table 4.2: Heave Potential (Hydrometer Analysis: Savage's Equation).

Method	Lab1	Lab2	Lab3	Lab4	Lab5	Lab6	Lab7
Hydrometer	Medium	Medium	Medium	High	Low	Very High	Low
Savage	Medium	High	Medium	Medium	Medium	Very High	Low

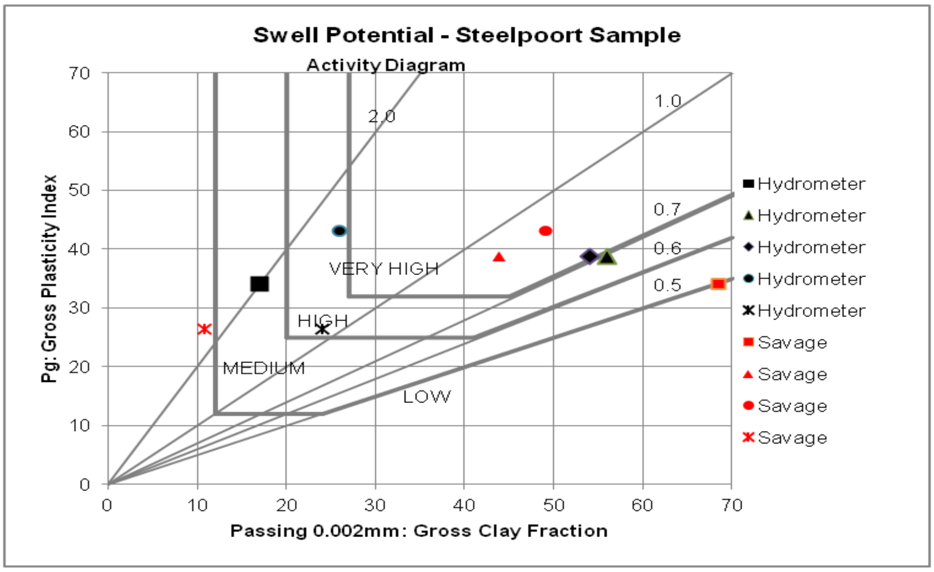


Figure 4.6: Swell potential: Steelpoort sample

4.5 Conclusion

Seven leading commercial laboratories were tasked with performing “Foundation Indicators”, which refer to the Atterberg Limits and Particle Size Analysis. Although the physical sieving provided comparable results down to the 0.425mm sieve, anything finer proved troublesome. The authors concluded that the problem probably lies with the preparation of the samples, as some details of preparation were found to have a major impact on testing done “in-house”.

The Atterberg Limits were performed on fractions passing the 0.425mm sieve, and preparation might have played a role there also. A grouping of laboratories obtained results that compared well, while two laboratories found substantially different results. The finer fractions, those passing the 0.075mm sieve, proved problematic as not all of the laboratories use the same method, although theoretically they should yield similar results. The results varied substantially and the range between the highest and lowest $P_{0.002}$ is alarming. It can be concluded that using “foundation indicators” alone as the basis for empirical heave prediction methods is a very risky approach and that other approaches need to be identified.

CHAPTER 5: ASSESSMENT OF CLAYS BY SMALL-SCALE SUCTION TESTS

5.1 Abstract

The relationship between suction and water content gives crucial information about a soil. The time and cost of determining the full soil water suction curve make it not feasible for small projects like economic housing. A considerable range of soil suctions can be determined easily and within a reasonably short time by using small samples, simple suction control and a high precision balance. It appears that in this way it may be possible to estimate heave potential and variability of soil properties at reasonable cost in an acceptable time.

5.2 Introduction

The tests here presented aim to address problems with the current commercial methods for foundation indicator tests. Usually such tests involve procedures primarily intended for road construction materials and they frequently give poor estimates of heave potential for high clay-content soils. Preoccupation with economy points to a need for simple tests with minimum skilled labour content, minimum opportunity for short-cuts and straightforward interpretation of results. Tests should directly relate to heave potential, rather than general properties. Suction potential gives an indication of how readily a soil can draw in water. Change in water content is the cause of volume change. Suction potential should be a good indicator of swell potential.

5.3 Indication of Shrink/Swell Potential

Kassa (2005) found that Atterberg limits are not a reliable indicator of volume change under load. Sridharan and Prakash (2000) found that Atterberg limits can sometimes indicate heave potential significantly higher or significantly lower than reality. Atterberg limits remain, however, the most popular indicators of heave potential. The Linear Shrinkage test performed on raw soil samples and allowed to air-dry slowly, gives a graphic indication of the extent to which a soil may change volume with water content, but does not indicate the likely pressures involved in this change of volume.

Clays expand when they draw in water. The force which can be exerted in this expansion depends on the strength with which they can suck in water. The suction of clays has long been recognized as a cardinal indicator of heave potential, but it is one of the least convenient indicators to measure. The plot of suction from saturation to desiccation is known as the soil water characteristic curve (SWCC). The SWCC is an indispensable part of a full unsaturated analysis, but it requires much time and skill to measure.

5.4 Recent Advances and Current Practice in Suction Measurement

Three papers presented at the International conference on Soil Mechanics and Geotechnical Engineering in Paris 2013 dealt with advances in techniques of suction measurement. Two other papers dealt with measuring suction in practical engineering projects. Advances in dew-point potentiometer technology point to extending the range of suctions measurable by this type of instrument (Macek *et al.*, 2013). The use of micro-porous membranes may allow quicker suction measurement in the 0-30 kPa suction range (Nishimura, 2013). The use of a centrifuge may speed up measurement in the 0 to 900 kPa range (Reis *et al.*, 2013). But in practical use, for assessing lime treatment on London Clay (Mavroulidou *et al.*, 2013) and for modeling the impact of climate changes on embankments and cuttings (Mendes and Toll, 2013), Whatman No.42 paper remained the method of choice. This time-honoured system takes two to six weeks for a suction measurement and requires careful laboratory technique.

5.5 Small Scale Suction Potential Measurement

A more limited indication of suction potential can be found by allowing samples to reach equilibrium at known temperature and humidity. Soil samples can be brought to equilibrium with saturated solutions of various salts (Blight, 2013). Blight's tests took typically 90 days. Although the equipment was cheap and unsophisticated, and the skilled labour component not large, the time frame is not feasible for normal engineering practice. The authors have modified the procedure to give quicker results, making enquiry into some important questions feasible.

The procedure involves using small pieces broken from a soil specimen. Breakage usually occurs along planes of existing weakness; the micro-structure and fabric of the soil are not greatly disturbed. Samples are placed in small glass weighing bottles with ground-in lids. When firmly closed, little air or water vapour leaves or enters in the time taken for weighing. Samples are weighed on an analytical balance and then placed with lids open in a container at controlled temperature and humidity.

Equilibrium moisture content is a measure of the suction potential of the soil under those conditions. Samples can be weighed periodically by closing the ground-glass lid and removing from the controlled atmosphere to the analytical balance. For most of the tests performed in this investigation suctions corresponding to saturated solutions of KCl and NaCl were used, though a number of tests used both higher and lower suctions.

Figure 5.1 shows curves of moisture content against time at constant suction for samples of 30 clayey soils from housing developments in central South Africa. The masses of the samples range between 25g and 35g. Starting water contents were arbitrary. The equilibrium moisture contents at 22 MPa suction (corresponding to a saturated solution of KCl at 20˚C) vary from 3.6% to 12.6%.

There appears to be a correspondence between moisture content and possible heave potential. Those with the highest water contents have Plasticity Index close to 40, which would normally be considered an indication of high heave potential. Those with the lowest water content have PI below 15, normally taken as an indication of low heave potential.

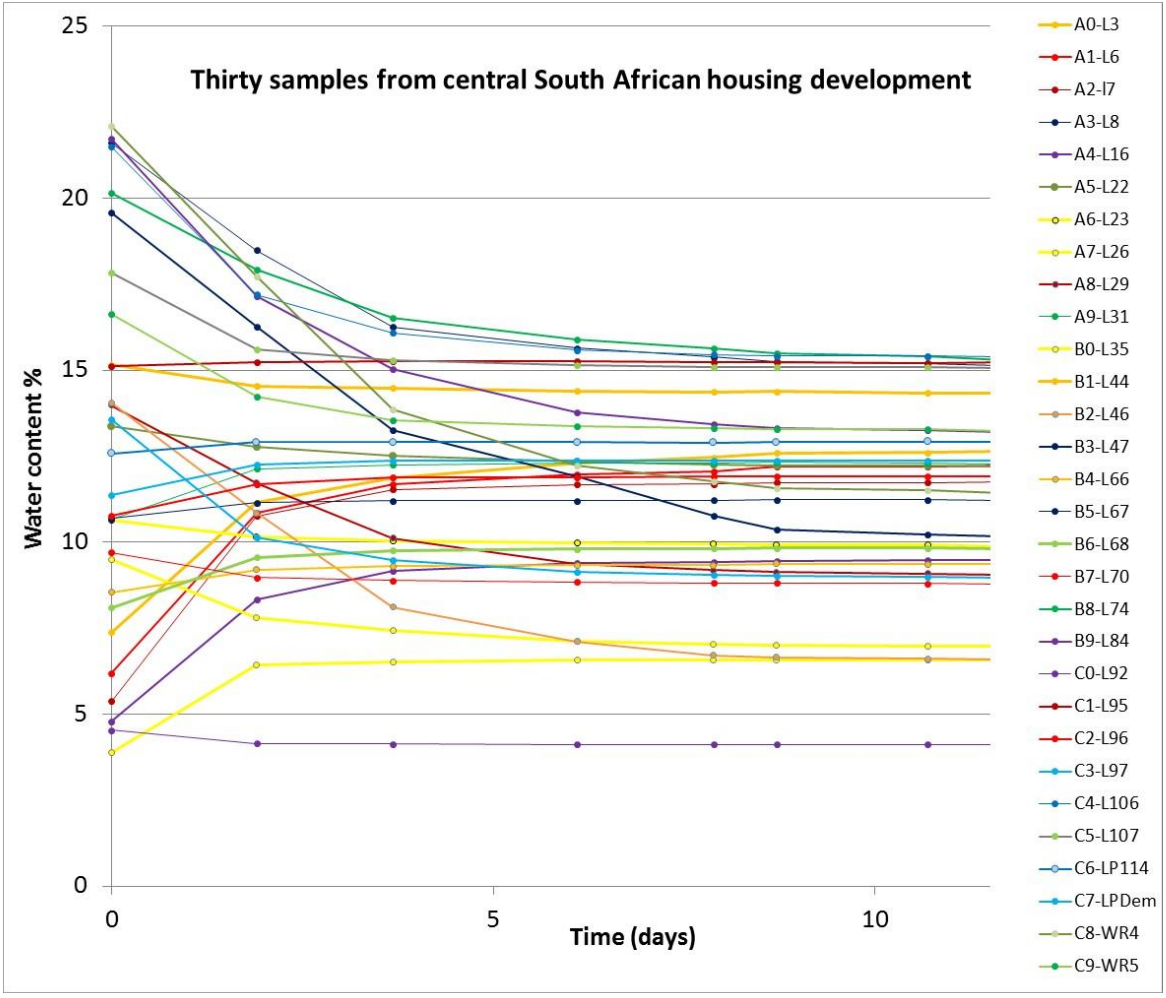


Figure 5.1: Thirty samples at 22 MPa suction.

These same samples were used to probe questions concerning differences of behaviour in wetting and drying, and the possibility of permanent change in properties due to oven drying.

Figures 5.2 to 5.4 show these 30 samples being tracked from equilibrium moisture content at 22 MPa, through oven drying and re-wetting by absorption of water from the air (Figure 5.2), wetting by the addition of a small amount of de-ionised water and re-equilibrating at 22 MPa (Figure 5.3), saturation with de-ionised water and re-equilibrating at 22 MPa (Figure 5.4).

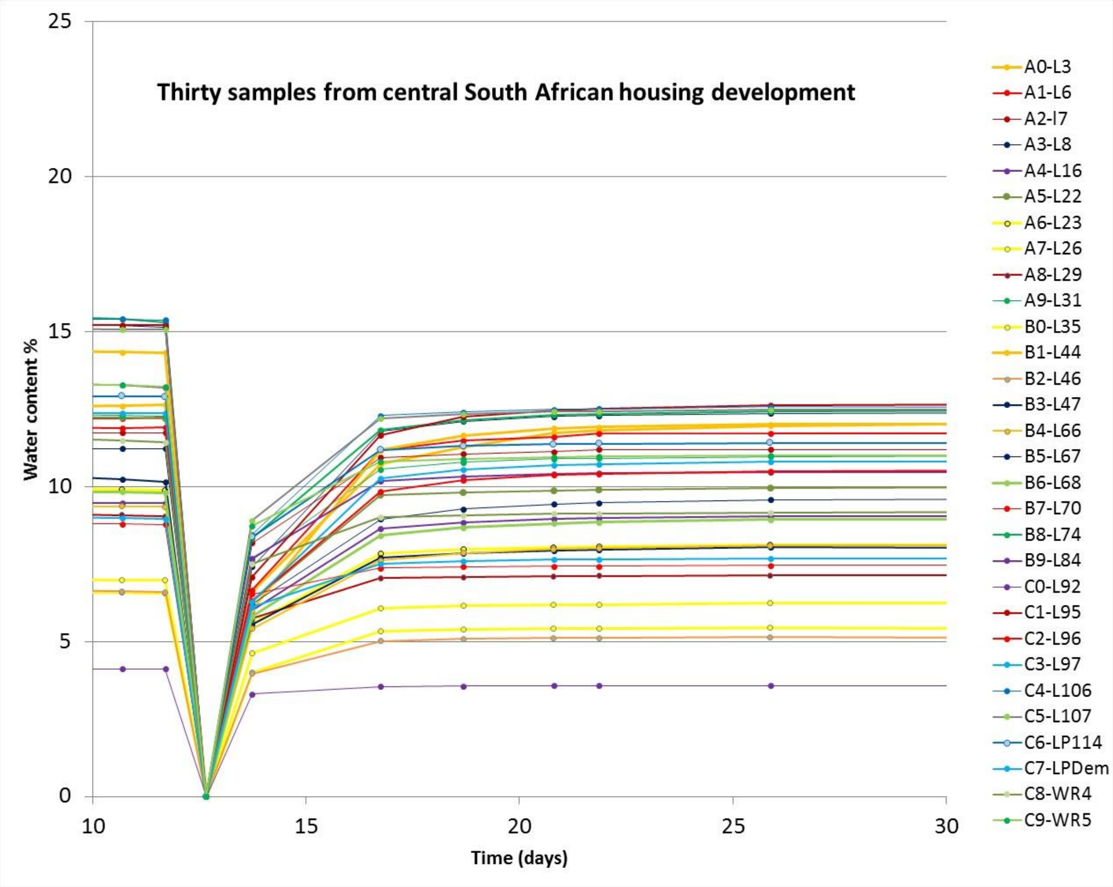


Figure 5.2: Thirty samples after oven drying.

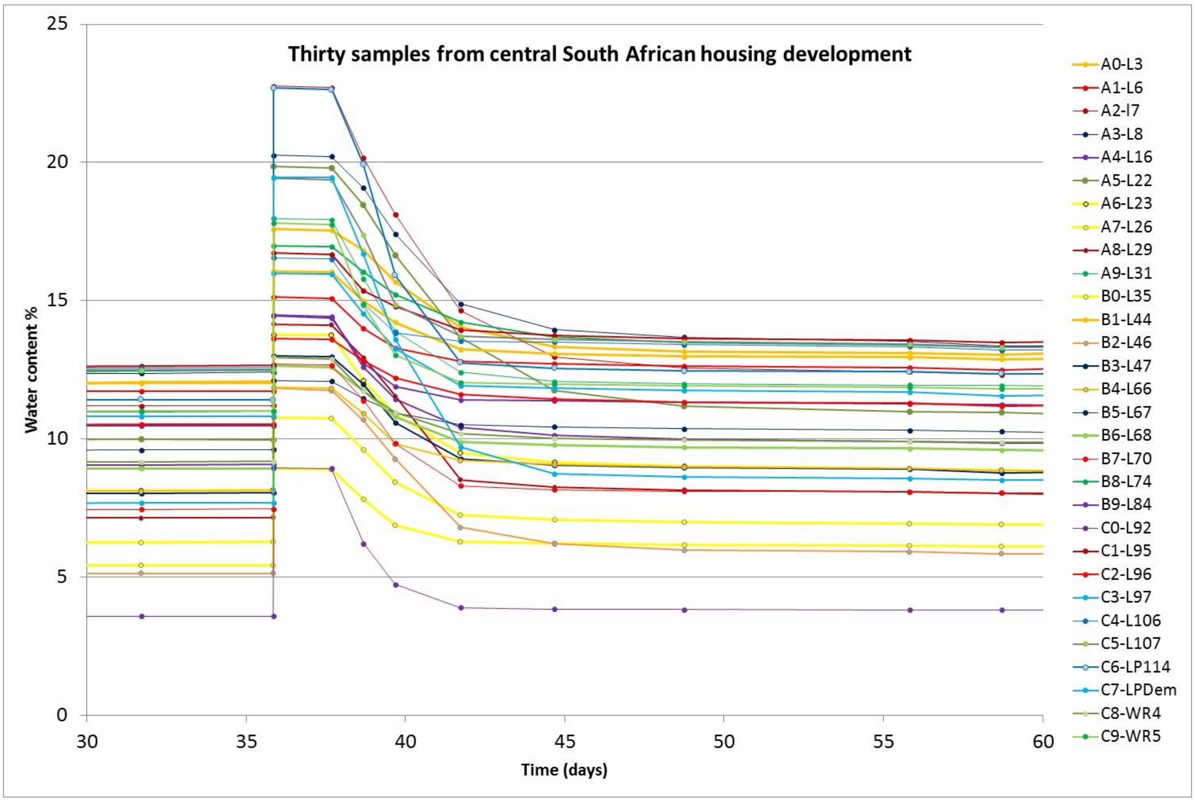


Figure 5.3: Thirty samples after wetting.

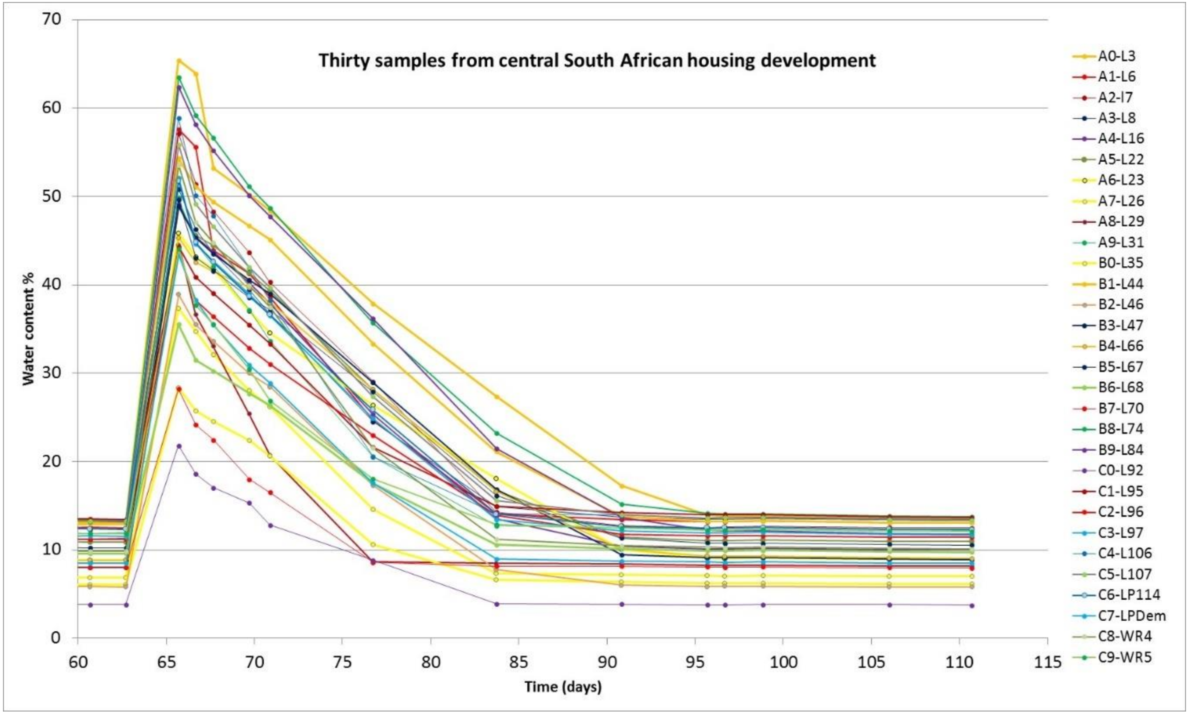


Figure 5.4: Thirty samples after saturation.

Figure 5.5 shows the water content reached by 30 samples at 22 MPa from initial moisture content after oven drying, wetting and saturation as a bar chart. Samples whose initial water content was very low are grouped predominantly on the left side. Where the initial water content of the soil was well above its 22 MPa value, oven drying significantly reduced the soil's subsequent suction potential, and even saturation did not restore the original suction potential. Where the initial water content was well below the 22 MPa value, the soil was not so greatly affected by oven drying and saturation could lead to higher retained moisture content at 22MPa. This suggests that Blight's (2012) contention that air drying can lead to permanent changes in some residual soils may commonly apply to many clayey soils.

Alternatively, it may be a consequence of the well-known hysteresis property of clays and a very high degree of saturation may be needed to regain the original drying curve. This test also suggests that in the majority of cases, whatever suction changes take place, the ratios of water content between the various samples remain substantially the same. The possibility of consistent relationship over a range of suctions were tested with 20 samples which were brought to equilibrium at 37 MPa then 10 MPa and then 2 MPa.

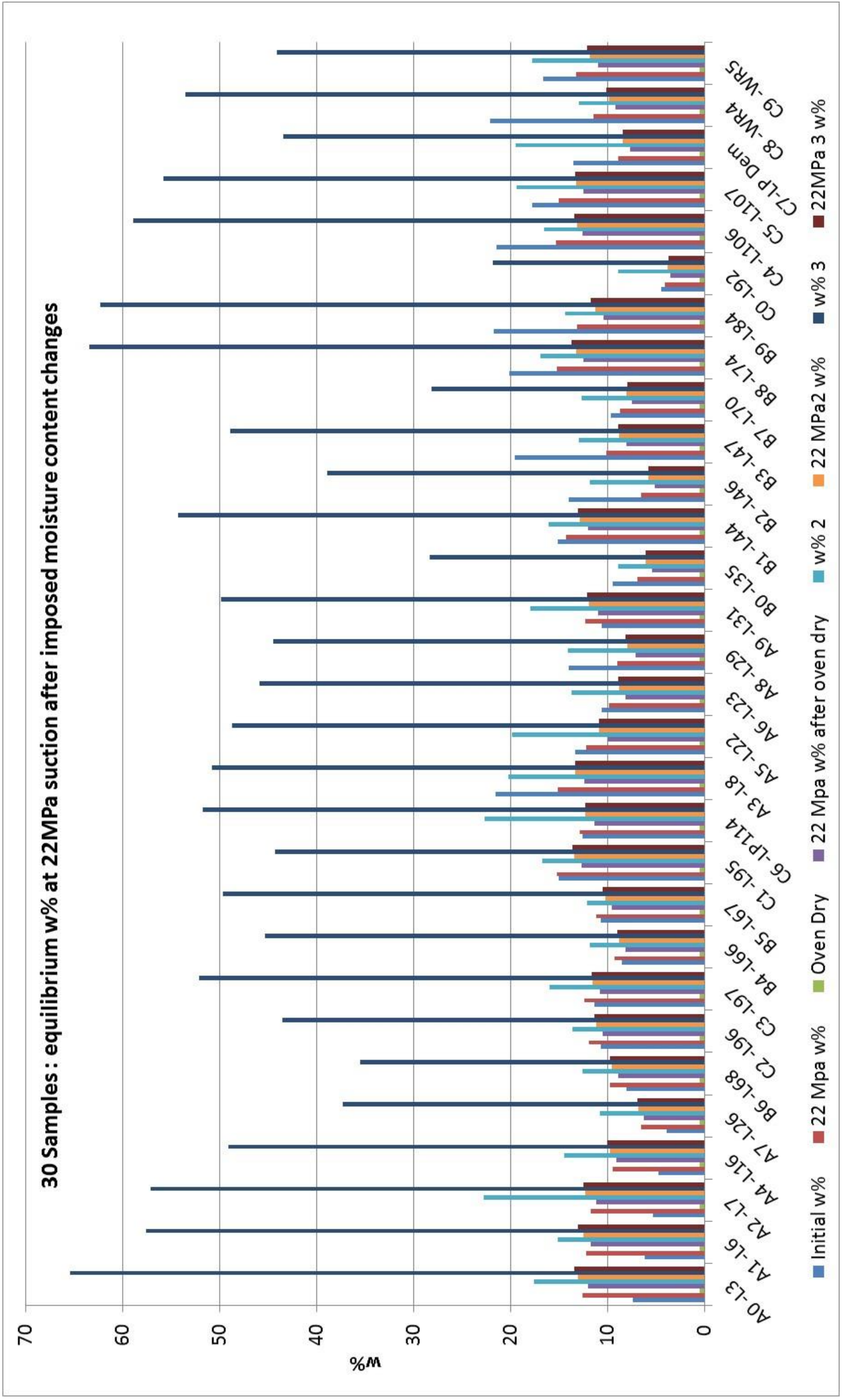


Figure 5.5: Water content reached by 30 samples at 22 MPa.

Figure 5.6 shows that almost all of the samples maintained a consistent relationship throughout this range of suctions. They also maintain this relationship after oven drying. This suggests that a single determination at one suction may allow estimation of suction potential over a considerable range of suctions. Figure 5.6 indicates equilibration of water content at 37 MPa, 10 MPa, 2 MPa and at 22 MPa after oven drying for the 20 samples.

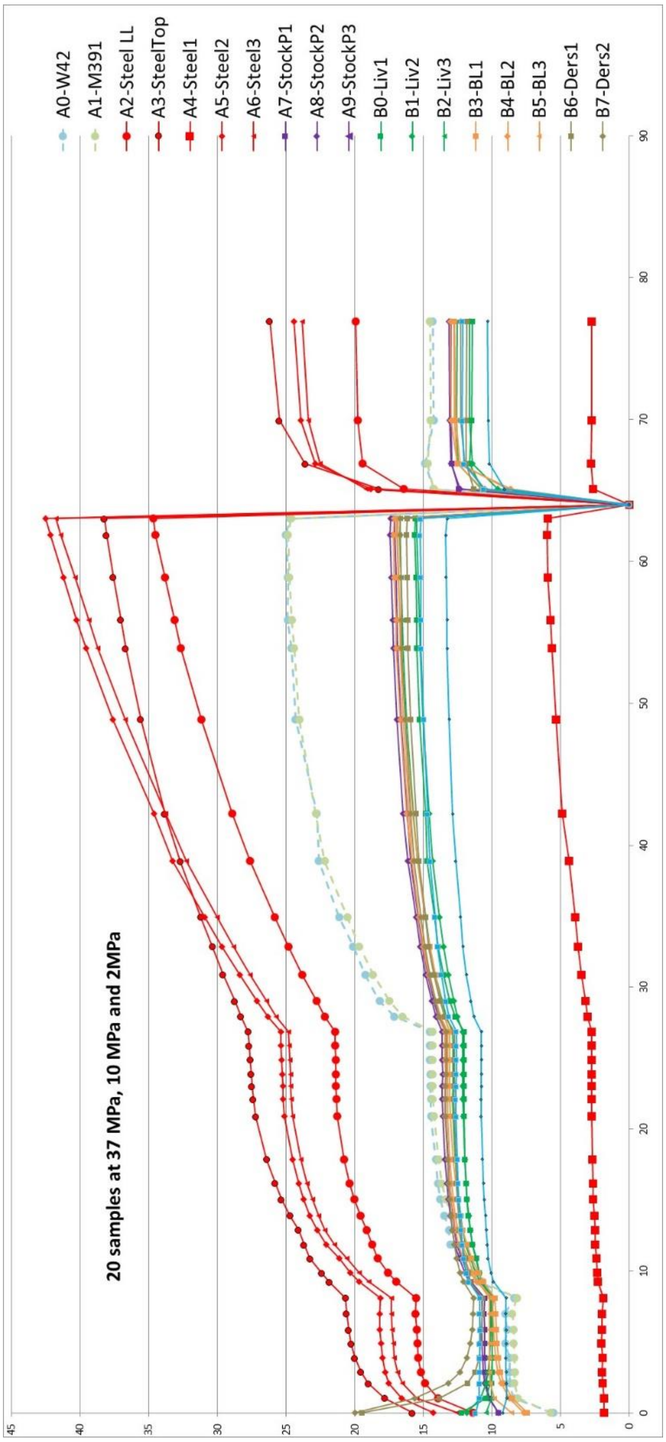


Figure 5.6: 20 Samples: Equilibrium of water content.

5.5.1 Reducing the time of testing

It could be expected that smaller samples would take less time to reach equilibrium. Figure 5.7 shows plots of moisture content against time for seven different-sized small samples of the same clay (PI = 40). The sample masses vary from 0.71g to 8.4g. All seven samples reached equilibrium water content in approximately one day. The equilibrium value reached by the 7 samples averaged 13.56% with standard deviation of 0.30%. The correlation coefficient with sample-size was 0.17, suggesting that there is no significant correlation between sample size and test outcome for this range of sample masses (8g to slightly less than 1g).

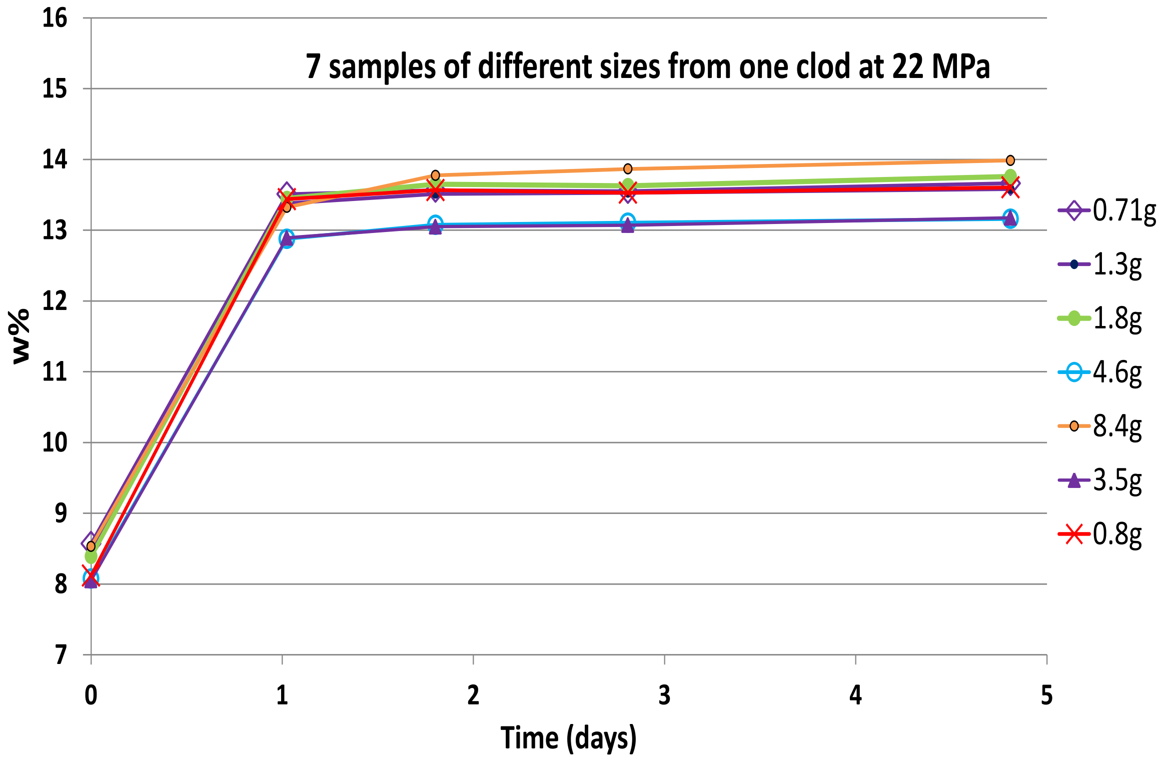


Figure 5.7: Equilibrium of 7 samples at 22 MPa suction.

5.5.2 Assessment of variability of soil properties

Soil is highly variable, and testing procedures usually lay stress on the need to take a sample of adequate size from which a representative portion is obtained by systematic mixing and division. The intent is to give characteristic median values. Reliability analysis is an increasingly accepted technique for accounting for uncertainties in various aspects of geotechnical analysis (Phoon, 2008). Reliability analysis needs an estimate of standard deviation as well as a median value of soil properties. The procedure described above allows a direct assessment of the variability of a soil by using a number of different small samples. The difference in time involved in testing one, five, or even twenty samples is small, since the samples are just weighed and put into a container where they are left to reach equilibrium.

Preparation time is minimal - specimens are simply selected from any preferred location in the sample. There is little scope for short-cuts in preparation. The soil micro-structure remains substantially intact. For samples smaller than about 10g, a time of only one or two days may be needed to reach equilibrium when suction is 20 MPa or more. The weighing procedure is quick compared to the time taken to reach equilibrium - each weighing takes only a few seconds.

Table 5.1: Mean and standard deviation for 5 samples of 6 different soils

	Average w% at 22 MPa	Standard Deviation
S1 Light brown clayey sand	7.12	0.23
S2 Dark brown clay	11.92	0.29
S3 Black clay	12.4	0.18
S4 Brown clay	13.86	0.32
S5 Light yellow silty clay	6.06	1.22
S6 Light brown sandy clay	11.09	1.76

Table 5.1 shows results for five specimens each of 6 widely different soils. The first four show only small standard deviation, suggesting that variability is small and a single test result would probably give a good representation of the properties of the soil. The results for the fifth and sixth samples show substantial standard deviation. Sample S5 is a kaolinitic clay presenting little problem for heave. Sample 6 was taken from a subsidy housing project site where results of foundation indicator tests suggested low heave potential but significant heave was in fact, encountered. Ten more samples of the soil S6 were tested. The results for the two tests were:

7.98, 12.22, 11.67, 12.02, 11.58 Average 11.09, S. Dev. 1.76, 11.63, 11.72, 11.37, 11.23, 11.98, 8.11, 11.50, 11.76, 11.13, 11.37 Average 11.13, S. Dev. 1.11.
Most of these results give equilibrium water content at 22 MPa close to 12%, indicating the probability of medium to high heave potential. There are, however, two very different results, one in each set. Both are close to 8%, indicating far lower heave potential.

These results suggest that in a very restricted area there is the possibility of major variation in properties for some soils. They suggest the possibility that a single laboratory determination of heave potential may give a misleading value and this could be the cause of the damage which occurred in this case.

5.6 Conclusions

The testing technique described here, using very small samples subjected to easily-controlled suctions, may provide a more convenient means of assessing probable heave potential than current commercial methods. Very little sample preparation is involved and there is little scope for short-cuts. A large enough number of test samples can be used to gain insight into the variability of the soil concerned without greatly increasing the testing time or labour cost.

CHAPTER 6: VARIABILITY IN SOIL PROPERTIES AND ITS CONSEQUENCES FOR DESIGN

6.1 Abstract

Samples from one particular layer of one particular test pit may give widely different results at different laboratories. Jacobsz and Day (2008) suggested possible slipshod testing. It appears that testing is sometimes not done as rigorously as it should be, but this does not convincingly explain all cases. Badenhorst *et al.* (2015) suggested that preparation differences may be significant. Preparation can lead to different results (Blight, 2012; Stott and Theron, 2015a), but will also not convincingly explain some cases. Many soil properties are linked to suction. A procedure for assessing suction potential with little sample disturbance was described by Stott and Theron (2015b). This technique allows reasonably rapid testing with little variation in preparation or testing procedure. Tests performed on a range of soils show large differences in variability. It seems possible that inconsistent laboratory results indicate that some soils cannot be reliably assessed with current procedures.

6.2 Introduction

The variability of soil properties has been noted by many observers, (e.g. Singh and Lee, 1970; Phoon and Kulhawy, 1999). There appears to have been relatively little heed paid to this observation by many practicing engineers. Little attention is usually drawn to it in tertiary level geotechnical engineering courses. Little attention is drawn to it in many well-known soil mechanics text books. “Craig’s Soil Mechanics” (2012) mentions variability in three places, Das (2006) makes no mention of variability in either “Principles of Geotechnical Engineering” or “Advanced Soil Mechanics”. It is mentioned once in “Soil Mechanics for Unsaturated Soils” (Fredlund and Rehardjo, 1993) and once in “Unsaturated Soil Mechanics in Engineering Practice” (Fredlund *et al.*, 2012). Blight’s “Unsaturated Soil Mechanics in Geotechnical Practice” (Blight, 2012) is one of few text books which not only stress the existence of variability but also point out the dangers of ignoring it.

Perhaps, therefore, it is not surprising that it is common practice, at least in the case of normal "bread and butter" engineering projects, to send one sample from each distinct horizon from a test pit to one laboratory for the performance of one set of a very limited selection of standard tests. There have been clear indications that this method of working may be inadequate. Jacobsz (2013) described a situation resulting from this procedure at an electricity substation. Samples from a test pit were sent to a reputable laboratory where the usual Technical Methods for Highways (TMH) (CSIR, 1986) foundation indicator tests were performed. Results indicated that the soil had low expansive potential and no major precautions were taken against heave in the foundation design. Significant heave damage did, however, occur. Stott and Theron (2016) noted a case where samples from a housing development were analysed by the "foundation indicator" tests from TMH1. These tests indicated no risk of heave and the foundations and superstructures were designed accordingly. Heave did, in fact, occur, and one house became structurally unsound and had to be demolished before its construction was even completed.

It is widely suspected that a prime cause of this situation is that engineering materials laboratories may be slipshod in their testing procedures e.g. Jacobsz and Day (2008). It has also been suggested that the tests may be critically dependent on details of sample preparation, which vary between laboratories (Badenhorst *et al.* 2015; Stott and Theron, 2015a). The warning of intrinsic variability noted by, for example, Phoon (2008), however suggests that it may rather be the normal practice of reliance on a single set of tests from each horizon which could be unsound. It is almost certain that the main reason for relying on only one test is the expense of multiple testing. This paper outlines an investigation to assess the intrinsic variability of soils which requires relatively little time and input of skilled labour and little increase in these inputs for obtaining a significant number of results. It may therefore have the potential for indicating intrinsic variability in an economically feasible way.

6.3 Testing of Soils for Typical Southern African Projects

The majority of soils tests in Southern Africa deal with sites where moisture content experiences marked seasonal variation and unsaturated conditions are normal. Light structures like roads and low-rise buildings provide a significant fraction of the samples tested. It has long been realized that soil suction is the defining feature of unsaturated soils and unsaturated soil mechanics experts e.g. Fredlund *et al.* (2012) affirm that the correct way to precede with any unsaturated soils problem requires determining the Soil Suction Curve – also known as the Soil Water Retention Curve or the Soil Water Characteristic Curve. All soils property functions required for non-saturated soils analyses can then be derived by means of this curve. Using these soil property functions, differential equations can be set up, boundary conditions can be defined and fully automated solutions follow. Unfortunately, the cost of producing the suction curve is quite high and the time required is considerable. Such a costly, time-consuming procedure is not feasible for most small Southern African engineering projects. Engineers continue to rely on simple, inexpensive tests and analyses which have been in existence for decades, but whose relevance and reliability may be questionable.

The Central University of Technology's Soil Mechanics Research Group has explored simple and potentially rapid suction tests as described by Stott and Theron (2015b). These tests use well known principles (Blight, 2013), maintaining samples at known temperature and humidity and using small sample size and high precision weighing to achieve significant reduction in time to reach moisture content equilibrium. One of the initial aims was to investigate the possibility of using a single suction value to assess a soil's expansive potential. This may seem a very unlikely possibility, but as can be seen in Figure 6.1, the relative values of water retention between various soils shows reasonable consistency over a considerable range of suctions. It can be seen that equilibrium is reached reasonably quickly for high suction/low water content conditions and much more slowly for low suction/high water content conditions. For most of this test, temperature was maintained at 20 degrees ± 0.1 degrees Celsius. For part of the test the samples were allowed to follow laboratory ambient temperature which varied between 19 and 25 degrees. This demonstrates the feasibility of performing such tests in very economical circumstances.

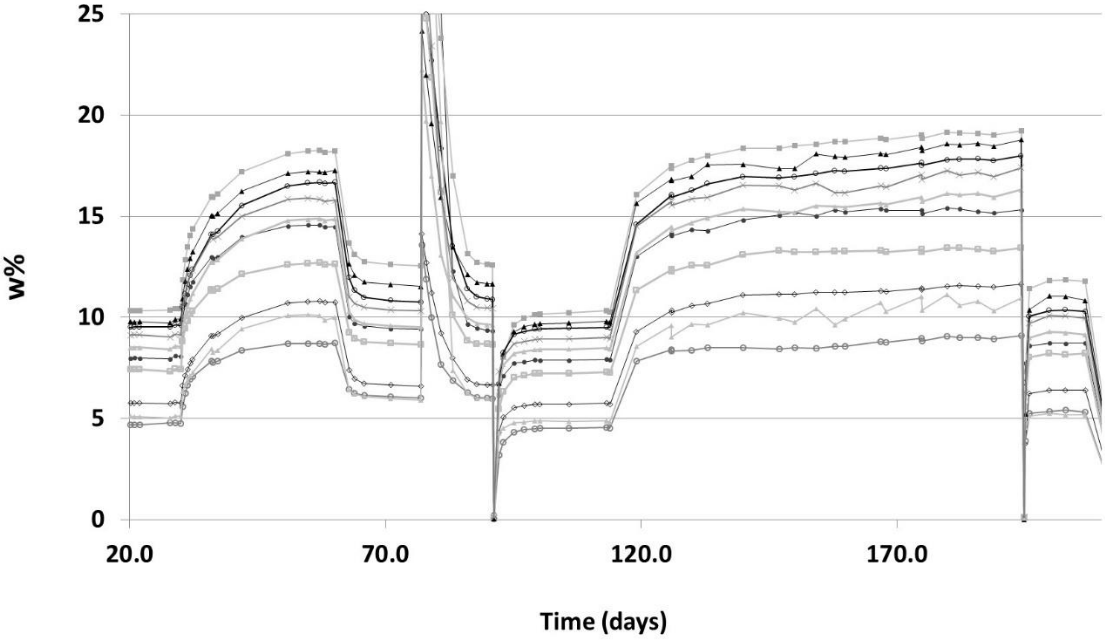


Figure 6.1: W%: time for 10 clays at various suction potentials.

Figure 6.1 shows w% versus time for 10 clays: over NaCl, water and KCl at 20˚C and over water at 19˚C to 25˚C with wetting and drying incidents. Time to reach equilibrium over water (low suction, high water content) is more than 30 days. Time to reach equilibrium over NaCl (38 MPa) and KCl (22 MPa) is about 5 days.

Not all soils follow this pattern. Figure 6.2 show curves for five specimens each of four different clayey soils, three of which follow the pattern of consistent relationship of water retention with suction, but the fourth breaks away from the pattern at low suction values and their curves cross over the curves of the other three. This soil is not typical; it is an almost pure Kaolinite from the Southern Cape. At high suction values the pattern of water content with suction variation is similar to the other clays, but is different for low suction values. The range of suction potential is also much larger than for the other soils at all suction values. This break in the normal pattern has been noted in relatively few soils, and usually only for “pure” clays, which are not a common occurrence. An example can be seen in Figure 6.3, which shows curves of multiple samples for each of four different soils.

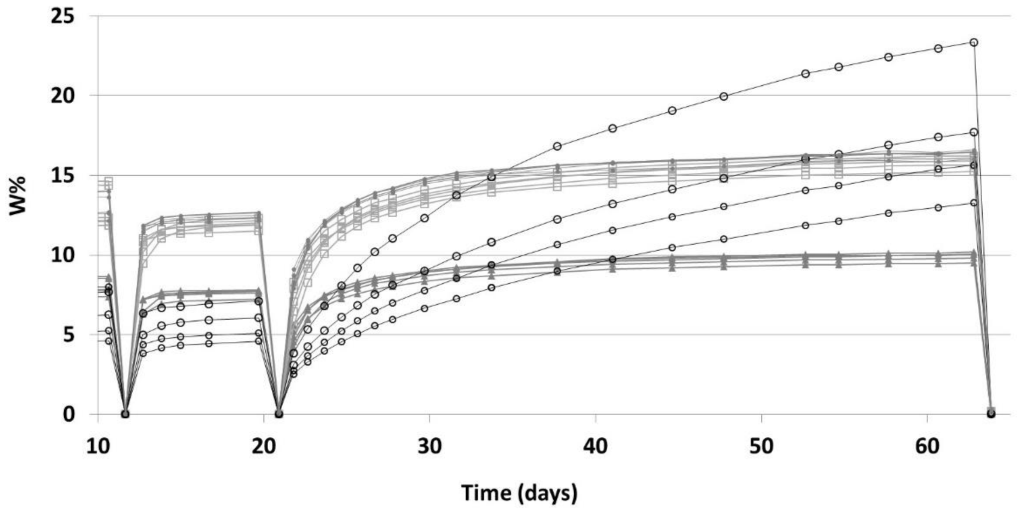


Figure 6.2: Five samples each of four soils over KCl and over water at 20˚C.

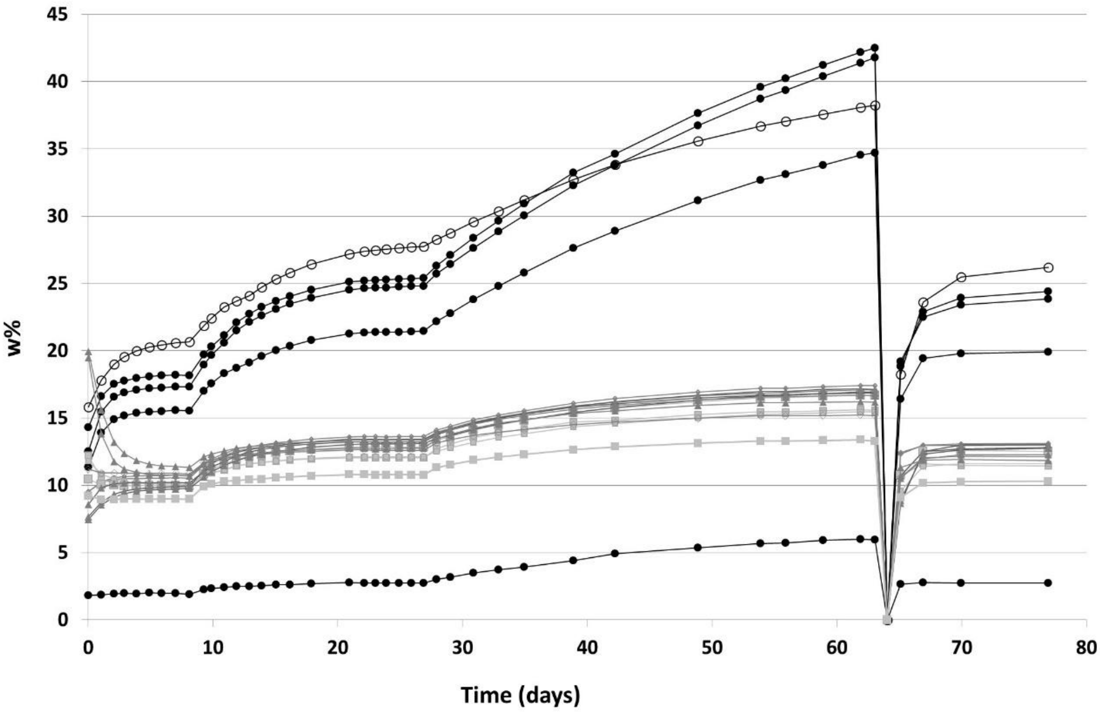


Figure 6.3: Twenty samples (4 different clayey soils).

Figure 6.3 indicates 20 samples (4 different clayey soils) at 37 MPa, 10 MPa, 1 MPa, oven-dried and 22 MPa. The five curves in Figure 6.3 with circular markers are for a clayey soil from Steelpoort in Limpopo. One of the curves (markers unfilled) does not maintain the usual pattern of constant relationship with the other samples. The sample showing the non-typical curve is not a natural clay sample, but was isolated from sediment in a settlement test of Steelpoort clay. It is pure clay (probably montmorillonite) from the upper layer of sediment following wet preparation. All of the other curves are for unprepared natural samples and all follow the normal pattern of maintaining a substantially consistent relationship.

6.4 Assessment of Variability

A noticeable feature of Figure 6.3 is the variation indicated by the curves for the different clayey soils. Most show little variation, but the Steelpoort clay (circular markers) shows very large variation, from the highest suction by far to the lowest suction by far. This observation of marked difference in variability led to a series of tests using multiple samples of several clayey soils and an assessment of the variability exhibited by these clays. Sample size was approximately 2-5 g. All samples were simply selected from appropriately sized pieces in the sample bags, or broken from larger lumps. They were then placed in glass weighing bottles with ground in lids after no further treatment. The samples were maintained at constant suction, either in a climate chamber or over solutions of various salts in a temperature-controlled chamber, until constant moisture content was achieved (usually from three to six days).

Figure 6.4 shows the results for 28 samples of Steelpoort clay at 28 MPa suction. Samples were supplied by Prof SW Jacobsz on two separate occasions. Samples were taken randomly from both batches. Both batches gave similar results, which are combined in this graph. Figure 6.4 tends to confirm the impression given by Figure 6.3 that there is huge variability in suction potential between individual samples of this soil. The lowest of the values for water-retention (2.7%) suggests a non-plastic soil with a PI possibly less than10, the highest retention (24.4%) suggests extremely plastic clay with a PI possibly greater than 50. The average retention of 17.3% suggests highly plastic clay with PI probably in the region of 35 to 40. The Coefficient of Variation (COV) is 35.6. Samples of this clay were sent to seven reputable soils testing laboratories.

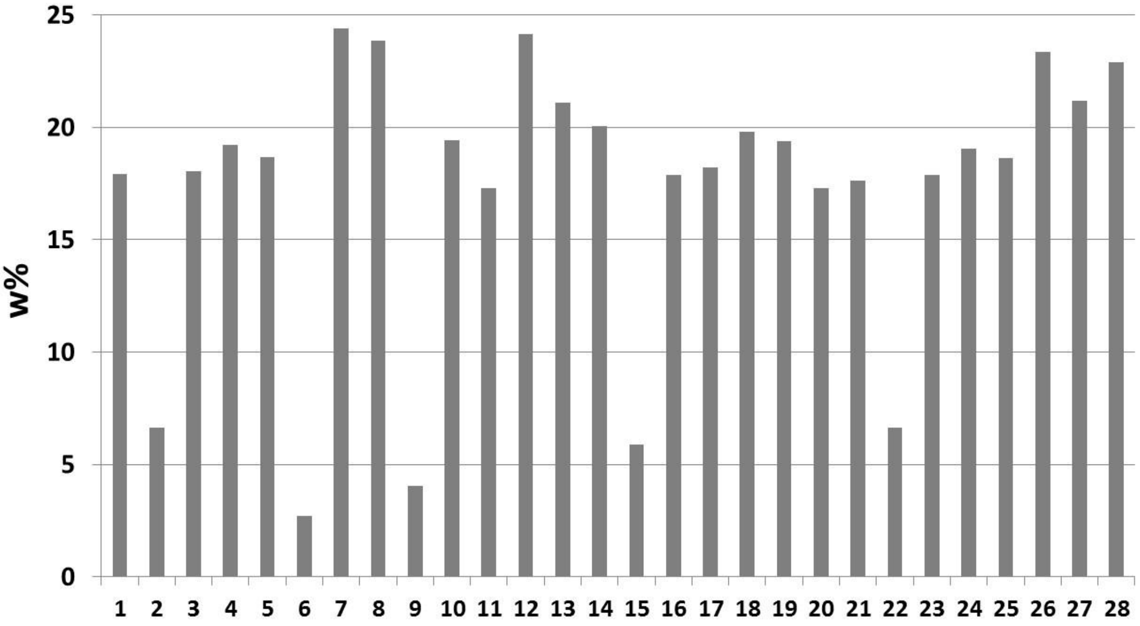


Figure 6.4: 28 samples of Steelpoort clay. Water retention at 28 MPa suction.

Figure 6.5 shows values of PI from these laboratories. The lowest PI is 17, the highest is 49 and the average is 34.2. The coefficient of variation is 33. These values correspond well with the suction results. They raise the question whether the discrepancies between commercial laboratories - which have been noted for so long by so many people - may be due not to slipshod testing, as is often thought, but to intrinsic variability in soil properties. Such variability could lead to serious consequences if ignored.

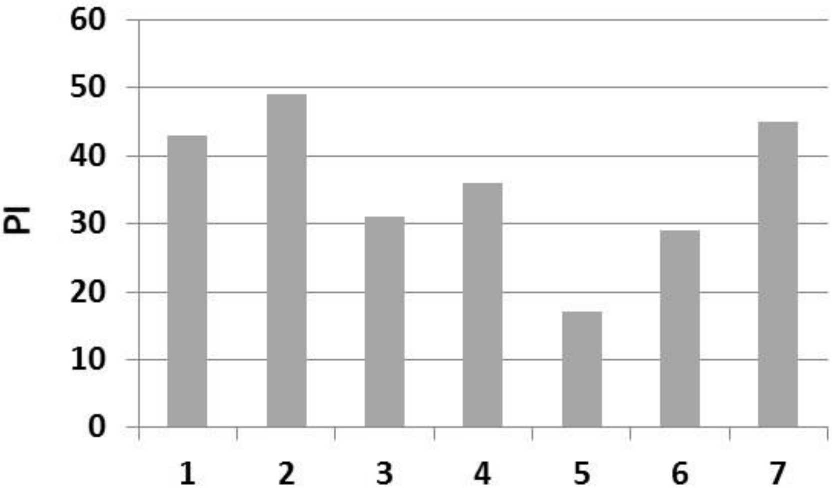


Figure 6.5: Steelpoort clay PI values from 7 soils laboratories.

Figure 6.6 shows suction values, again at 28 MPa, for a typical clayey soil from Bloemfontein. The retention values of Figure 6.6 range from 5.8 to 8.7 with mean of 6.8 and the COV is 13.1. Samples of this soil were sent to 5 commercial laboratories for testing.

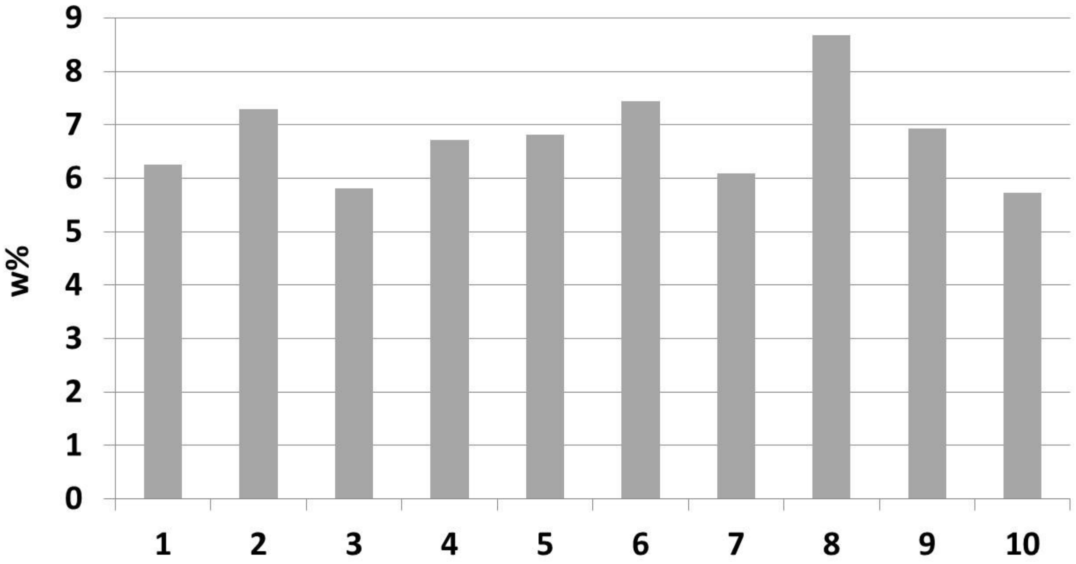


Figure 6.6: Ten samples of Brandwag clay. Water retention at 28 MPa.

Figure 6.7 shows the values of PI obtained in these tests. PI's range from 19 to 38 with mean 28 and COV 17.1. The variability here also corresponds well with that of the suction values.

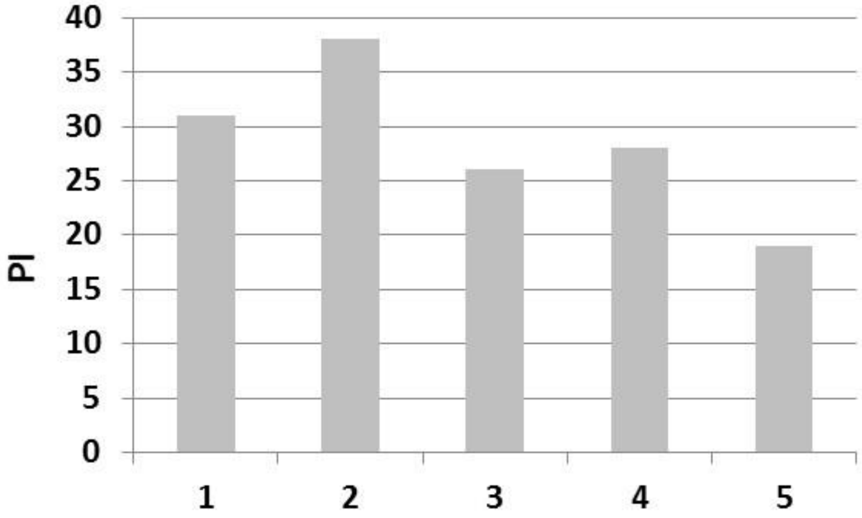


Figure 6.7: PI for Brandwag clay from 5 laboratories.

It should be noted that although PI is the most commonly used heave indicator in Southern Africa it is not a direct measure of heave. Sridharan and Prakash (2000) noted many instances of poor correlation between PI and heave potential. Suction is directly related to heave potential and may be a better indicator than PI.

6.5 Sample Preparation and Value of Suction for Consistent Comparison

If the above way of assessing soil variability is to give consistent results and allow different soils to be compared meaningfully with each other, then two features need to be considered.

6.5.1 Hysteresis

Hysteresis has a significant effect on suction values in soils. Figure 6.8 shows the water retention at suction 28 MPa of ten different clayey Free State soils from both wet and dry condition. The retention for the initially dry samples is on average 18% less than for the initially wet samples. This is due to hysteresis effects.

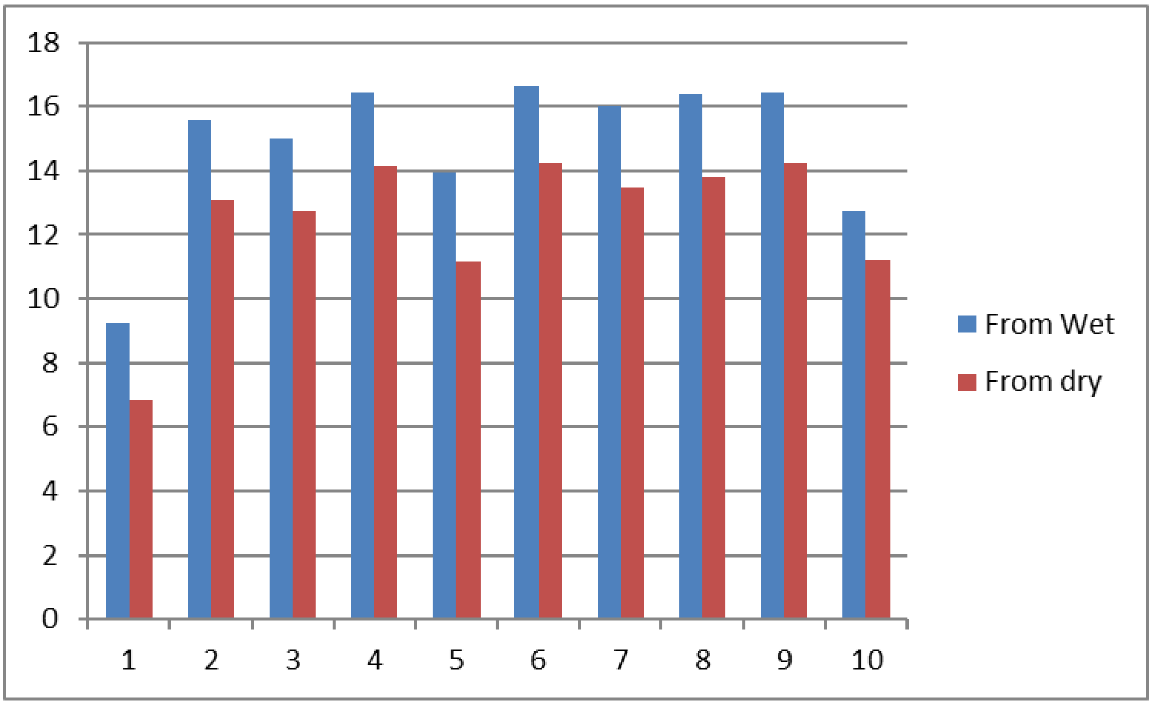


Figure 6.8: Differences in suction due to hysteresis.

On the principle of preferring in-situ conditions it might be logical to consider determining suction potential from natural moisture content. Unfortunately, sample bags are often casually treated; they are often punctured, and stored and transported in the sun. Samples often reach the testing stage at well below their original moisture content. It appears that unless a concerted effort is made to improve sample treatment and storage, the most feasible consistent procedure would be to test dried samples. A reasonable procedure might be too dry at 40 or 45 degrees, since it is unlikely that any South African soil will be dried at a temperature higher than this under all but the most exceptional field conditions.

A commercially more attractive procedure would be to dry at 105˚C so that an additional drying step to establish water content would be eliminated. A disadvantage of this is that drying is known to affect the properties of residual soils more seriously than transported soils (Blight, 2012). Investigations are being undertaken to assess this point.

6.6 Suction Value

In general, the higher the suction, the quicker a soil stabilizes to its equilibrium water-content. But as can be seen in Figures 6.1 and 6.3, the lower the suction, the greater the equilibrium water-content and hence the less sensitive the weighing procedure required to differentiate between soils. Tests were performed on a number of clays to assess the effect of suction value on COV.

Table 6.1 shows values for ten samples each of ten clayey soils at suctions of 22 MPa, 38 MPa and 180 MPa. The COV for each soil at any of the suctions measured is not far from the average of all of the values. A relatively quick test at reasonably high suction would probably give an adequate indication of variability over an appropriate range of water content.

Table 6.1: COV for 10 samples of 10 clayey soils.

Soil	22 MPa	38 MPa	180 MPa	Average
Belcher 2	15.3	15.1	15.2	15.2
Lerato Park 1	10.5	10.9	10.7	10.7
Lerato Park 2	5.2	6.0	5.2	5.5
Fichardt Park	5.0	5.8	5.6	5.5
Botshabelo R	19.4	18.9	22.0	20.1
Botshabelo B	2.8	2.6	3.3	2.9
Dersley	7.1	7.0	7.1	7.1
BK 3270	2.3	2.3	2.4	2.3
Cecelia 5A	7.4	7.5	7.6	7.5
Cecelia 5B	14.2	14.0	14.4	14.2

6.7 Conclusions

Some soils appear to show little intrinsic variability others show very large variability. Conducting one set of tests on some soils would therefore probably lead to reasonable values for use in design. In other cases, this procedure could lead to very unsound design. The investigation described here gives a reasonably quick, easy and inexpensive way of assessing soil variability. The question remains of how to proceed to design. The best way of dealing with such uncertainty is undoubtedly Reliability Based Design. This is not likely to be an attractive solution for a large number of engineering practitioners since it requires time-consuming and skills-intensive procedures like Monte-Carlo analysis. But in view of the apparent variability of some soils it might be preferable to invest the time and effort required rather than risk expensive failures.

It may also be worth noting Phoon's comment "*probabilistic techniques do exist to calculate the probability of failure efficiently. The chief drawback is that these techniques are difficult to understand for the non-specialist, but they are not necessarily difficult to implement computationally*." (Phoon, 2008 p 27). If a decision was made that such an approach was advisable, then specialists would probably be prepared to develop the required software for non-specialists at an acceptable price. Another possibility might be to accept the most unfavourable values as indicated by a variability assessment (or some statistically acceptable compromise). This would probably lead to simpler, but less economic designs than the first alternative. To do nothing, and to continue to base designs on isolated test results, is likely to perpetuate the occurrence of expensive failures - which are not at all uncommon in certain fields, such as low cost housing.

CHAPTER 7: ESTIMATION OF SHRINK/SWELL POTENTIAL AND VARIABILITY OF CLAYS BY SMALL-SCALE SUCTION TESTS

7.1 Abstract

The relationship between suction and water content gives crucial information about a soil. Small projects like economic housing do not warrant the time and cost of determining the full soil water suction curve. A considerable range of soil suctions can easily be achieved within a reasonably short time by using small samples, simple suction control and a high precision balance. It appears that in this way it may be possible to estimate heave potential and variability of soil properties at reasonable cost in an acceptable time. Variability assessment appears to be of great value and may offer significant potential for improving the reliability of foundation design on shrink/swell soils.

Keywords: Soil suction, expansive clay, shrink/swell, heave potential, Soil variability.

7.2 Introduction

The tests presented here are part of an attempt to address problems with the current commercial methods for foundation indicator tests prevalent in Africa and also in other parts of the world. Such tests often involve procedures primarily intended for road construction materials and they frequently give poor estimates of volume change potential for undisturbed, in-situ, high clay-content soils. This is not very satisfactory since shrink/swell is the most frequent cause of damage to housing and infrastructure in many countries. Considerations of economy point to a need for simple tests with minimum skilled labour content, minimum sample preparation, very little opportunity for short-cuts and straightforward interpretation of results. Tests should relate directly to expansive potential, rather than general soil properties. Suction potential gives an indication of how readily a soil can draw in water. Change in water content is the cause of volume change. Suction potential should therefore be a good indicator of swell potential. Variability in soil properties is another potential source of problems not only in design for expansive soils, but for many kinds of soils analysis. Little attention is usually drawn to it in tertiary level geotechnical engineering courses. Little attention is drawn to it in many well-known soil mechanics text books. “Craig’s Soil Mechanics” (2012) mentions variability in three places, Das (2006) makes no mention of variability in either “Principles of Geotechnical Engineering” or “Advanced Soil Mechanics”. It is mentioned once in “Soil Mechanics for Unsaturated Soils” (Fredlund and Rehardjo, 1993) and once in “Unsaturated Soil Mechanics in Engineering Practice” (Fredlund *et al.*, 2012). Blight’s “Unsaturated Soil Mechanics in Geotechnical Practice” (Blight, 2012) is one of few text books which not only stress the existence of variability but also point out the dangers of ignoring it. “Reliability-Based Design in Geotechnical Engineering” (Phoon, 2008) is dedicated to dealing with variability in several aspects of soil mechanics and merits widespread attention.

7.3 Indication of Shrink/Swell Potential

Kassa (2005) found that Atterberg limits are not a reliable indicator of volume change under load. Sridharan and Prakash (2000) found that Atterberg limits can sometimes indicate expansive potential significantly higher or significantly lower than reality. Atterberg limits remain, however, the most popular indicators of shrink/swell potential. The linear shrinkage test, performed on raw soil samples allowed to air-dry slowly, gives a graphic indication of the extent to which a soil may change volume with water content. This gives a clear and reliable indication of how much a soil may shrink or swell under zero loading, but it does not indicate the likely forces (or pressures) which can be exerted in this change of volume and therefore the shrink/swell potential under load. Clays expand when they draw in water. The force which can be exerted in this expansion depends on the strength with which they can suck in water. The suction of clays has long been recognized as a cardinal indicator of heave potential, but it is one of the least convenient indicators to measure. The plot of suction from saturation to desiccation is known as the soil water characteristic curve (SWCC). The SWCC is regarded as an indispensable part of a full unsaturated analysis (Fredlund *et al.*, 2012), but it requires much time, skill and expense to measure.

7.4 Recent Advances and Current Practice in Suction Measurement

Three papers presented at the International conference on Soil Mechanics and Geotechnical Engineering in Paris 2013 deal with advances in techniques of suction measurement. Two other papers deal with measuring suction in practical engineering projects. Advances in dew-point potentiometer technology should lead to extension of the range of suctions measurable by this type of instrument (Macek *et al*., 2013). The use of micro-porous membranes may allow quicker suction measurement in the 0-30 kPa suction range (Nishimura, 2013). The use of a centrifuge may speed up measurement in the 0 to 900 kPa range (Reis *et al.*, 2013). Such advances should hopefully lead to quick, convenient and economic assessment of the full range of soil suctions in the future. But in practical use, for assessing lime treatment on London Clay (Mavroulidou *et al.,* 2013) and for modeling the impact of climate changes on embankments and cuttings (Mendes and Toll, 2013), Whatman No.42 filter paper remains the method of choice. This time-honoured system takes typically two to six weeks for a suction measurement and requires careful laboratory technique (Bulut *et al.*, 2001).

7.5 Small Scale Suction Potential Measurement

A more limited indication of suction potential can be found by allowing samples to reach equilibrium at known temperature and humidity. Soil samples can be brought to equilibrium over saturated solutions of various salts (Blight, 2013). Blight’s tests took typically 90 days. Although his equipment was cheap and unsophisticated, and the skilled labour component not large, the time frame is not feasible for normal engineering practice. In this paper the authors introduce modifications to Blight’s procedure which give quicker results, making enquiry into some important questions feasible.

This procedure involves using small pieces broken from a soil specimen. Breakage in high clay-content soils usually occurs along planes of existing weakness; the micro-structure and fabric of the soil are not greatly disturbed (See Figure 7.1). Samples are placed in small glass weighing bottles with ground-in lids. When closed, little air or water vapour leaves or enters in the time taken for weighing. Samples are weighed on an analytical balance and then placed with lids open in a container at controlled temperature and humidity (see Figure 7.1). Equilibrium moisture content is a measure of the suction potential of the soil under those conditions.

Figure 7.1: Above: samples in climate chamber. Temperature and humidity can be controlled to give a wide range of suction. Below samples in a readily obtainable storage container whose lid is air-tight. Sample bottles stand on a perforated platform above a solution whose vapour pressure is known at various temperatures.

Samples can be weighed periodically by closing the ground-glass lids and removing them from the controlled atmosphere to the analytical balance. For many of the tests performed in this investigation suctions corresponding to saturated solutions of KCl and NaCl were used, though both higher and lower suctions were also employed (as for example in generating the SWCCs shown in Figure 7.22). A climate chamber was used for many of the tests, often at settings which give the same suctions as KCl and NaCl. A climate chamber has the advantage of convenient control over a wide range of suctions and quicker equilibration due to active circulation. A description and flow-diagram of a streamlined procedure suitable for routine testing is detailed in section 10.

Figure 7.2 shows curves of moisture content against time at constant suction for samples of 30 clayey soils from a housing development in central South Africa. These 30 samples cover a very wide range of shrink/swell potential. The masses of the samples range between 25g and 35g. Starting water contents were as found in the sample bags and would probably be close to natural field values in most cases. The equilibrium moisture contents at 22 MPa suction (corresponding to a saturated solution of KCl at 20˚C) vary from 3.6% to 12.6%.

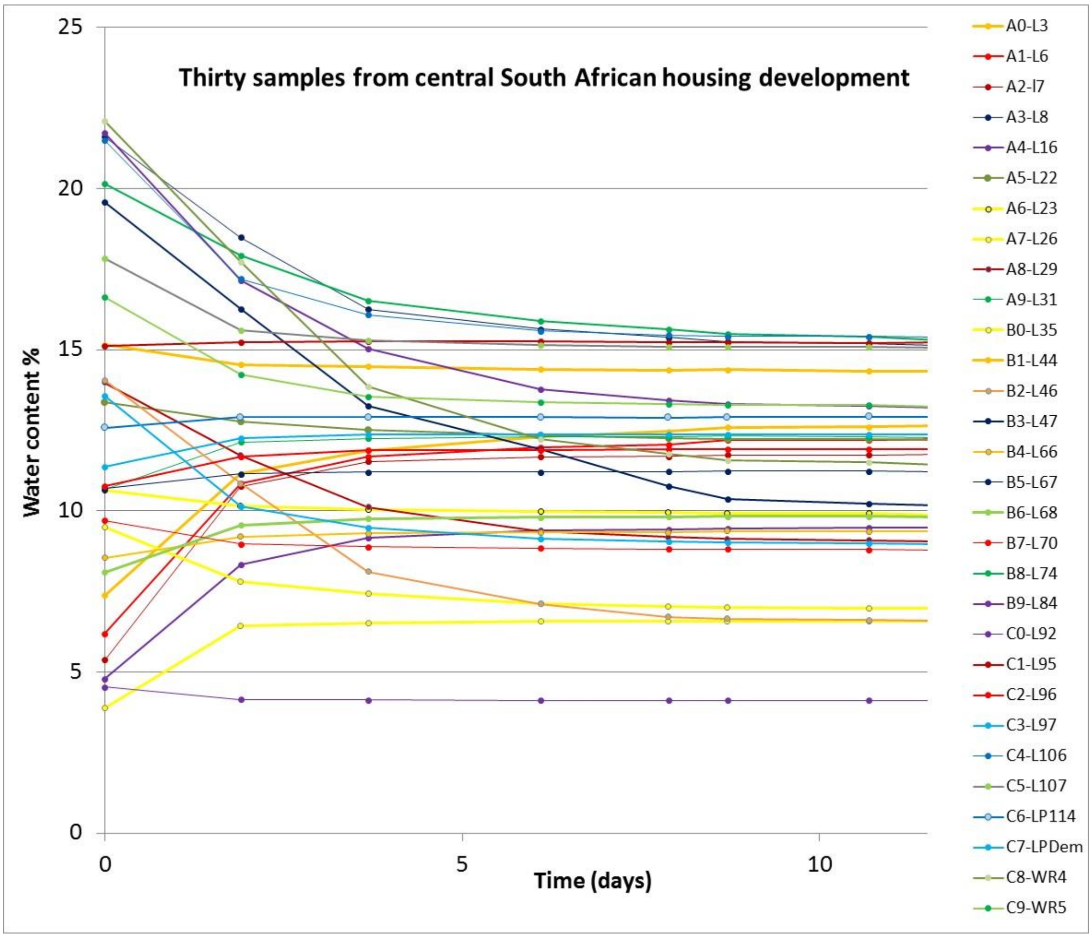


Figure 7.2: Thirty samples at 22 MPa suction.

Correspondence between moisture retention and heave potential appears to be supported by the fact that the highest water retention samples have Plasticity Index (PI) in the region of 40, which would normally be considered an indication of high expansive potential. Those with the lowest water retention have PI below 15, which would normally be considered an indication of low expansive potential. More detailed tests have shown high coefficient of variability for many clayey soils, as can be seen in Figures 7.17 and 7.18 and also Figures 7.19, 7.20 and 7.21. Comparisons of expansive potential are therefore only loosely indicated by single tests. Multiple duplicate testing is needed for worthwhile comparisons of shrink/swell potential, as discussed in section 7.6.

These 30 samples were used to probe questions concerning changes in suction due to conditions likely to be met in the lifetime of a structure (particularly prolonged rainfall and drought) and the possibility of change in suction potential due to oven drying of test specimens. These questions are independent of the variability noted above and the conclusions are meaningful for individual or multiple samples of each soil. Figures 7.3 to 7.5 show these 30 samples being tracked from equilibrium moisture content at 22 MPa, through oven drying and re-wetting by absorption of water from the air (Figure 7.3), wetting by the addition of a small amount of de-ionized water and re-equilibrating at 22 MPa (Figure 7.4), saturation with de-ionized water and re-equilibrating at 22 MPa (Figure 7.5).

It is clear from Figure 7.3 that the equilibrium water retention at 22 MPa suction is noticeably less after oven drying than before drying. Figure 7. 4 shows that equilibrium water retention after wetting to close to natural moisture content is higher than before wetting, but not as high as before oven drying. Relative values of suction between the various samples remain quite similar.

From Figure 7.5 it is clear that the equilibrium water retention at 22 MPa suction after saturation is higher than after partial wetting, but in most cases it does not reach the value attained before oven drying.

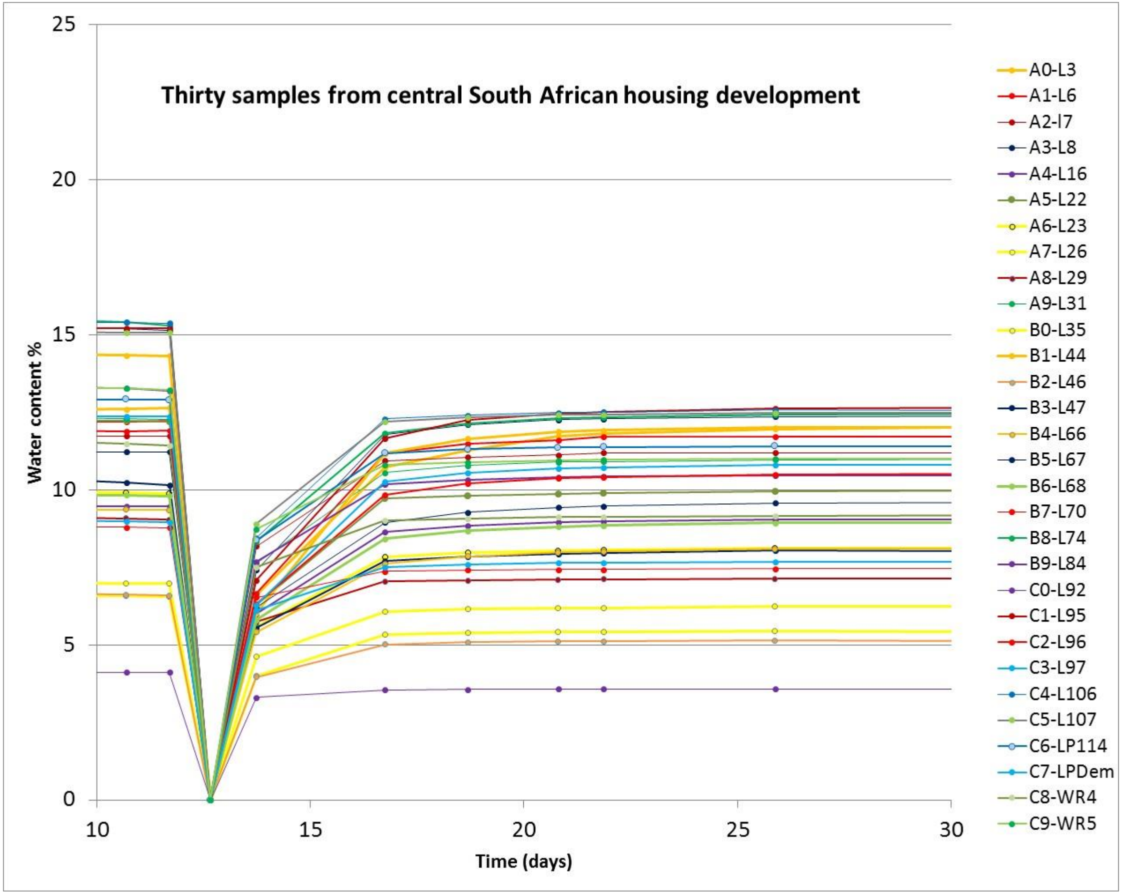


Figure 7.3: Thirty samples after oven drying.

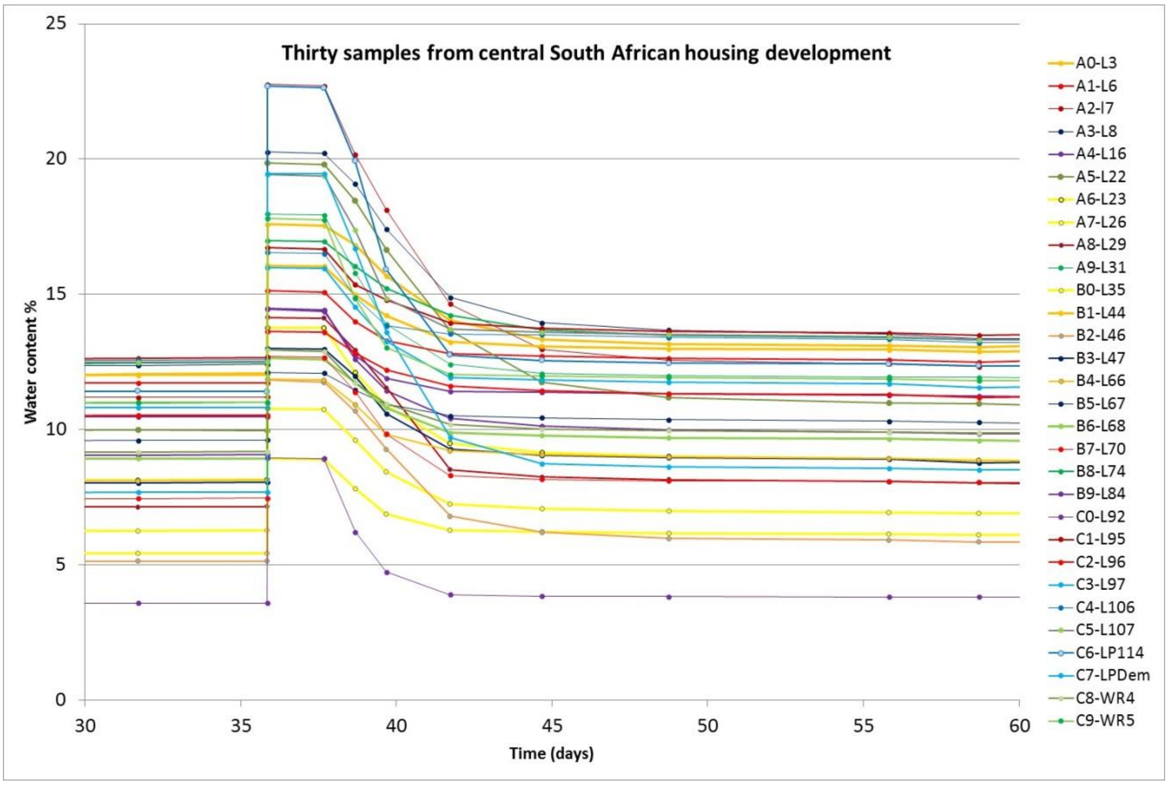


Figure 7.4: Thirty samples after partial wetting.

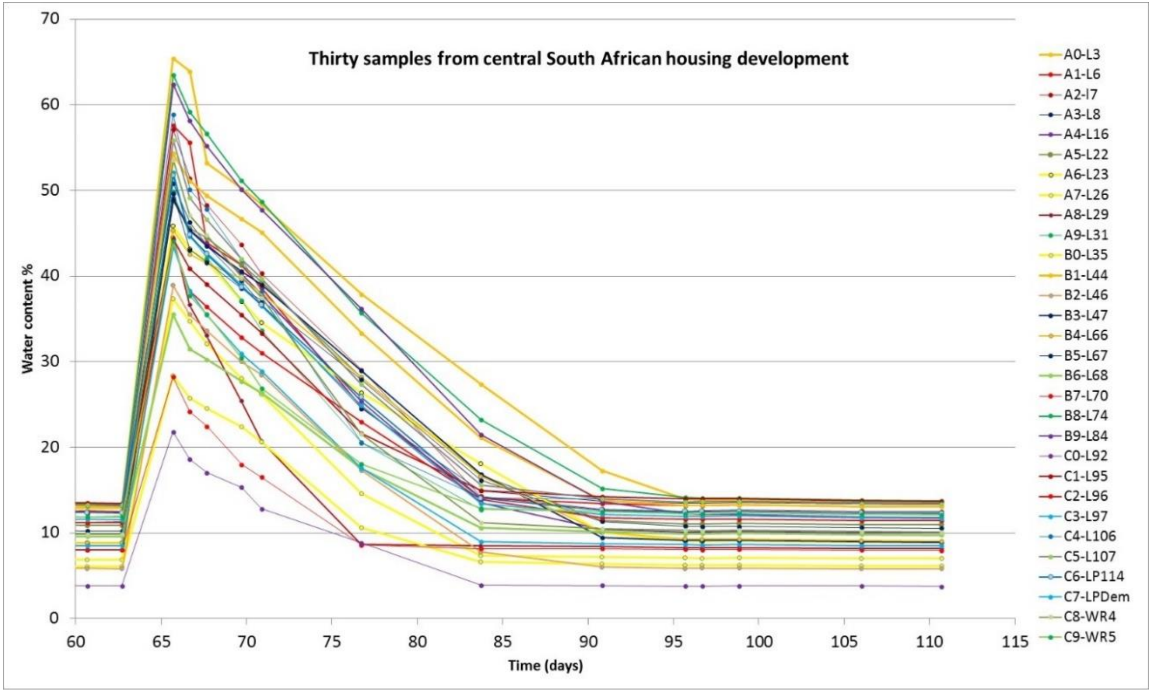


Figure 7.5: Thirty samples after saturation.

7.5.1 Discussion of results from this test

From initial (natural) moisture content the low-suction samples reached equilibrium moisture content after about 4 days, the high-suction samples reached constant water content after about 9 days. After oven drying and returning to 22 MPa suction conditions, all samples reached equilibrium after about 5 days. This indicates that equilibrium is reached more quickly in gaining water content than in losing it. In most cases the equilibrium water content was significantly lower than that reached before oven drying. After wetting to water contents close to, or slightly higher than, the original values and re-stabilizing at 22 MPa, all of the samples reached moisture contents higher than the value after oven drying, but some were substantially lower than the original values.

Following saturation, most of the samples reached equilibrium at somewhat higher moisture content, but lower than that reached before oven drying. Figure 7.6 shows moisture content changes as a bar chart. Samples whose initial water content was very low are grouped predominantly on the left side. Where the initial water content of the soil was well above its 22 MPa value, oven drying significantly reduced the soil’s subsequent suction potential, and even saturation did not restore the original suction potential. Where the initial water content was well below the 22 MPa value, the soil was not so greatly affected by oven drying and saturation could lead to higher retained moisture content at 22 MPa.

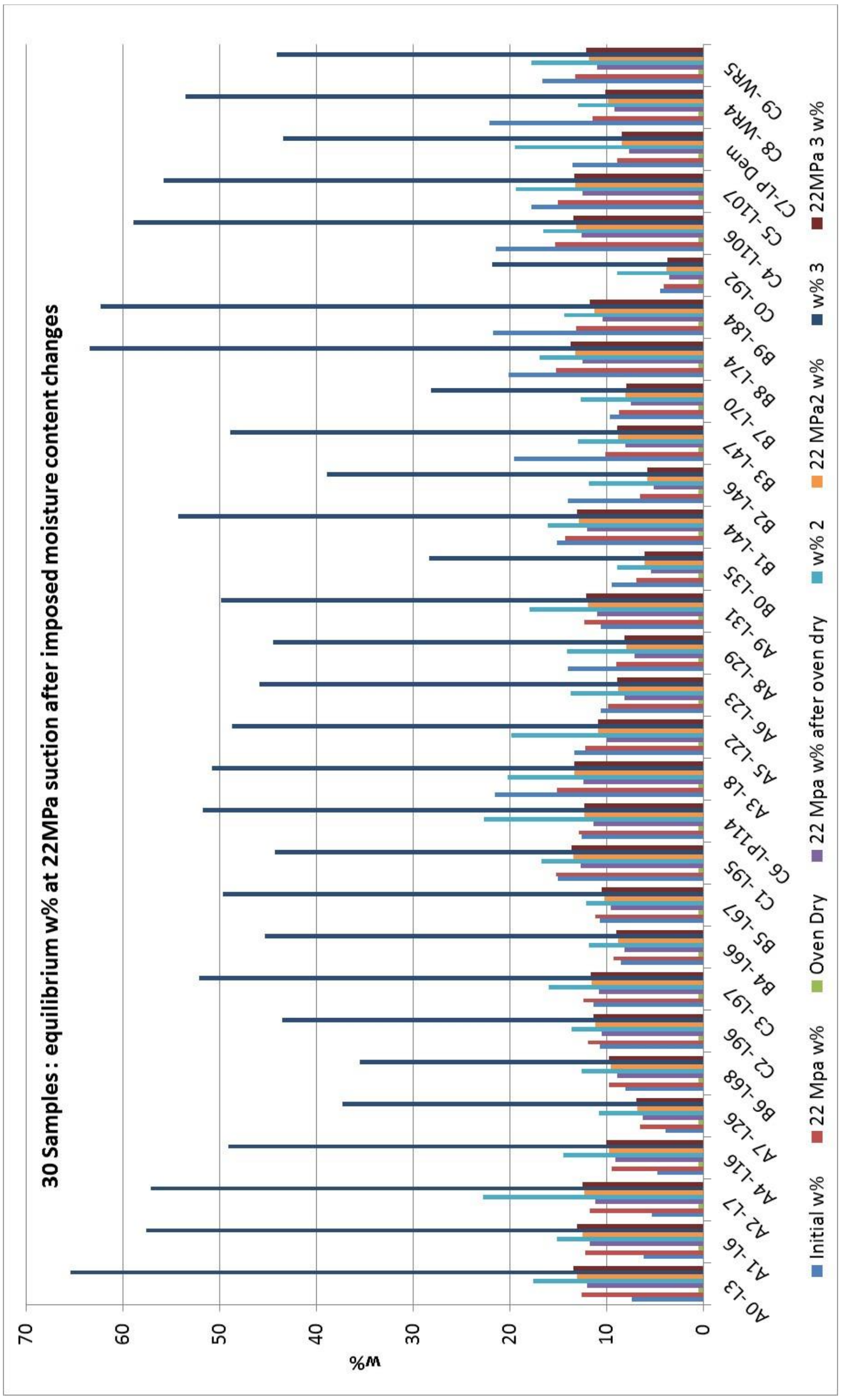


Figure 7.6: Water content at 22 MPa.

Figure 7.6 shows the water content reached by 30 samples at 22 MPa from initial moisture content, after oven drying, after wetting and after saturation.

This suggests that Blight's contention (Blight, 2012) that air drying can lead to permanent changes in some residual soils may commonly apply to many, if not most, clayey soils. These points can be seen more clearly in Figure 7.6 a, enlarged from the right hand side of Figure 7.6, and in Figure 7.6 b, enlarged from the left hand side of Figure 7.6.

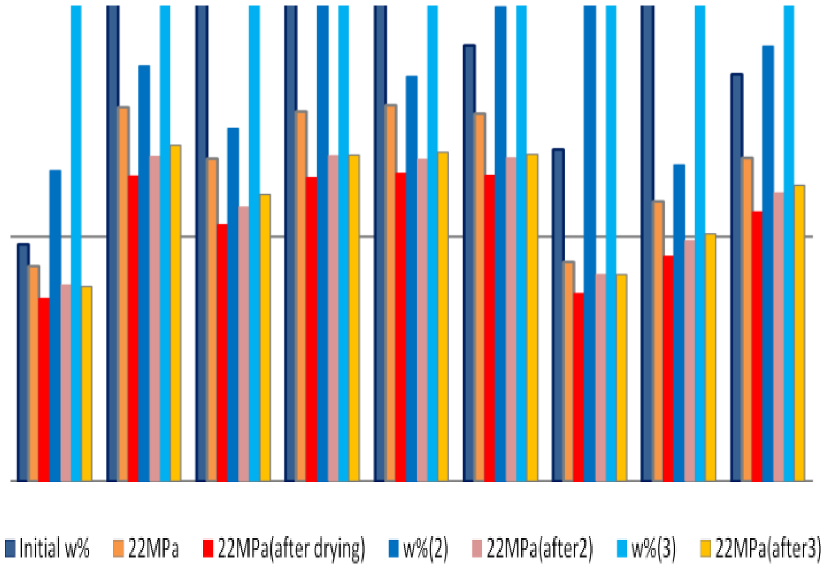


Figure 7.6a: Initial water content well above that at 22 MPa suction.

From Figure 7.6a it is clear that the water retention after oven drying is considerably less than the value before drying and pre-oven drying water retention is not recovered even after full saturation. This suggests that the structure of the clay has been permanently changed by oven drying.

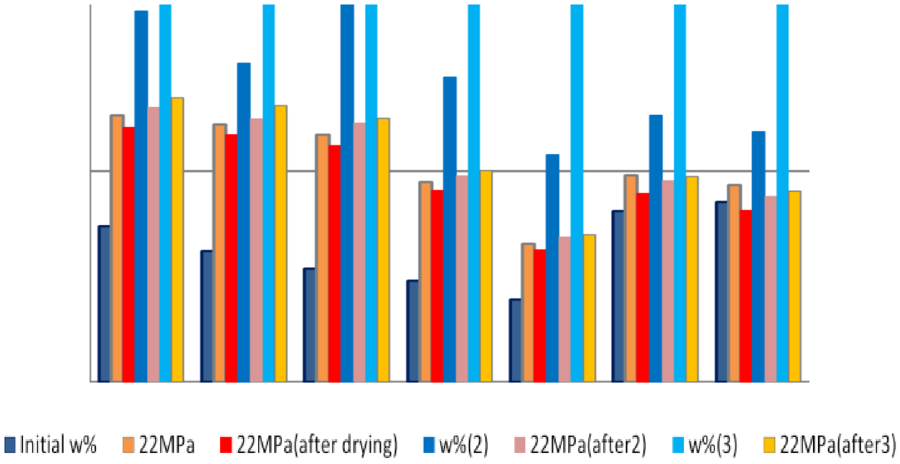


Figure 7.6b: Initial water content below that retained at 22 MPa suction.

Figure 7.6b shows that water retention is only slightly less after oven drying and retention after saturation is usually greater than before oven drying. This suggests that part of the permanent structural change noted in oven drying had already been accomplished by field moisture conditions.

This test also suggests that in the majority of cases, whatever suction changes take place, the ratios of water content between the various samples remain substantially the same. The suction-history pattern moves up or down according to the general suction potential of the sample. In drought conditions soil suction can be expected to rise to 100 MPa or more in some situations e.g. under the North side of a Southern hemisphere building (Bester *et al.*, 2016). Oven drying might therefore not give a major distortion in many cases.

The consistent relationship over a wide range of suctions has been observed for hundreds of other samples besides those shown here, but some soils have been found which do not follow this pattern at very low suction/high water content values. High kaolinite content clays appear to retain more water at very low suction/high water content than would be expected according to the normal pattern. Some very high Cation Exchange Capacity (CEC) clays tend to retain less water than expected according to the normal pattern at very low suction/high water content). This occasional lack of conformity to pattern is likely to have little relevance for the problem of heaving foundations since it is generally high suction which causes shrink/swell problems. Kaolinite has low suction potential at any water content and is not known for causing shrink/swell damage.

High CEC clays generally cause severe volume-change problems and the small reduction sometimes observed at high water content does not significantly change their problematic nature. Of more relevance to the question of economically assessing expansive potential is the fact that equilibrium water content is reached far more quickly at high suctions than at low values. Equilibrium is also reached more quickly from initially dry conditions than from wet. This suggests that testing at fairly high suction from low initial moisture content could give useful results in an economic time-frame.

7.5.2 Reducing the time of testing by reducing sample size

It could be expected that smaller samples would take less time to reach equilibrium. Figure 7.7 shows small samples broken from one corner of a small clod of high plasticity clay from an electricity sub-station site in central South Africa. The samples were originally adjacent to each other in an attempt to eliminate variability in the material, so that difference in results would be attributable to sample size only.

Figure 7.7: Small samples broken from a clod of clay.

Samples tend to break along existing planes of weakness and their structure and fabric are likely to be similar to that of the original sample. Such simple preparation takes little time and leaves little opportunity for short-cuts in preparation, which is widely thought to be a cause of inconsistent results in many kinds of tests. Figure 7.8 shows plots of moisture content against time for seven different-sized small samples of the same clay (PI = 42) at 22 MPa suction. The sample masses vary from 0.71g to 8.4g.

All seven samples were close to equilibrium water content after one day in a climate chamber at 22 Mpa. Equilibrium value reached by the 7 samples averaged 13.56% with standard deviation of 0.30 giving a Coefficient of Variation (COV) of 2.2. The correlation coefficient with sample-size was 0.17, suggesting that there is no significant correlation between sample size and test outcome for this range of sample masses.

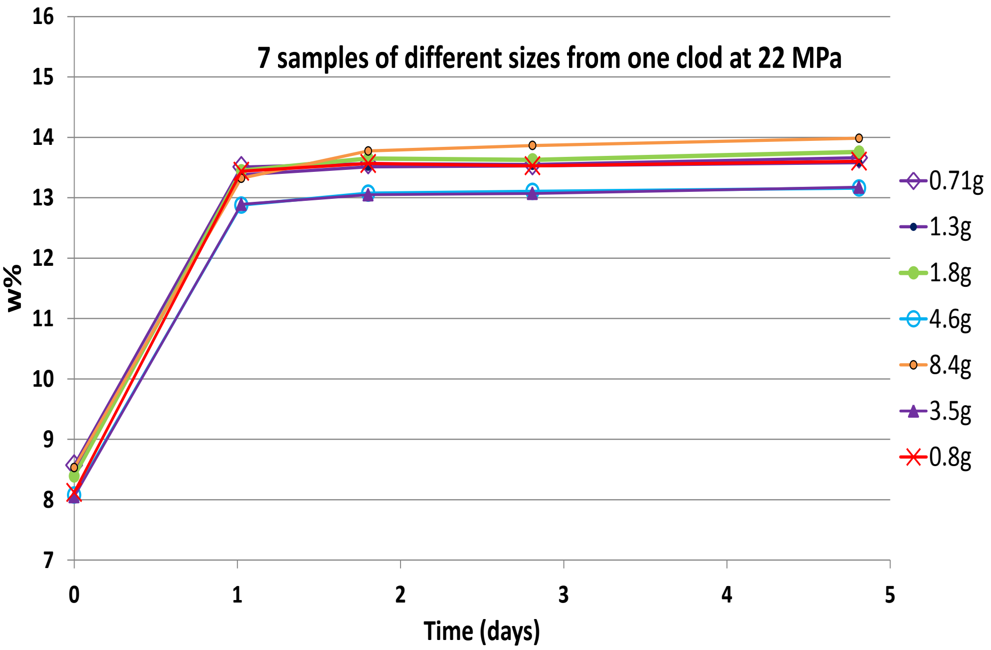


Figure 7.8: Equilibration of 7 samples (0.71g to 8.4g) at 22 MPa suction.

7.6 Assessment of Likely Sources of Error

The range of variation in the above test raises the question of the accuracy of the experimental procedure. Reliability of the method depends on accuracy in weighing. The aspect of the procedure most likely to involve weighing error is the determination of the oven-dry mass. To assess the probable errors in this, samples of 10 different soils were oven dried and the change in weights after leaving the oven were tracked until equilibrium mass was reached. The loading plate of the balance was protected with an expanded polystyrene pad to reduce conduction of heat into the balance and hence reduce temperature induced errors. The enclosure around the loading plate was closed to reduce convection current effects and the samples were taken through the weighing procedure as quickly as possible to minimize temperature change effects in the balance. Samples were repeatedly weighed in sequence until all showed equilibrium of readings. Results are plotted in Figure 7.9.

In the cooling of the sample containers from oven drying temperature (105˚C) to room temperature the change of weight of air in the container must be accounted for since the ground glass lids of the sample bottles are not air-tight against the pressure differential developed in the cooling process. The volume of the sample containers was measured to be 70 ml. The density of dry air at 105˚C is 0.0009217 g/ml, giving the weight of air in the container as 0.0645g. At the ambient temperature at the time of the test (25˚C) the density of dry air is 0.001196 g/ml giving the weight of dry air in the container as 0.0837g. Relative humidity in the laboratory was 36%. The adjustment required for density to take account of water vapour at this humidity is 0.000003 g/ml giving a weight adjustment of 0.0002g The change in weight due to air density factors could therefore be expected to be 0.0837 + 0.0002 – 0.0645g = 0.0194g.

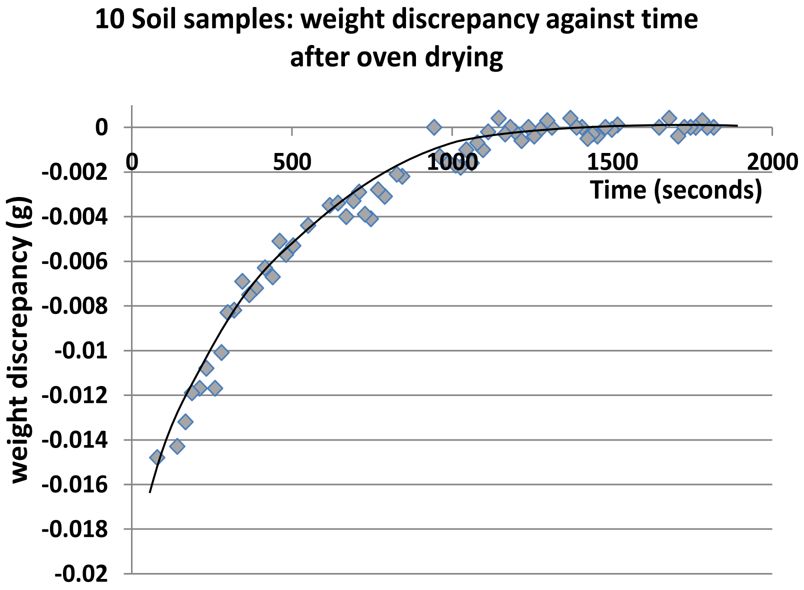


Figure 7.9: Progress towards constant temperature.

Figure 7.9 shows progress towards constant temperature of 10 samples from oven drying. Extrapolating to time of leaving the oven shows a gain in weight up to a constant value of about 0.019g after about 1500 seconds (25 minutes).

Similar tests were performed on three more sets of ten samples. All showed similar results and it was concluded that if samples were weighed after 25 minutes of cooling at ambient temperature with lids closed the results were likely to be accurate to within 0.001g. The smallest sample mass in the test under review was 0.7129g. An error of 0.001g could account for an error of 0.14% in moisture content. This does not account for the observed range of almost 1%.

A second possible source of error is the leakage of water vapour into or out of the weighing bottles while closing the lids on removal from the constant suction container before weighing. Samples were usually dealt with in batches of 20. Closure was always performed with both hands to halve closure time. The time to close the lids of all 20 samples was usually 18 seconds, and it is unlikely that the time could have reached 25 seconds in any test.

On six occasions the first sample of a batch was transferred immediately from the constant suction container to the balance. Figure 7.10 shows change in mass with time for a sample left with lid open for 3 minutes. Throughout this time the loss of water vapour was fairly constant at approximately 17.9 x10^{-6} g/s. The rates for the six samples were: 10.9 x10^{-6} g/s,17.9 x10^{-6} g/s, 14.8 x10^{-6} g/s, 9.5 x10^{-6} g/s, 12.5 x10^{-6} and 13.6 x10^{-6} g/s (average 13.2 x10^{-6} g/s). The maximum observed rate of moisture loss was 17.9 x10^{-6} g/s, which would mean the loss of 25 x 17.9 x10^{-6} g = 0.0004g for the last sample closed at the slowest probable handling rate (25 s.).

Figure 7.10 shows the loss of weight due to loss of water vapour with the sample bottle open after removal from a sealed container of saturated KCl solution to an analytical balance.

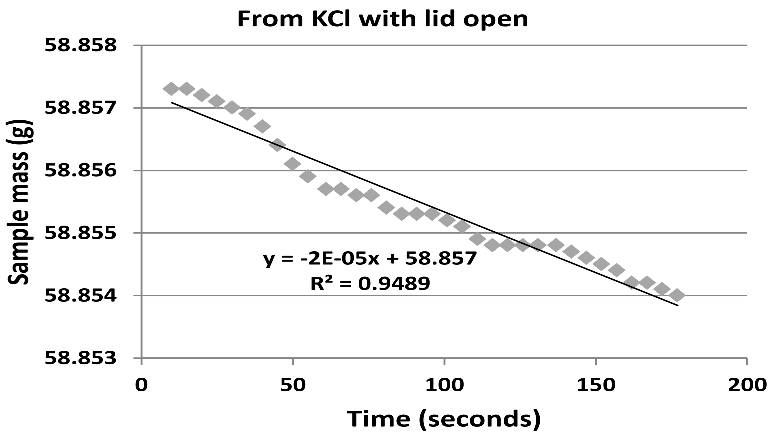


Figure 7.10: Loss of weight due to loss of water-vapour.

A third possible source of error is transfer of water vapour to or from the atmosphere due to imperfections in the air-tightness of the ground glass lids of the sample bottles. After removing 20 samples from the constant suction chamber and closing all lids, 330 seconds are typically required to weigh the 20 samples. It is conceivable that the procedure might take 500 seconds if unfavourable conditions led to unusually slow stabilization of the balance.

This possible source of error was examined by repeated sequential weighing of 10 samples after removal from an atmosphere of KCl and closing the lids of the sample bottles. The procedure was similar to that illustrated in Figure 7.9, where progress towards equilibrium was plotted until constant mass was achieved. Figure 7.11 shows the results of this investigation.

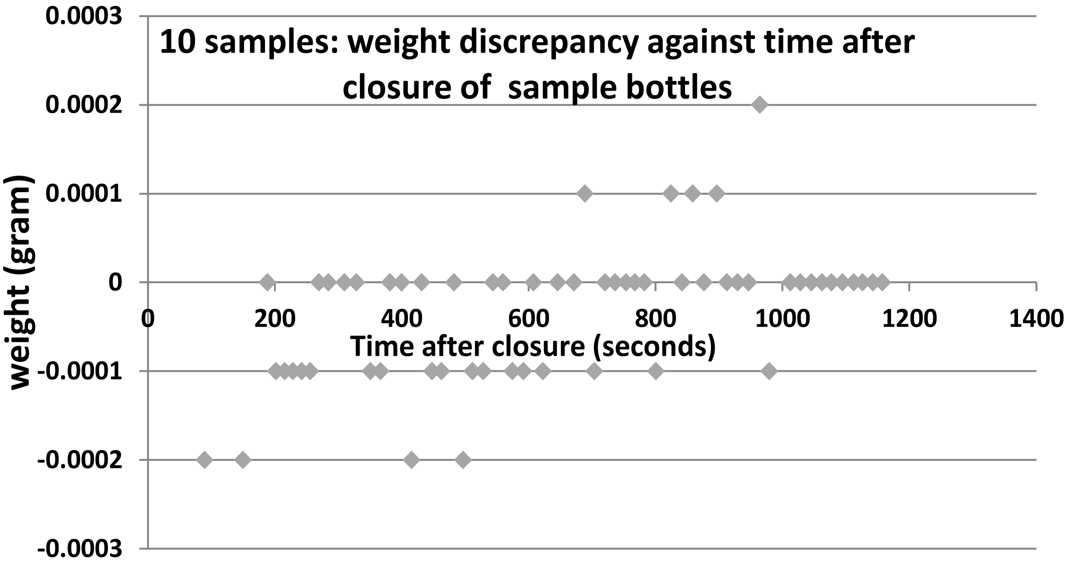


Figure 7.11: Weight discrepancy against time

Figure 7.11 shows that no clear trend of either losing or gaining weight is discernible. The majority of readings start and remain at the equilibrium value. Results not at the equilibrium value are all within 0.0002g of that value. The balance reads to 0.0001g; the pattern of results suggests that the balance may have an accuracy of ± 0.0002g and that in the 20 minutes' duration of the test, transfer of moisture to from the closed bottles was not large enough to be demonstrated by this balance.

Although tests performed over periods of the order of 20 minutes did not detect a clear pattern of water vapour movement into or out of the container, it has been observed that over a period of several days there is certainly such movement and the samples gradually move towards equilibrium with ambient conditions.

The fourth possible source of error considered was inaccuracy of the balance itself. The balance used throughout these tests has an internal calibration mass and can be set to check consistency by weighing this mass ten times and assessing the standard deviation in the ten values. The consistency was checked periodically throughout the tests. The reported standard deviation was typically 0.00005g, and in all cases less than 0.0001g. As an additional check, two empty sample bottles were weighed repeatedly, two weighing sequences for each bottle. In no case was any measurement more than 0.0001g above or below the dominant value. From this it would appear that the balance readings are probably accurate to ± 0.0001g if good laboratory technique is followed. Following these investigations into likely accuracy, a test was run where samples were closed and weighed alternately in ascending and descending numerical order on removal from the constant suction environment. It was expected that there might be small relative displacements between alternate measurements for the first and last closed samples due to the considerations illustrated in Figure 7.10. The resulting retention values can be seen in Figure 7.12, where the alternately first and last sealed and weighed samples for one of the soils have accentuated markers. Part of the plot after stability was reached is magnified at the lower part of the figure (where the curves of other samples have been omitted for clarity).

It might have been expected that a slight, alternating narrowing and widening of the gap between first and last samples would result from water vapour loss in the 18 seconds between closures. From the average rate of loss determined above this might be expected to be about 0.0004g. The dry weights of the first and last samples were 1.8656g and 2.0735g respectively. Allowing for possible balance errors of 0.0001g in worst combination this would suggest a likely change in water content of no more than 0.032%. Without allowance for worst-case balance error the likely change expected would be not more than 0.021%. The plots in Figure 7.12 show variations of about 0.05% in which the values for the samples move largely in sympathy. This suggests that limiting factors for accuracy may not depend on mass measurement considerations but rather on considerations like lack of constancy in temperature and humidity control. Nevertheless, it appears that the method is capable of distinguishing between suction potentials of different soils and different samples of the same soil to a high degree of accuracy. It also appears capable of detecting variability in suction potential of a particular soil over a very small spatial range.

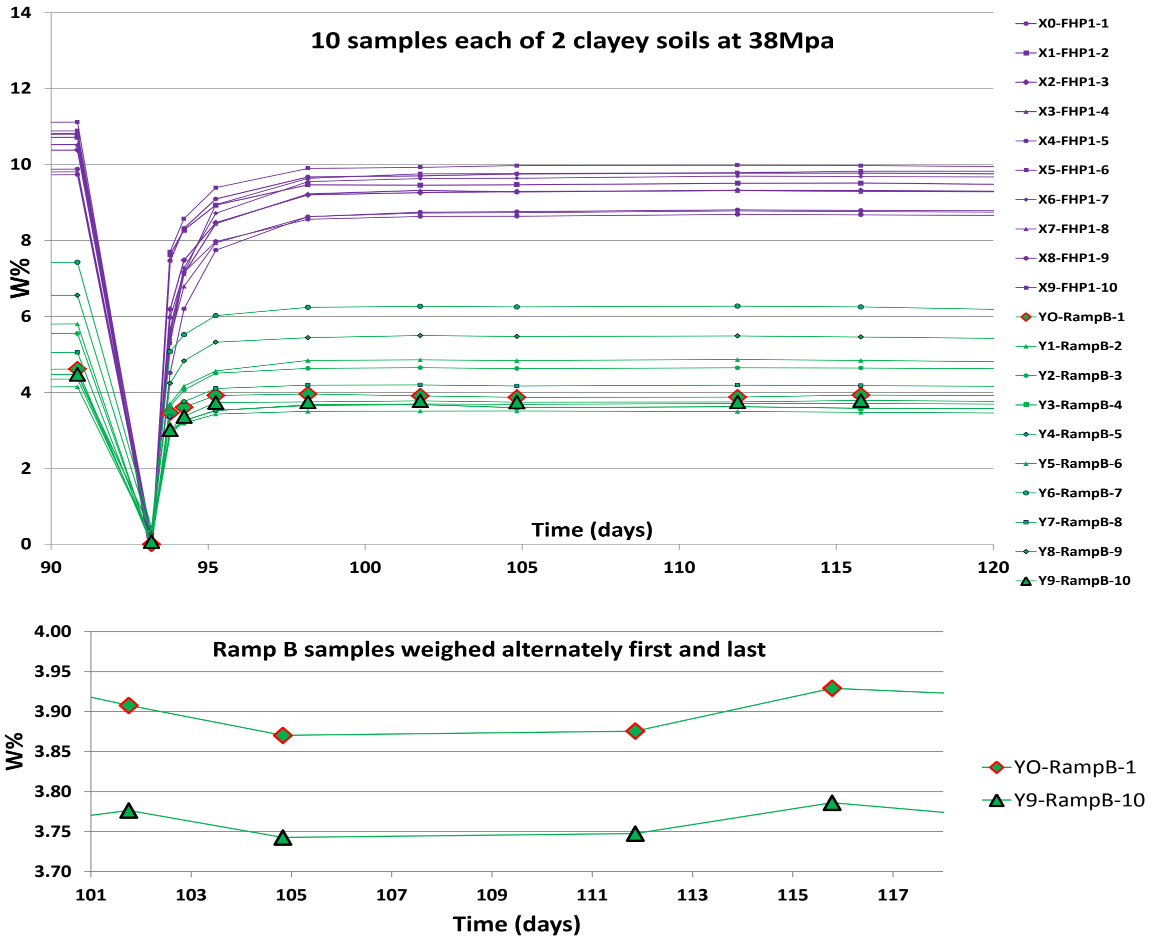


Figure 7.12: Samples tested in alternately reversed order. Part of the graph is magnified (below) where only the first and last closed values for one of the soils are shown.

7.7 Assessment of Variability of Soil Properties

Variability of soil properties has been noted by many observers, (e.g. Singh and Lee, 1970; Minty *et al.*,1979; Phoon and Kulhawy,1999a; Phoon and Kulhawy ,1999b; Jaksa, 1995). There appears to have been relatively little heed paid to this observation by many practicing engineers. Little attention is usually drawn to it in tertiary level geotechnical engineering courses or popular text books. It has been suggested that variability in measured properties may be due to differences in operator technique and laboratory processes (Minty *et al.*, 1979), or careless laboratory procedures (Jacobsz and Day, 2008). Figure 7.12 (upper) shows effective suction potential for two soils which received practically identical treatment and minimal preparation, by one operator. They show substantial variability between individual samples and one soil shows more than twice the variability of the other.

This suggests that variability may be a property of the soil rather than an artifact due to poor testing procedures. Whatever the reason for variability it is clear that it must play a role in the soundness of geotechnical design. The field of Reliability Based Design (RBD) has been developed to provide a rational way of taking variability of various kinds into consideration in assessing the probability of success or failure of geotechnical designs. Coefficient of Variation (COV) is a fundamental input to RBD. COV is defined as standard deviation divided by mean expressed as a percentage. COV is typically assessed by considering variability across soils databases and across testing methods for the property under consideration rather than being sample specific. Hence rather crude approximations are commonly used. For example, Phoon and Ching (2013) give a range (in their Table 1) of mean for PI of clay and silt (10-40) and guideline estimate of COV as (3-12%)/mean.

Such empirical estimates imply considerable lack of precision and could be a contributing factor to reluctance towards application of RBD in engineering practice. The small-scale suction testing procedure described here allows an assessment of actual sample-specific variability relatively quickly and easily. The authors have related such specific assessments to some engineering projects and the following brief case studies illustrate that they can be of value in practical situations even without rigorous application of RBD.

7.7.1 Five brief variability case studies

Case1. Five test pits were dug in a geotechnical investigation at a building site in Central South Africa. A layer of clearly identifiable dark brown residual clay was evident in four of these pits. Five samples each from the first two pits were tested by small-scale suction tests. They showed a COV of 27. Five samples from each of the other two pits were tested. They showed a COV of 25.

The COV of the combined 20 samples was 27. Figure 7.13 shows the combined values. It appears that high variability may be a consistent property of this soil and was taken into account in the design of the new structure. When access roads were constructed near this site several years ago the roads developed an undulating profile. Removal of underlying clay was needed to reach an acceptable standard. Early appreciation of the variability of the material could have led to its removal initially and saved considerable expense.

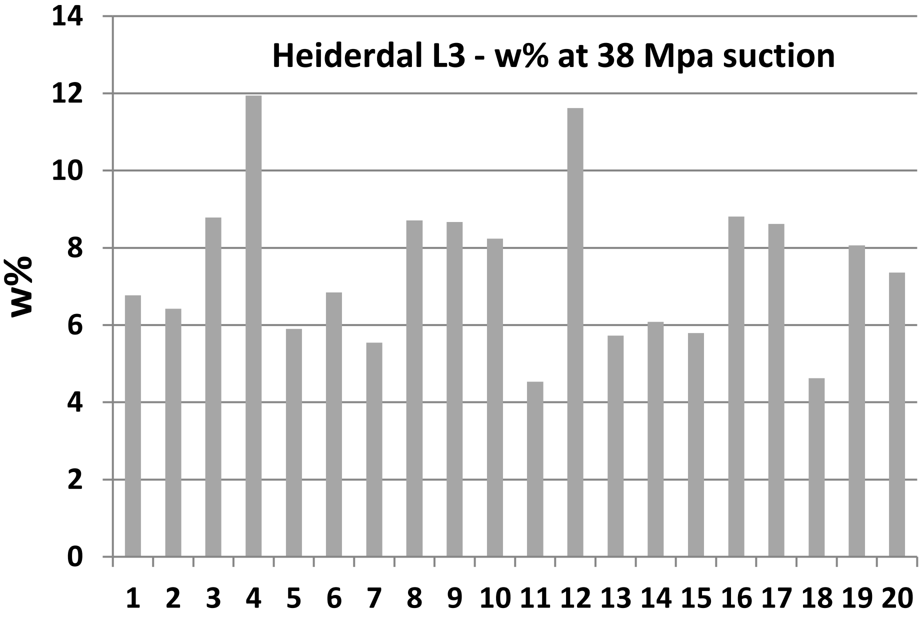


Figure 7.13: Water retention at 38 MPa suction.

Figure 7.13 indicates the water retention at 38 MPa suction for 20 samples from a layer of residual clay in 4 test pits.

Case 2. A similar situation was noted at a road project on a similar geological formation about 150km away. Undulations in the road indicate wide variation in expansive potential of the sub-grade. Tests on soil from an adjacent housing project which has suffered significant heave damage showed substantial variability in suction potential.

Figure 7.14: Undulations in road.

Case 3. Samples were taken from the proposed site of an electricity sub-station in Central South Africa. A clod from this site was used for the test of Figure 7.7. Samples were broken from adjacent locations on one lump of soil in an attempt to eliminate variability of material so that the effect of sample size only would be assessed. Water retention at 22 MPa varied by a little under 1%, showing a co-efficient of variation (COV) of 2.2.

Ten samples from a test pit at this site were tested. A COV of 2.4 was found in a test at 22 MPa suction. Ten samples from a second test pit on the site were tested. A COV of 2.5 was found when tested at 22 MPa and also when tested at 140 MPa. Results are shown in Figure 7.15 and Figure 7.16. Low variability appears to be a consistent property of this soil. All values indicated high expansive potential. Foundation indicator tests were performed on a number of samples and all tests indicated high expansive potential with very little variability. The foundation was designed accordingly and no significant problems occurred. This suggests that where variability is low, current methods of design may be adequate.

Figure 7.15 shows the water retention at 22 MPa from a test pit at the electricity sub-station in central South Africa. This clay gave almost the same COV from three different scales of distribution and suggests a possible fractal aspect to variability.

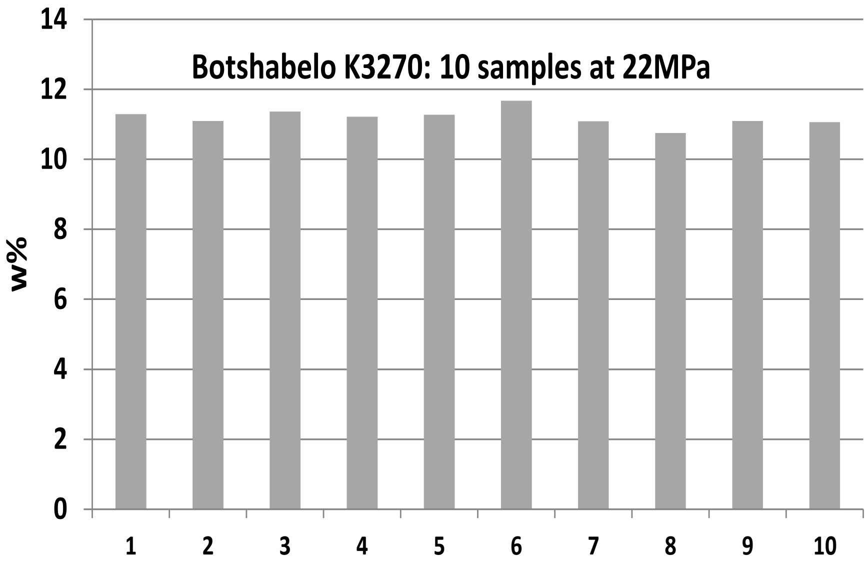


Figure 7.15: Water retention at 22 MPa.

Figure 7.16 shows water retention at 22 MPa and 140 MPa in a second test pit for the same clay layer as in Figure 7.15 at an electricity sub-station site in Central South Africa. COV is 2.5 in both cases.

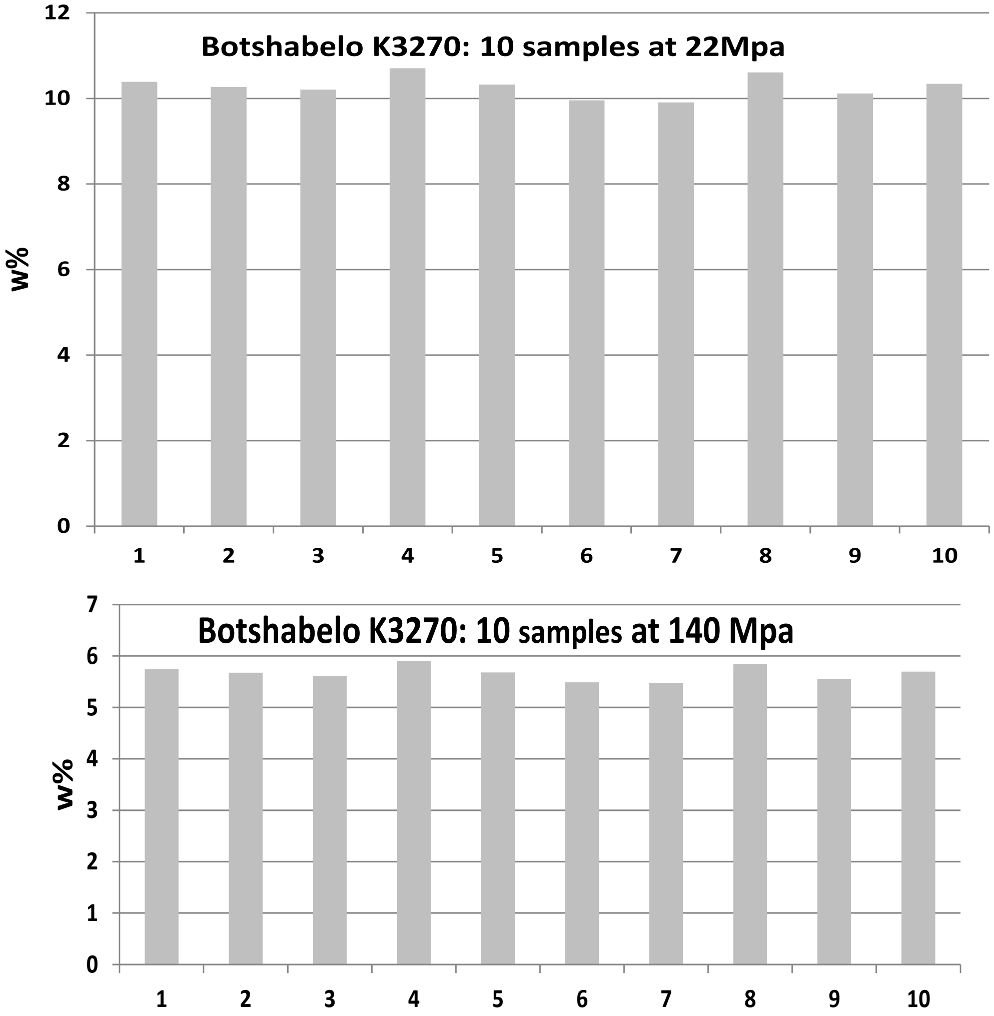


Figure 7.16: Water retention at 22 MPa and 140 MPa (2nd test pit).

Case 4. A less fortunate situation occurred at a sub-station in Northern South Africa. Normal foundation indicator tests were performed on samples from a test pit at the site. The tests indicated non-expansive soil and the foundations were designed accordingly. Shortly after completion serious heave damage occurred. Suction tests on samples of the soil concerned gave results as in Figure 7.17. The figure indicates that the variability of suction potential is very high. The COV for these 20 samples is 39. It appears that the common practice of relying on results from a single sample could lead to very unsound design.

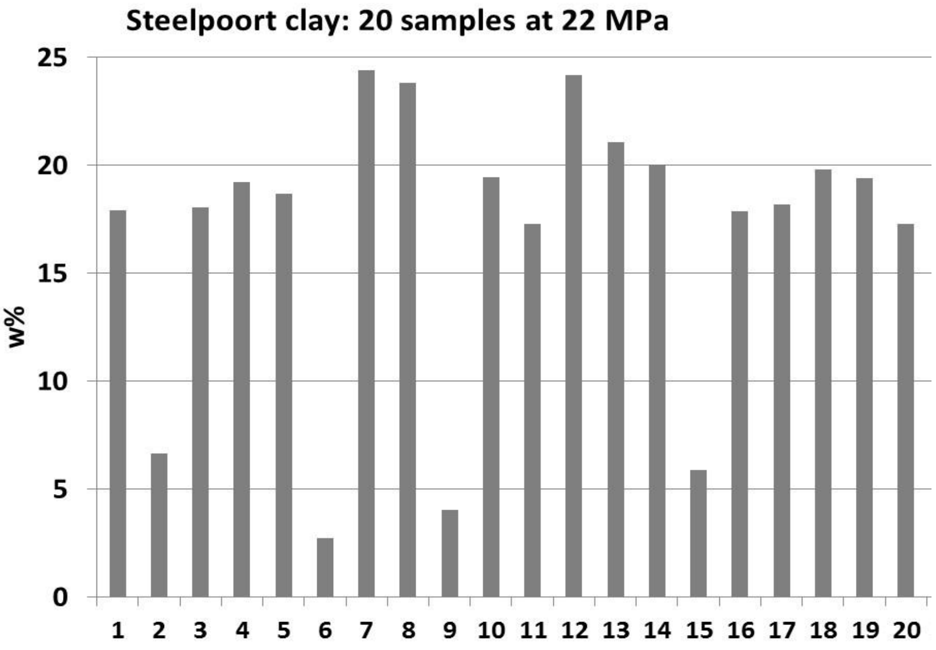


Figure 7.17: Variability of suction potential.

Samples of this clay were sent to seven reputable soils testing laboratories for foundation indicator tests. PI is the most commonly used property for indicating shrink/swell potential. Figure 7.18 shows values of PI from these seven laboratories. It is evident that the original tests on which the foundations were designed happened to be from a very unfortunately chosen sample. The resulting damage might have been avoided if the exceptionally large variability had been assessed and allowed for in the design process. It is doubtful whether RBD would have given satisfactory results using the estimated COV from (Phoon and Ching, 2013) as in section 7.7 above.

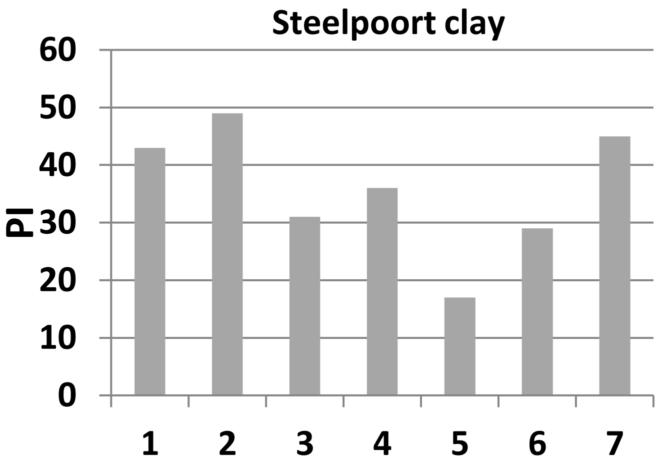


Figure 7.18: COV for PI of 7 laboratories.

In Figure 7.18, the COV for PI of 7 laboratories is 34, which is in good agreement with 39 from a considerably larger sample space of suction values. It is far more convenient to assess 20 samples by suction tests than to send multiple samples for duplicate testing.

Case 5. A house was built close to a test pit where laboratory results indicated no risk of heave. With the following rain the house suffered such severe heave damage that it had to be demolished before the roof was installed. Suction tests showed COV 16, very similar to the soil of Figures 7.19 and 7.20. The sample tested in the geotechnical investigation was again unrepresentative of the general ground conditions. Suction tests would have given warning of this and almost certainly have led to better foundation design.

7.7.2 General observations on variability

Cases 3 and 4 above represent opposite ends of the variability spectrum. Very few soils so far tested by this technique, have had COV smaller than 2 and none greater than 39. Values from a more typical soil are illustrated in Figures 7.19 and 7.20.

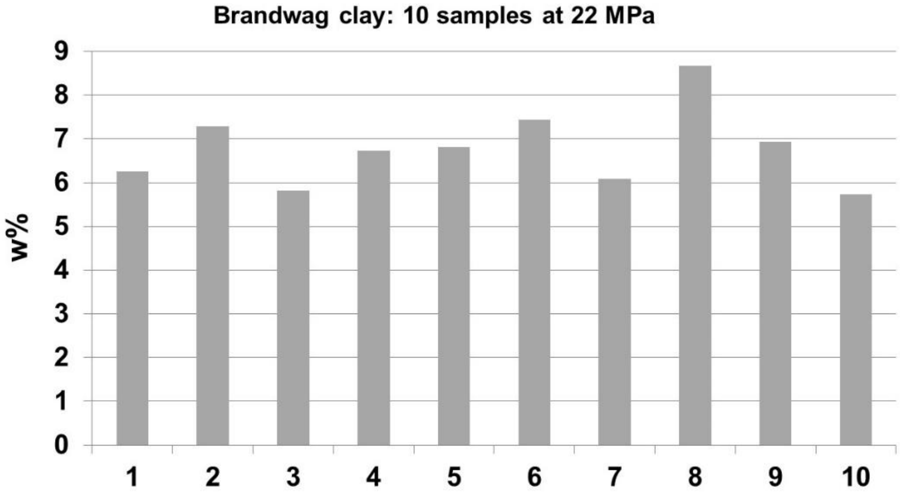


Figure 7.19: COV of 13 for 10 suction test samples.

Figure 7.19 shows COV of 13 for 10 suction test samples of typical clayey soil from Central South Africa. Figure 7.20 shows COV for PIs from 5 different laboratories as 17 – again close to the value from suction tests on a bigger sample space.

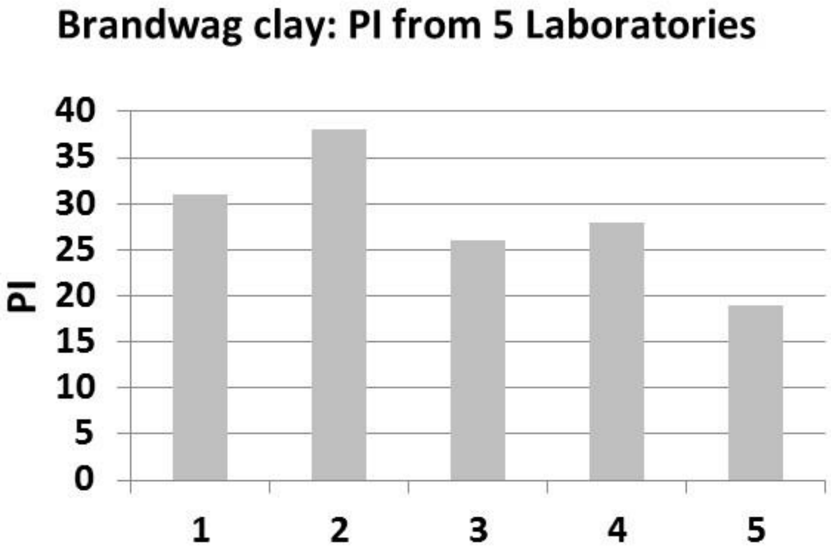


Figure 7.20: COV for PIs from 5 different laboratories.

This soil is from a building site in Central South Africa. Not all correlations with multiple laboratory PI tests are as close as this, as might be expected in view of the weaknesses noted in section 2 concerning the reliability of Atterberg limits for predicting heave. Figure 7.21 shows 8 samples from a roads project tested at 8 different testing laboratories (top) and also in 8 suction tests (below) – the commercial laboratory results show COV of 29 where the suction tests show COV of only 20. Both sets of results are, however, adequate to give warning of the danger of basing design on one set of test results.

In order to assess shrink/swell potential across a wide range of soils in a consistent and systematic manner two factors need to be considered: a standard suction at which to measure water retention, and a consistent approach to hysteresis

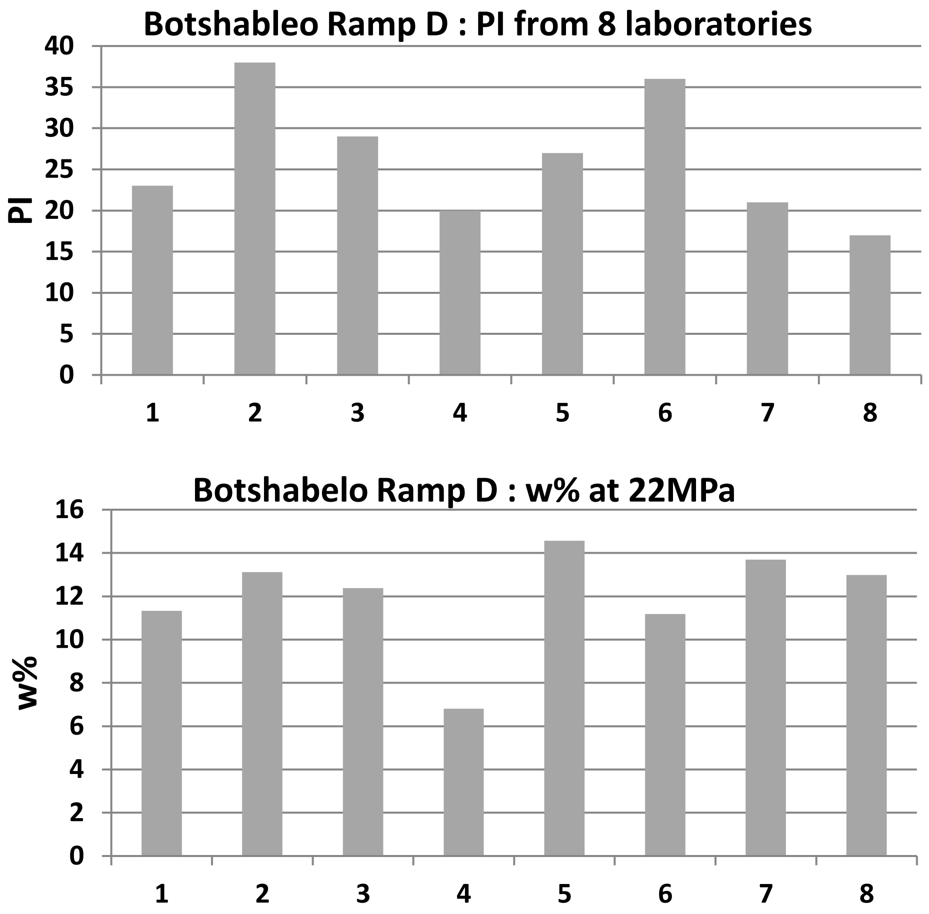


Figure 7.21: PI and water retention values for a clayey soil.

7.8 Considerations for assessment of shrink / swell potential

7.8.1 Suction value for measurement

Factors influencing the choice of standard suction value should include convenience, time taken to reach equilibrium, ease of attaining acceptable accuracy and cost. Stable water retention is achieved quickly at high suctions and slowly at low suctions but water content is low at high suctions and therefore more precision is required in weighing samples. Figure 7.22 shows plots of equilibrium water retention over the full suction range relevant to soil mechanics for 5 sets of 4 samples through one drying and wetting cycle. These are effectively soil water characteristic curves plotted with linear scales. The SWCC is commonly drawn with semi-log scales. As Blight (2013) pointed out, log scales give a much distorted view of reality and linear scales allow a better understanding of the true situation. From these plots it appears that at very high suctions (greater than about 150 MPa) all of the plots take on steeper gradients and converge, so that the spread of water content becomes inconveniently small. From about 20 MPa to 120 MPa the plots have only a small gradient and remain at a fairly constant spacing. From about 20 MPa to 0 MPa the curves rise at a very steep gradient, indicating that water content determination may be inconveniently sensitive to accuracy in suction measurement.

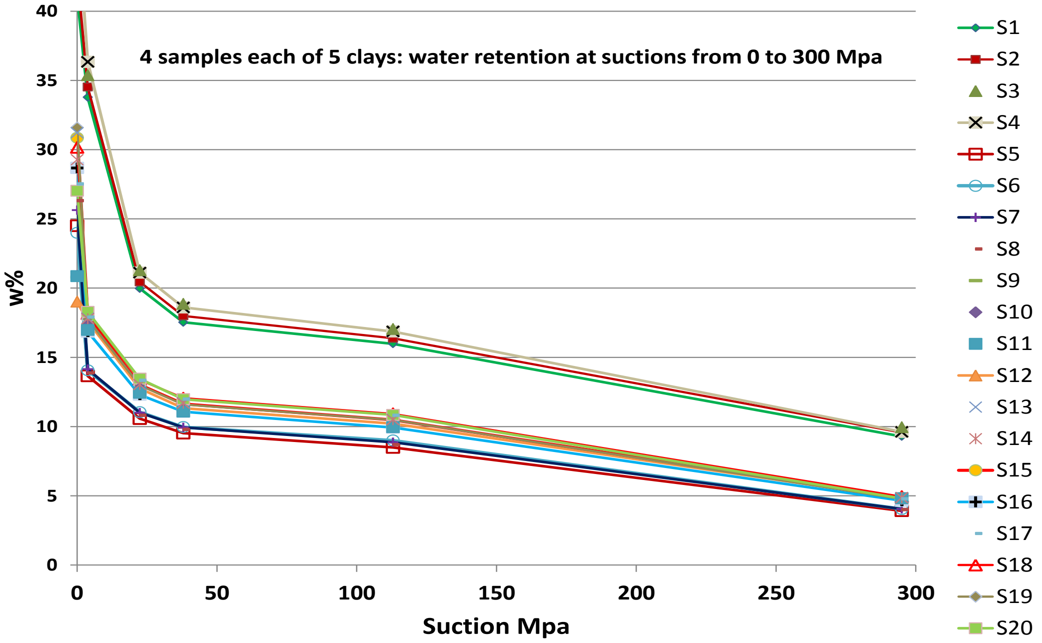


Figure 7.22: Water retention: suction for 5 clayey soils.

From these plots it appears that low suctions may be a poor choice for standardization from the points of view of both excessive testing time and sensitivity to suction. The ideal range appears to be in the region of 20 MPa to 100 MPa, where sensitivity to suction and testing time are both favorable. For equilibrating samples over saturated solutions of known suction it appears that NaCl would be a very good choice. It is cheap, readily available, non-toxic and needs no handling precautions. Its suction value is close to 38 MPa throughout the temperature range from 15˚C to 25˚C. This is also a very convenient suction for the climate chamber, since temperatures close to ambient require a relative humidity of only 75% for 38 MPa suction and this poses negligible risk of condensation problems. KCl would also appear to be suitable, with a saturated solution providing 22 MPa over the same range of temperature.

7.8.2 Hysteresis

To give comparable results across different samples the question of hysteresis needs to be considered. Figure 7.23 shows water retention for samples of 20 different soils at 22 MPa suction after the samples started wet and dry. The difference in retention varies from about 10% to 30%, with most fairly close to the average of 20%. For the purpose of assessing potential shrink/swell it might be preferable to start from the shrinkage limit of the soil or from the driest condition the soil is likely to experience in the lifespan of the structure concerned. In the sub-humid and semi-arid conditions where shrink/swell problems are usually most severe suction values can be very high during periods of drought. For convenience and uniformity, values used in this paper are from oven dry unless otherwise indicated.

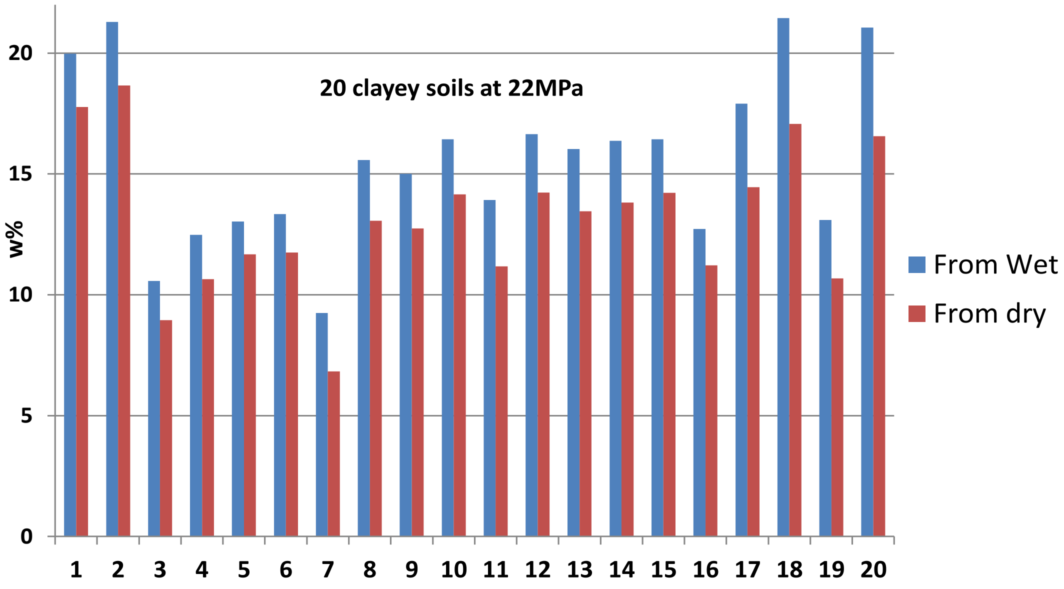


Figure 7.23: Water retention differences at 22 MPa due to hysteresis

7.9 Considerations for Assessment of Variability

Table 7.1 shows COVs for 13 soils at 3 suctions. Due to the excessive time taken, only 1 set was tested at very low suction. COV does not seem to be sensitive to suction value; all measured values are within 1.2 of the average, most are within 0.5. The most suitable range for evaluating shrink/swell potential (section 7.8.1) appears to be also suitable for COV. It should therefore be feasible to perform one test for both shrink/swell and variability. The table deals with the typical range of variability for Central South African clayey soils tested to date. About half of them show COV indicating that multiple sample testing is essential for accurate shrink/swell evaluation.

Table 7.1: COV: 10 samples of 13 soils.

Soil	0 MPa	22 MPa	38 MPa	180 MPa	Average
Vrede 6	10.8	10.6	10.0	-	10.5
Big Lump	-	1.9	1.5	1.7	1.7
Belcher 2	-	15.3	15.1	15.2	15.2
Lerato P 1	-	10.5	10.9	10.7	10.7
Lerato P 2	-	5.2	6.0	5.2	5.5
Fichardt P	-	5.0	5.8	5.6	5.5
Botsha R	-	19.4	18.9	22.0	20.1
Botsha B	-	2.8	2.6	3.3	2.9
Dersley	-	7.1	7.0	7.1	7.1
Cecelia 5 A	-	7.4	75	7.6	7.5
Cecelia 5 B	-	14.2	14.0	14.4	14.2
Brandwag	-	13.0	13.6	13.4	13.3

7.10 Considerations for Design

Variability of shrink/swell potential appears to hold out far greater risk for design than high shrink/swell potential alone. In the cases encountered in practice, some of which have been presented in section 6.1, where foundation indicator tests have pointed to high expansive potential and variability was low, foundations were designed for high swell and suffered little significant damage. Where high variability exists, but variability was not taken into account, substantial damage occurred. How then should design proceed for a soil with significant shrink/swell potential? It would be possible to do multiple tests and then design for the most unfavorable value found. This should be much safer than accepting the value from only one set of tests. The design might, however be over-conservative and unnecessarily expensive. It would appear that the only rational method, for which a reasonable assessment of acceptable risk can be made, is reliability based design. Small scale suction testing can give sample specific values relatively easily and there may be a good fit between RBD and these tests in such cases.

7.11 Testing Procedure

The following procedure, as shown in Figure 7.24, is suggested as providing a balance between speed, simplicity, ease of preparation, economy of apparatus and reliability of results.

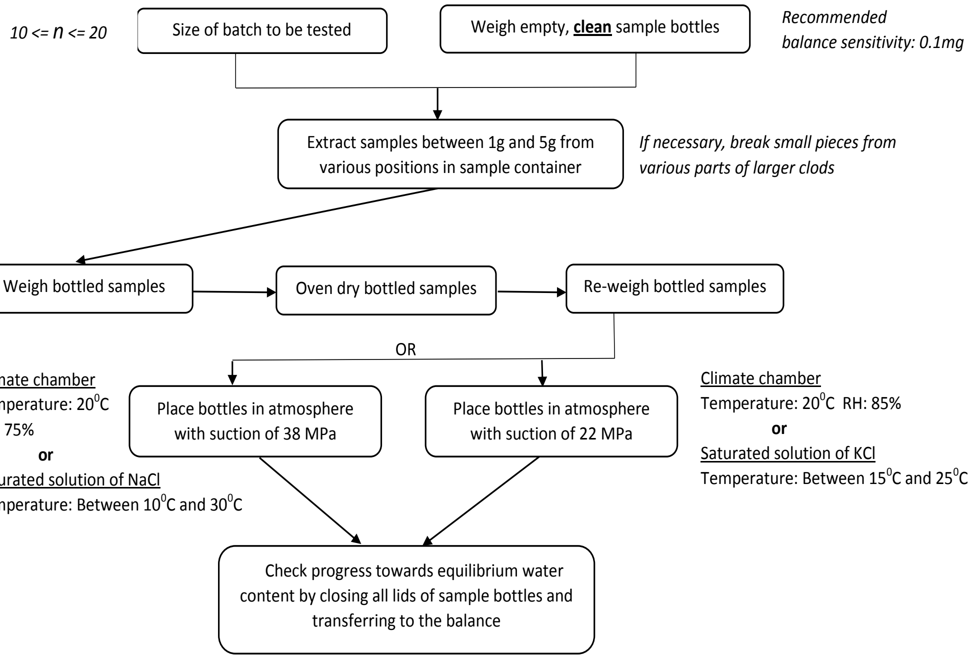


Figure 7.24: Testing procedure flow diagram.

Three days may be required to approach within 1% of equilibrium value, and 5 days may be required for effective equilibrium over saturated solutions. Equilibrium is reached in noticeably less time in a climate chamber. At 38 MPa, water retention below 4% corresponds with low swell potential; retention above 10% corresponds with high swell potential; retention above 13% corresponds with very high swell potential. At 22 MPa water retention below 5% corresponds to low swell potential; retention above 12% corresponds with high swell potential and above 16%

corresponds with very high swell potential. COV greater than 15 may indicate more troublesome shrink/swell problems than high expansive potential alone.

7.12 Conclusion

Moisture retention at an applied suction appears to be a good indicator of shrink/swell potential although it may not be an indicator of other soil properties such as shear strength. The testing technique described here, using very small samples subjected to easily-controlled suctions, may provide a more convenient and reliable means of assessing probable shrink/swell potential than current commercial methods. Very little sample preparation is involved and there is little scope for short-cuts which may adversely affect results. The soil micro-structure remains substantially intact. A large enough number of test samples can be used to gain insight into the variability of the soil concerned without greatly increasing the testing time or labour cost. A measure of the variability of individual soils can be of value in indicating cases where current testing procedures give inadequate warning of shrink/swell problems. Specific COV values for individual soils provided by this method could make RBD a very attractive tool for design in a field well known for widespread failures following traditional design procedures.

CHAPTER 8: SHORTCOMINGS IN THE ESTIMATION OF CLAY FRACTION BY HYDROMETER

8.1 Abstract

The estimation of clay fraction is important for predicting the engineering properties of a soil. SANS 3001 GR3 (SABS 2011) specifies a procedure for clay fraction determination using a hydrometer. It has long been suspected that there may be flaws in this approach. Some of the possible sources of error have been suggested, but little or no change has been made in the standard procedures for assessment of clay fraction in well over half a century. This paper deals with a microscopic examination of some typical South African clayey soils to assess the adequacy of dispersion and possible consequences for clay fraction determination in currently specified hydrometer procedures. Clays are examined both with and without dispersant, and with and without labelling of clay minerals using an exchangeable cation dye.

Keywords: Hydrometer analysis, clay fraction, dispersion of clays, de-flocculation.

8.2 Introduction

An estimation of the clay fraction of a soil is required for a number of soil evaluations, including common methods of assessing heave potential relating to foundation design. Van der Merwe's method (1964) uses the Plasticity Index (PI) and clay fraction. Skempton's "activity", is defined as PI / clay fraction (Skempton, 1953). The method of estimating clay fraction by hydrometer as specified in the South African standard SANS 3001 GR3 (SANS 2011) is very similar to that specified in Britain, America, Australia and many other countries. It is, however, somewhat dubious in its efficiency. Savage suggested that the hydrometer method may be doubtful due to four factors (Savage, 2007): (i) Stokes law assumes all particles to be spherical, while clays are flaky. (ii) De-flocculation of many clays is seldom fully completed at the time of testing. (iii) Clay particles are partially carried down by the larger particles. (iv) A relative density of 2.65 is assumed for all particles, which may not be true. Savage (2007) proposed a method of estimating clay fraction indirectly by using Skempton's activity equation. Unfortunately, there seems to be no clear pattern of correlation between hydrometer results and Savage's method (2007). Savage did not give examples and the examination of samples by the Central University of Technology (CUT) Soil Mechanics Research Group revealed no clear pattern of correlation (some values higher, some lower than the hydrometer). There appears to be no way of telling which gives the better estimate or what the likely margins of error may be. The method does not appear to have found wide acceptance. Progress has been made on Savage's first point, the question of non- sphericity of particles. It has been addressed by laser scattering techniques for particle suspensions. (e.g. Konert and Vandenberghe, 1997; McCave *et al.*, 1986; Ma *et al.*, 2000). This technique has enabled an allowance to be made for particle shape and has generally led to a small but significant increase in clay fraction estimation. Such an allowance is not specified in SANS 3001 GR3. Savage's fourth point seems to have drawn little attention, since almost all non-organic soil components have densities reasonably close to 2.7, and the likely error due to this factor is probably quite small. His remaining two points concern dispersion and obviously merit attention.

Research currently being done on the theoretical aspects of dispersion of clay particles suggests that the problem is far from well understood e.g. Robinet *et al. (*2011) and it remains very difficult to assess most aspects of dispersion for any specific clay and solute system. Experimental research on de-flocculation/dispersion using non-traditional de-flocculants appears to be currently concentrated on ceramics, e.g. Al-Lami (2008). Such dispersants produce functional groups acting as spacers between clay particles and may be too expensive for routine soils testing at this stage of development. Work on deflocculation/dispersion relevant to soil mechanics continues to use methods and dispersants which have been in use for many years (e.g. Rodriguez *et al.*, 2011; Rolfe *et al.*, 1960). Attempts to assess the magnitude of error likely to be involved in incomplete dispersion continue to use the hydrometer itself as the instrument of investigation (Nettleship *et al.*, 1997; Rodriguez *et al.*, 2011). This paper is primarily concerned with Savage's second point, the dispersion of clay particles.

His third point, clay being carried down with larger particles, follows from this as a matter of course.

8.3 Theoretical background to dispersion of clays

The behaviour of dispersants is complex and appears to be still imperfectly understood. This outline synthesises information from Das (2008); Zschimmer and Schwartz (2014); Nettleship *et al.* (1997) and Robinet *et al.* (2011) Clay particles carry charges which leave their inner structure negatively charged and tend to leave the charge of their outer edges pH dependent. When active clay soils are mixed with water two things tend to happen. Water molecules, which are polar (their atomic structure leaves one side positively charged and the other side negatively charged, while remaining neutral as a whole) surround cations (positively charged metal ions) in the soil. When coated with water the cations become mobile. They are strongly attracted by the negative charges in the interior of some types of clay minerals and penetrate between the tetrahedral and octahedral sheets of these clays, forcing the sheets apart. This is the reason why some clays can increase in volume powerfully when wetted. Secondly, the charged outer edges of the clay particles attract oppositely charged ions which form a diffuse layer whose concentration diminishes with distance from the clay surface. Multi-valent ions provide multiple electric charges and relatively few of them need to congregate around a clay particle to balance the charge on the surface of the clay. The resulting field surrounding the clay has marked peaks at the ions and troughs between them. This allows adjacent clay particles to maintain mutual electrical attraction by fitting troughs on one particle to peaks on another.

In order to assess clays by their rate of precipitation, as in the pipette and hydrometer methods, it is necessary to disperse the particles of clay into the water through which they precipitate. Mechanical agitation is essential for this, but is not sufficient on its own, chemical dispersion is needed to break the bonds of electrical attraction holding assemblages of clay particles together.

Dispersants work in three ways. The first is to replace multi-valent ions at the clay surface by mono-valent ions. When an individual clay particle is surrounded by sufficient monovalent ions to render it electro-neutral, the field surrounding it is relatively uniform; clay particles in such a state cannot attract each other by fitting electrostatic peaks to troughs. The second way is by reacting with multi-valent ions to form chemical complexes, making them unavailable for attraction to clay surfaces. The third is by forming functional groups which act as spacers between the clay particles, effectively preventing them from approaching each other.

The combined action of clay particles, cations and dispersing agents is complex. Above a certain concentration of dispersant, the diffused double layer starts to become thinner, repulsion between the particles reduces, and at higher concentration turns to attraction, allowing flocculation to occur. Sodium Hexametaphosphate is one of the most popular dispersing agents. It is specified in the standards for assessing clay fraction in Britain, America, Canada, Australia, Japan and other countries. It provides mono-valent sodium ions to coat the clay surface as well as phosphate groups to form complexes with multi-valent cations. Sodium Carbonate may be added to increase alkalinity; this has been found to improve the dispersive efficiency slightly in some circumstances (Rolfe *et al.*, 1960) and to extend the useful life of the dispersant (Nettleship *et al.*, 1997). A mixture of Sodium Hexametaphosphate and Sodium Carbonate is specified as the dispersant for the hydrometer procedure of SANS 3001 GR3.

8.4 Background and Aims of the Study

Investigations are being undertaken by the CUT Soil Mechanics Research Group seeking solutions to the problem of large numbers of failures in government subsidy houses due to heaving foundations. It appears that in some of the failures investigated, the geotechnical investigation had given misleading indications of clay fraction. In one case hydrometer analysis indicated less than ten percent clay on a site where notable shrinkage cracks in the ground surface suggested at least 20% clay content. Since hydrometer analysis is normal for almost all construction projects in South Africa such shortcomings in the method may be relevant for a wide range of situations. The aim of this study was to gain an insight into the reliability of the clay fraction indicated by the hydrometer for a range of clays typical of those found in construction projects in South Africa. It is common practice among researchers to examine clays with the electron microscope. This has the advantage of very high magnification. Preparation involves samples being treated by techniques such as drying and coating with gold (Nettleship *et al.*, 1997). This does not replicate conditions in the hydrometer. A series of exploratory tests were conducted at the geotechnical research laboratory of CUT to examine the possibility of using an optical microscope / digital camera combination to investigate the efficiency of the dispersion of clays using method GR3. The results suggested that dispersion was not always satisfactory. Many clay particles appeared to remain as conglomerations, others remained adhering to silt and sand particles.

The procedures used by soil science laboratories differ somewhat from those for engineering materials. Previous cooperation with the Soil Science Department of the University of the Free State had sometimes found higher clay fractions indicated by their procedures. It was arranged for six samples to be prepared by the UFS soil science laboratory using their normal method. The dispersant is 50g per litre Sodium Hexametaphosphate solution, the amount applied depends on soil type; sonification in a dismembrator is specified for clay soils, mechanical dispersion is of shorter duration but at a higher speed than SANS 3001 GR3. The samples were examined to determine whether dispersion by this treatment was visibly more effective than SANS 3001 GR3 for these six soils.

8.5 Equipment, Materials and their Usage in the Investigation

8.5.1 Microscope and camera

An optical microscope with objectives of 10x, 40x, 60x and 100x was equipped with a digital camera with resolution of 9 mega pixels. Magnification resulting from the combined effects of the microscope’s lenses and the camera was assessed by measurements on a 100 lines-per-millimetre diffraction grating. A drop of sample prepared for hydrometer analysis was placed on a microscope slide, covered with a cover-slip and photographed at various magnifications. Photographs were taken at various locations on the slide. Most of the photographs in this investigation were taken using the microscope’s 40x objective since more powerful lenses give a very small depth of focus.

8.5.2 Magnification

The combined optical and digital magnification can be defined in different ways. The image on the computer screen used to view the images showed lines spaced at 10 microns on the diffraction grating spaced at 40mm on the screen when using the 40x objective. This implies a magnification of 4000 times. Alternatively, the 10 micron spacing on the diffraction grating corresponds to 150 pixels on the photographs produced by the camera using the same objective. The most convenient way of indicating magnification is by incorporating a reference object of known size. All of the photographs below have a rectangle superimposed to indicate the scale. The length of each rectangle represents 30 microns and the breadth represents 2 microns.

8.5.3 Variations in procedure

Samples were also prepared employing variations to the normal procedures in order to examine the influence of time of soaking in dispersant, time of agitation, concentration of dispersant and volume of dispersant used. Method GR3 specifies only minimum times of soaking and agitation. All of the times involved are within these specifications, and this part of the investigation served only to verify whether this aspect of the specification is adequate. Examination of the concentration of dispersant was prompted by the finding of a difference in hydrometer yield for certain clays using the Japanese and American standards, which specify different concentrations of Sodium Hexametaphosphate (Mishra *et al.*, 2011).

8.5.4 Methylene blue

In addition, samples were treated with methylene blue (MB) with the aim of labelling clay particles for positive identification. Methylene blue ($C_{16}H_{18}$ N_3SCl) is an effective indicator of clay; it readily exchanges places with cations in the clay mineral structure, the amount depending on the cation exchange capacity (CEC) and specific surface area (SSA) of the clay minerals (Turoz and Tosun, 2012). Active clays like montmorillonite have high CEC and SSA and readily take in methylene blue. When MB is available in large concentrations such clays rapidly become totally opaque and appear in photographs as dark blobs in which no structure can be seen. Inactive clay minerals like Kaolinite have low CEC and SSA and show little colouring until high CEC / SSA fractions present are already deeply stained. Progressive addition of small amounts of dye can therefore give an indication of the types of clay mineral present in a sample and also help to establish whether the clay-size particles which can be seen adhering to silt and sand particles are, in fact composed of clay minerals. Any additive to the soil solution which affects the cation balance will inevitably influence the effectiveness of the dispersant. Only small quantities of methylene blue were therefore added to the dispersed samples. It could be expected that silt and sand would not be coloured, high CEC / high SSA clays (e.g. montmorillonite) would be coloured after adding very little dye, whereas low CEC / low SSA clays (e.g. Kaolinite) would be coloured only after the addition of considerably more dye.

8.6 Theoretical Considerations and Strengths/Weaknesses of the Method Employed

Soil mechanics and soil science generally consider all particles of 2 microns and smaller to be “clay-size” particles, and those from 2 microns to 60 microns (or some other arbitrary figure of this order) to be of silt-size. But particles and agglomerations of clay minerals typically range from about 0.1 micron to slightly more than 2 microns; non–clay particles typically range from about 1 micron upwards (Robinet *et al.*, 2011). Some clays, e.g. kaolinite and halloysite may have particles considerably larger than 2 microns as can be seen in electron micrographs by Buhmann and Kirsten (1991). There is therefore a range where size classification may not correspond with mineral classification. Certain important aspects of soil behaviour (e.g. volume change) depend on clay mineral content, while hydrometer and pipette analysis attempt to establish only particle sizes, not mineral content. Many of the individual particles observed were in this ambiguous range of 1 to 2 microns raising the question of whether they are clay particles which need to be dispersed, or silt particles, which do not flocculate and should not need dispersion.

The magnifications possible with the optical microscope and camera combination used in this investigation are probably sufficient to distinguish most of the range typical for clay through silt to sand, but not adequate to measure the smallest particles in this range. Since all samples remained in aqueous suspension, all of the smaller individual particles were subject to Brownian motion. At the highest magnification (100x objective – 37.5 pixels per micron, 10 000x magnification on the computer screen), particles at the lower end of the clay-size range could be distinguished in many of the samples, but their Brownian motion hindered observation or measurement since they suddenly appear in the focal plane and disappear as they move away from the focal plane. Photographing them was not very successful, possibly because the exposure time of the camera/computer combination was too long. Many small particles were visible and could be photographed where they formed part of large agglomerations or were attached to silt or sand particles.

8.7 General Considerations

The following considerations in terms of the microscopic investigation should be noted:

(a) Those samples extracted for microscopic investigation at the UFS laboratory were taken by pipette after a settling time calculated to give only silt and clay sized particles at the depth of extraction (larger particles having settled below this level). The largest particle sizes measured were of the order 50 microns, suggesting that the sample was, indeed, restricted to clay and silt-sized particles. Samples prepared in the CUT laboratory were taken immediately after agitation and some samples contained particles considerably larger than 50 microns, allowing examination of sand grains as well as silt.

(b) The cover slip over the sample was supported by the largest particles and consequently a depth of about 50 microns was filled with suspension for the UFS samples and up to about 100 microns for the CUT samples. Depth of sharp focus at high magnification is far smaller than this and consequently photographs necessarily have most of their field out of focus.

(c) Since clay-sizes range from 2 microns downwards, the concentration of suspension specified in the hydrometer method allows too many clay particles in a depth of 50 microns for convenient optical differentiation. This made dilution of the hydrometer samples necessary. The majority of samples were diluted with three times their own volume of de-ionised water. This dilution was arbitrarily chosen and was considered adequate for this purely qualitative investigation.

(d) The gap of approximately 50 to 100 microns between slide and cover slip allows evaporation of the suspension’s water around the edges. It may be possible to seal around the edge of the cover slip and prevent evaporation, but it was found that the movement of water caused by evaporation was helpful in distinguishing between clay-size particles which were dispersed and free-floating, from those which were attached to silt or sand particles or formed agglomerations with other clay particles. This consideration results in a very limited time available for the observation of each slide.

(e) When samples dry out they conglomerate, making it difficult to draw conclusions about the behaviour of the clay in conditions relevant to the pipette and hydrometer tests. Only observations in suspension conditions were considered in this investigation.

8.8 Samples Used in the Investigation

Samples of six widely different clay soils (from Free State, Northern Cape, Western Cape and Limpopo) were mechanically agitated as in procedure GR3 but without first soaking in dispersant. Samples of the same soils were prepared with both dispersant and mechanical agitation at CUT as per SANS 3001 GR3 and at the Soil Science laboratory of UFS using standard soil science procedures. From the six soils two were selected as showing typical features and illustrating the general effectiveness of the investigation's procedures. One soil appears to show fair dispersion, the other very inadequate dispersion. The first of these samples is shown in Figures 8.1 to 8.5, the second in Figures 8.6 to 8.13. They provide a reference frame and show widely different clays with and without dispersant both with and without methylene blue. Features of some of the other soils are shown in the remainder of the photographs.

8.9 Observations

Figure 8.1 is of an olive-grey residual clay from a proposed housing development in Bloemfontein. Tests at the UFS Soil Science Laboratory gave LL 72, PI 26 and clay fraction 41% (by both hydrometer and pipette methods).

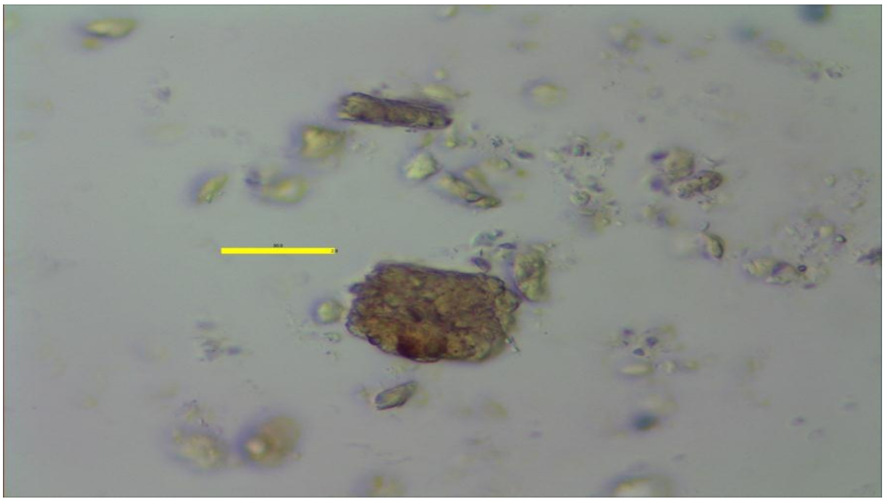


Figure 8.1: Olive grey residual clay.

Figure 8.1 shows this clay not treated with dispersant but mechanically stirred as specified in SANS 3001 GR3 Comparison with the 30 micron by 2-micron rectangle suggests that there are particles of about 2 microns attached to several of the silt particles. Some clay-size particles appear to be dispersed in suspension. Note the faint pinkish cloudy patches covering a considerable part of the field of view (most of which is not within the lens's range of sharp focus).

Figure 8.2 shows the same sample (without dispersant) after the addition of 3mg of methylene blue per 1g of soil. Many of the particles of 2 microns and a little larger adhering to the silt particles show a faint blue outline. Much of the pinkish cloudy area has taken in methylene blue and appears to be composed of extremely small clay particles. Little of the field of view is in focus but some deeply stained individual particles smaller than 1 micron are discernible.

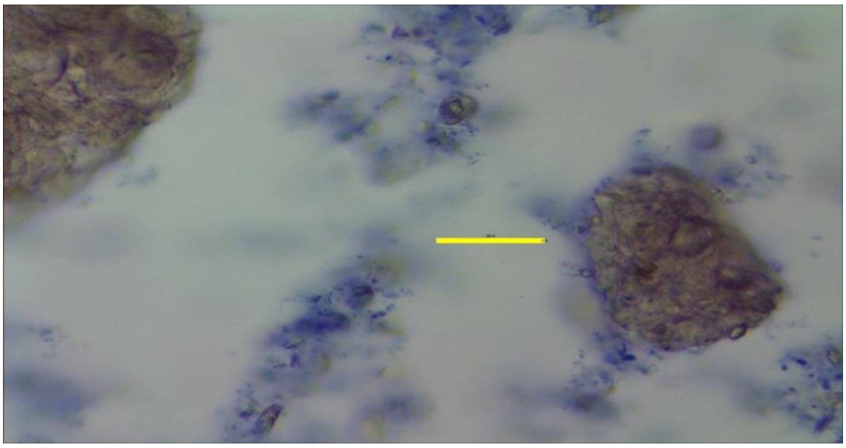


Figure 8.2: Olive grey residual clay with 3 mg/g methylene blue.

Figure 8.3 shows the same sample (without dispersant) after the addition of a further 5mg/g. of MB dye. None of the pinkish cloud remains, it appears to have taken in dye and can be deduced to be an aggregation of very small clay particles. Almost all of the silt particles appear to be completely covered with clay. The assembly moves as a flexible unit in the currents caused by evaporation around the edge of the cover slip. No individual clay particles can be seen which are not part of an aggregation.

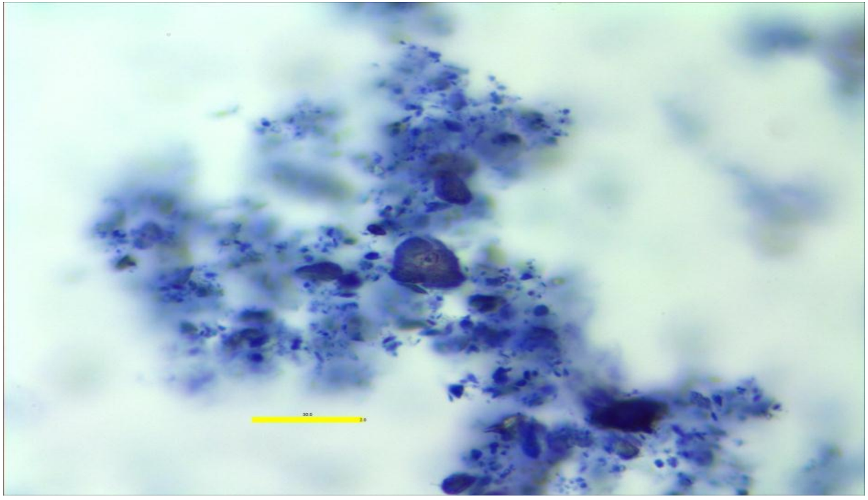


Figure 8.3: Olive grey residual clay with 8mg/g methylene blue.

It appears that a large number of very small clay particles bind considerable numbers of various kinds of particles into associations. Currents caused by evaporation of the suspension's water show that these associations are flexible but strongly tied together. Figure 8.4 shows the same soil after treatment with dispersant as specified in SANS 3001 GR3. Comparison with Figure 8.1 shows a very large increase in clay-size particles dispersed throughout the water. There are, however, some clay-size particles adhering to silt particles and a number of small agglomerations of clay-size particles with no visible silt core.

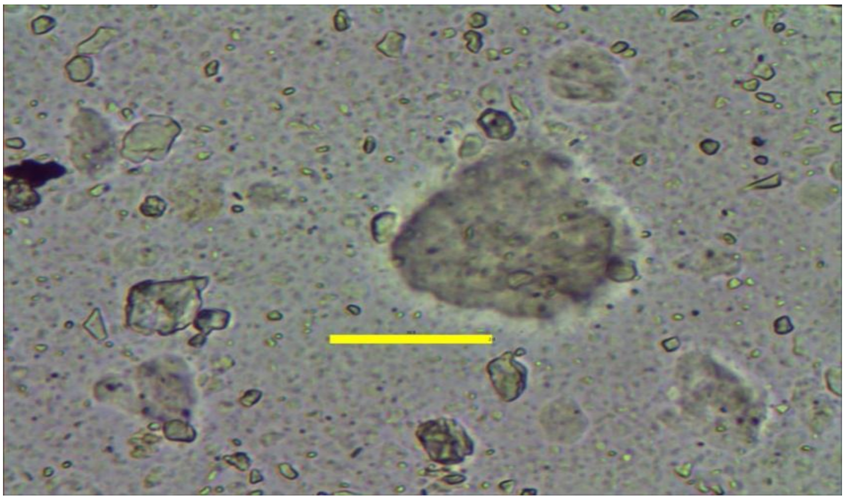


Figure 8.4: Olive grey residual clay treated with dispersant.

Figure 8.5 shows this same dispersant–treated sample after the addition of methylene blue. While this sample shows good dispersion compared to the untreated state, it is apparent that dispersion in not complete.

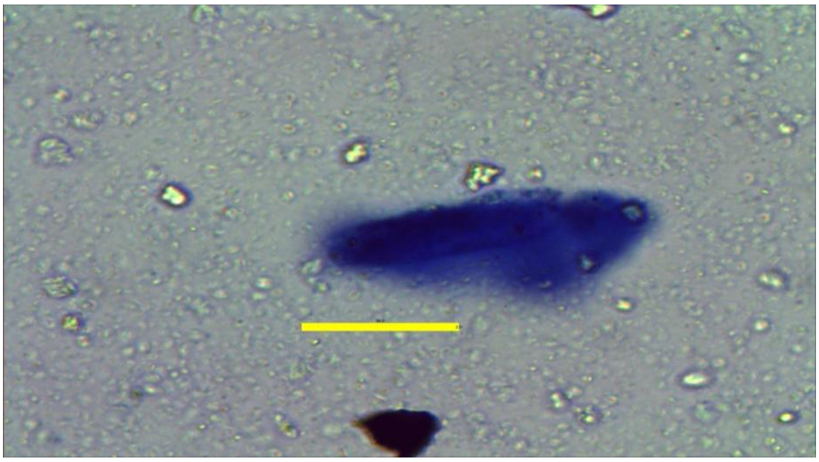


Figure 8.5: Olive grey residual clay with dispersant and methylene blue.

The majority of dispersed particles appear to be about 2 microns in size, but small deeply stained particles of less than one micron can also be seen, it is difficult to discern whether they are free or attached to larger clay-size particles. The agglomerations of clay-size particles, the clay-coated silt particles and the agglomeration of very fine clay will probably not precipitate at a rate which will ensure their contribution to the clay fraction being recorded by the hydrometer. The small agglomerations of clay-size particles show little if any staining, suggesting very low CEC. The silt particle at bottom centre appears to be coated with very small, high CEC particles which are very darkly stained. The deeply stained agglomeration in the centre is about 50 microns in length and 25 microns in width. It appears to be made of very small, high CEC particles and it seems to engulf several silt and clay-size particles. Such agglomerations were not uncommon in this sample, but probably not common enough to ensure a meaningless clay fraction from hydrometer analysis.

Figure 8.6 is of a red-brown soil from Limpopo Province which has a history of giving variable results in soil tests and causing difficulties in construction. Treatment was only mechanical stirring of the raw soil without dispersant. Commercial laboratory results for samples sent by CUT as part of a parallel testing programme ranged between 17% and 56% for clay fraction, and between 31 and 43 for PI. Many of the silt-sized particles appear to have clay-sized particles adhering to them.

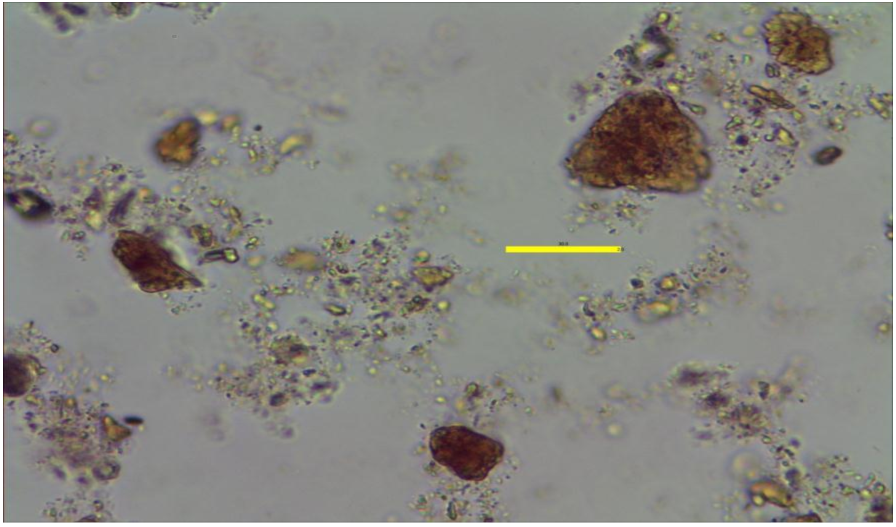


Figure 8.6: Red Brown soil from Limpopo after mechanical stirring

A faint cloudy pinkish haze, as noted in Figure 8.1, is again evident. Most of the silt particles appear to be clustered together in loose associations. As in the case of the olive grey Bloemfontein soil there are shadowy pinkish bands associated with the distinctly visible particles. There are also many clay-size particles adhering to most of the silt-size particles.

In Figure 8.7 a large grain of silt appears to be mostly covered with clay-size particles. Part of the grain, however, is completely clean and free from clay coating. It is possible that it was struck by one of the paddles of the mechanical stirrer and some of the coating was torn away.

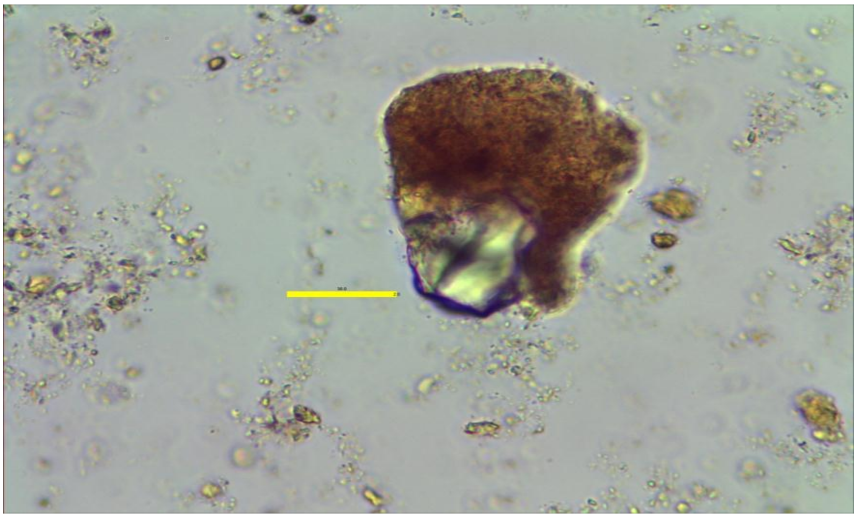


Figure 8.7: Large grain of silt of red brown Limpopo soil.

The coating of the lower right area seems to have come loose from the large particle but remains attached to the clay coating above. The pinkish haze and loose groupings of silt particles are again evident.

Figure 8.8 shows the same sample after addition of 3mg/g of methylene blue. A few small blue spots are visible on the larger silt-sized particles but the majority of particles of about 2 micron remain un-stained. A clearly visible cloud of very small blue-stained particles has largely replaced the faint pink cloud, suggesting that the cloud consists of very small and possibly translucent clay particles with high CEC / SSA.

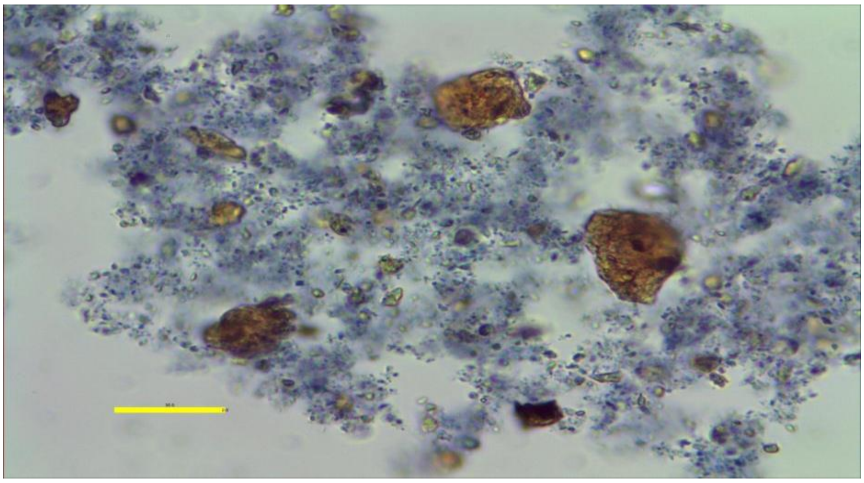


Figure 8.8: Red brown soil from Limpopo with 3mg/g methylene blue.

Most of the silt grains which appear to be covered with clay-size particles have scattered spots of dark blue stain suggesting a few high CEC clay particles among many low CEC particles.

Figures 8.9 and 8.10 show the same sample after the addition of a further 3 mg/g. of methylene blue. Figure 8.9 shows a grain of silt, apparently covered with low CEC clay almost completely enveloped by high CEC clay particles.

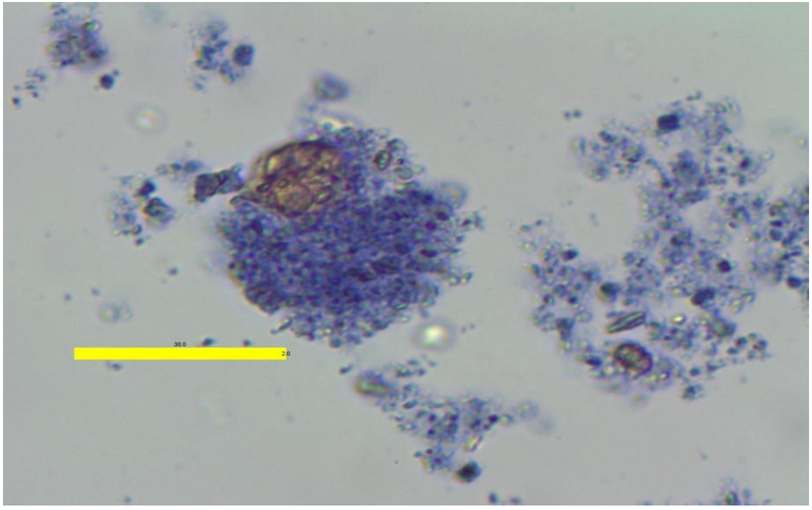


Figure 8.9: Red brown Limpopo soil with 6mg/g of methylene blue.

A number of small clay particles appear to be dispersed into the surrounding water, but the majority are associated with aggregations. The dense mass of deeply stained clay particles appears to almost completely engulf a silt particle covered with barely-stained clay-size particles. In Figure 8.10 a large agglomeration of clay appears to contain several small silt particles of various types.

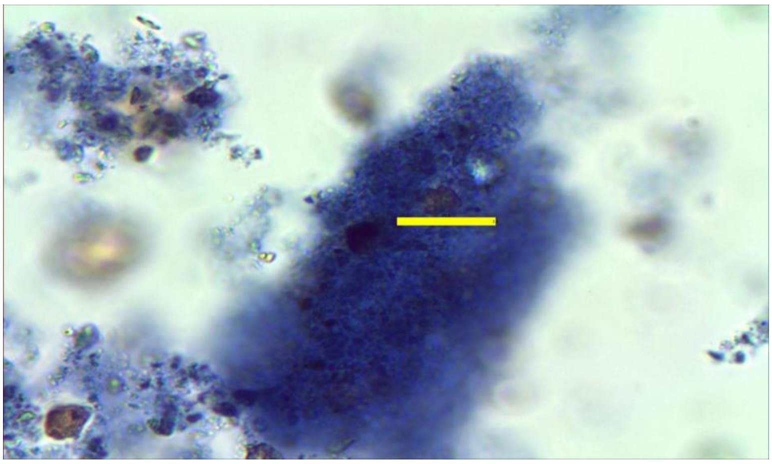


Figure 8.10: Large agglomeration of clay of red brown Limpopo.

The large blue structure is more than 150 microns by 50 microns in size. Within this structure, silt particles can be distinguished. Much of the agglomeration appears to consist of clay particles of about 1 micron or smaller. The agglomeration slightly above and left of centre appears to consist almost entirely of a different species of clay particles of about 2 microns which are very lightly stained. Very few soil particles are visible which are not part of an agglomeration. Figure 8.11 shows a sample of the same soil treated with dispersant.

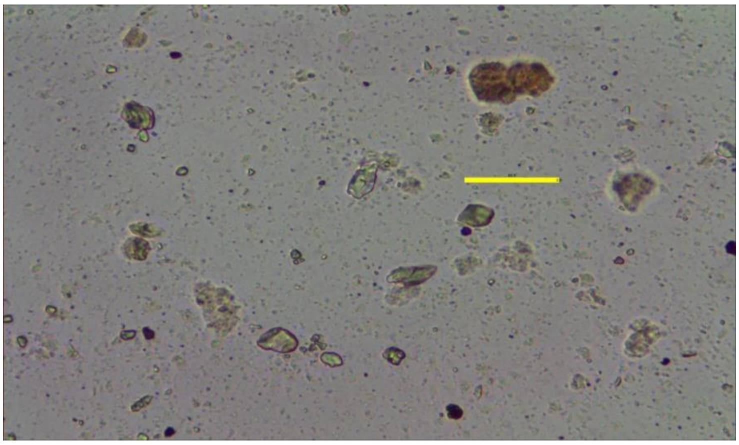


Figure 8.11: Red brown Limpopo soil after treatment with dispersant.

Many small clay particles are dispersed, and compared to Figures 8.6 and 8.7 the groupings of silt particles have largely disappeared. Yet many of the slit particles remain totally or partially coated with clay-size particles and many agglomerations of two or more clay-size particles can be seen. There are also cloudy pinkish areas similar to those in Figure 8.1.

Figures 8.12 and 8.13 shows the same soil after addition of methylene blue. In comparison with Figures 8.9 and 8.10, where no dispersant was used, dispersion is clearly improved, but most of the silt particles are seen to be covered with clay and the faint pinkish clouds again appear to be revealing themselves as very fine clay particles which are not well dispersed and may settle in the hydrometer as silt-size aggregates rather than as individual clay particles.

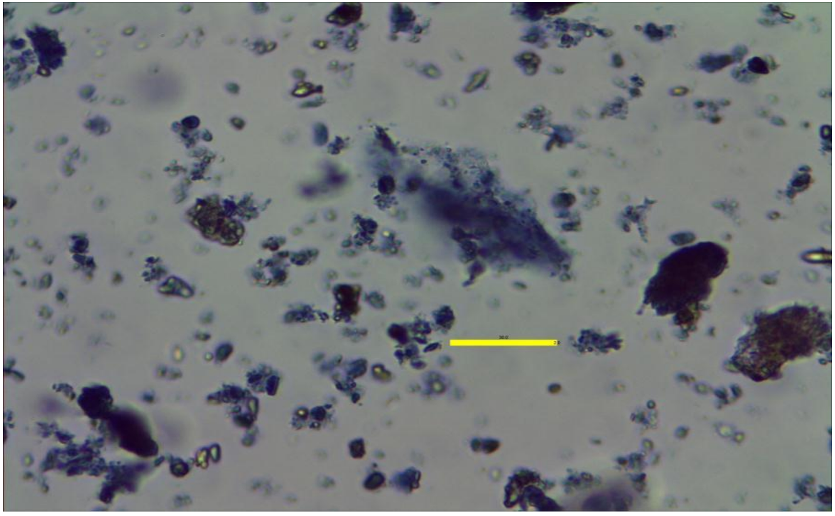


Figure 8.12: Red brown Limpopo soil with addition of methylene blue.

Figure 8.13 shows an exceptionally large agglomeration of small clay particles engulfing several silt particles and numerous 1 to 2-micron clay particles against a background of well-dispersed clay particles. This sand-size grouping of silt and clay is unlikely to settle at the rate expected of clay; nor are the numerous smaller aggregations. Nettleship *et al.* (1997) came to the conclusion that their anomalous observations of settlement in the hydrometer might be explained by agglomeration taking place while particles were settling during the test. It seems more likely that this could be the case here than that such an agglomeration could have survived 15 minutes of stirring at 1500 rpm after prolonged soaking in dispersant.

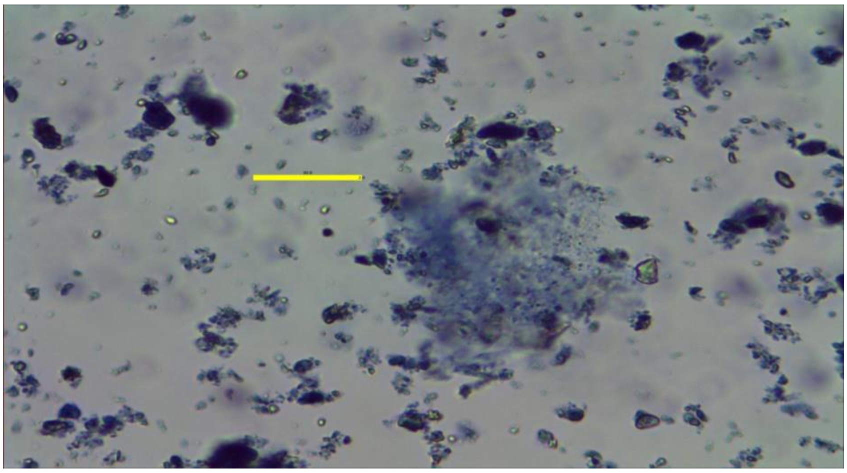


Figure 8.13: Red brown Limpopo soil indicating large agglomeration

The amount of clay which is obviously not dispersed in Figures 8.12 and 8.13 suggests that it is very unlikely that the hydrometer will give a reliable estimate of the true clay fraction of this soil. Figure 8.14 shows a soil from a housing project in the Northern Cape. The figure shows the soil after treatment with dispersant and mechanical agitation at UFS Soil Science laboratory. The majority of silt particles remain covered with clay-size particles. Hydrometer analysis had shown the clay fraction for almost all of the samples from this site to be very low. This led to a low value of Van der Merwe's (1964) predicted heave being accepted for design. Heave damage did, however, occur on the project.

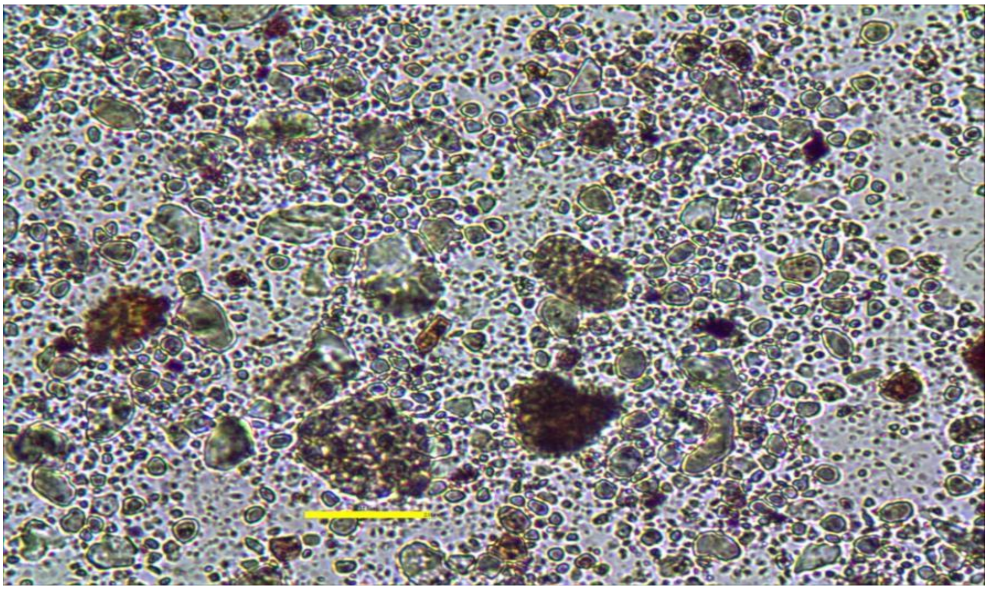


Figure 8.14: Soil from subsidy housing project in the Northern Cape.

A considerable fraction of the clay-size particles visible in Figure 8.14 are attached to silt particles of various sizes. It is not clear whether all the agglomerations of clay-size particles have a silt core, but it appears that much of the clay in this sample has not been dispersed. Hydrometer analysis could underestimate the clay content quite drastically. This might explain the unexpected damage which occurred at the housing project.

Another sample was prepared according to SANS 3001 GR3 (soaking in dispersant and mechanical agitation) at the CUT laboratory. This sample was taken a few metres from where a house became structurally unsound due to heave and was demolished before the roof was installed. Soil apparently identical to this from a nearby test pit was assessed by hydrometer as containing only 6% clay. Figures 8.15 and 8.16 are the sample from the CUT laboratory. In Figure

8.15 many of the visible clay-sized particles are attached to silt particles. In the figure many apparently clean silt particles can be seen as can many dispersed particles smaller than 2 microns. The larger particles are all covered with clay size particles.

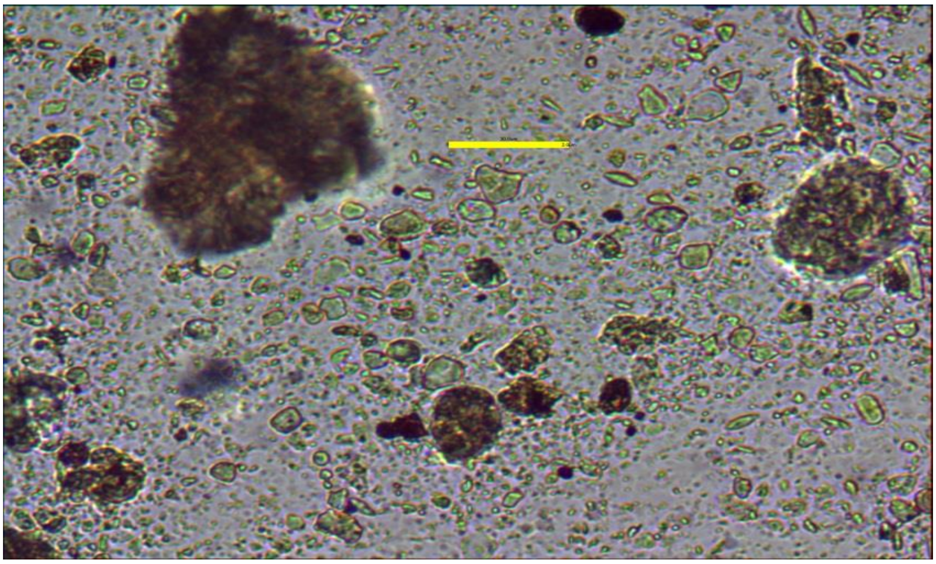


Figure 8.15: Sample of soil from Northern Cape housing project.

The same sample is shown in Figure 8.16. A large number of clay size particles form an agglomeration around a thin, thread-like structure – possibly a root-hair, while towards the top right many similar size particles appear to be well dispersed. Considerably more clay is undispersed than is dispersed. This would suggest the likelihood of a misleading estimate of clay fraction by the hydrometer method.

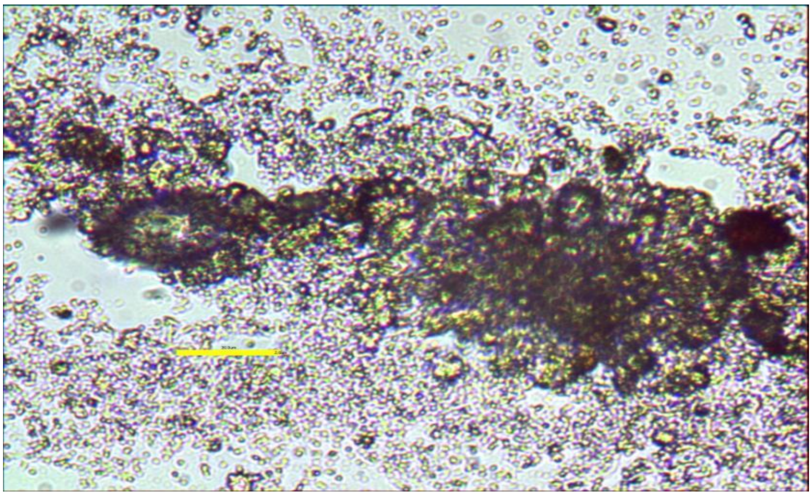


Figure 8.16: Agglomeration of large number of clay sized particles.

Figure 8.17 shows a low-activity kaolinitic soil described as “light yellow silty clay” from the Western Cape after treatment with dispersant and mechanical stirring to the specification of SANS 3001 GR3. and addition of methylene blue. It has LL 34, PI 13 and LS 4.6 suggesting that its heave potential is very low. All of the clay particles are about 2 micron or slightly larger in size, which is consistent with the clay being kaolinite. This is the only sample tested which showed no small, high CEC clay particles; all of the other samples showed a range of sizes and probable clay mineral types. Many agglomerations of clay particles are evident. Many appear to be properly dispersed, but many are not.

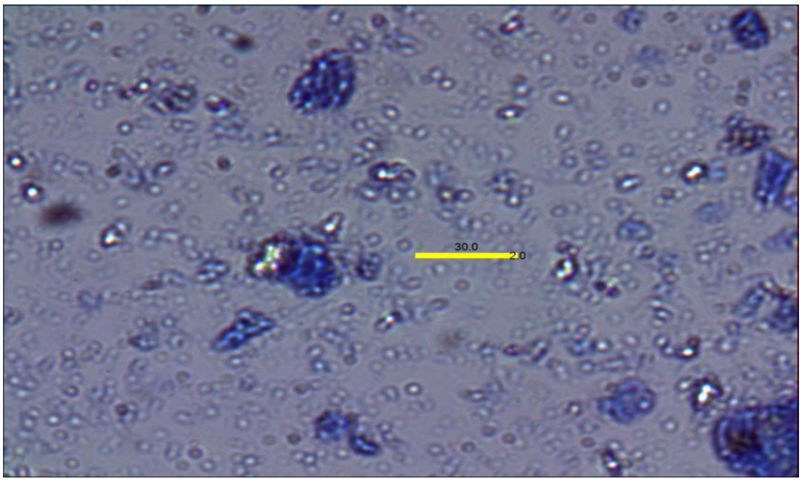


Figure 8.17: Western Cape yellow kaolinitic soil.

Since it is unlikely that these will settle at the rate expected of individual clay particles the clay fraction determination is again likely to be unreliable. The description “silty clay” seems inappropriate since little, if any silt is evident. The agglomerations of clay are, however of silt size and it could be that they have settled in the hydrometer at the rate expected of silt-size particles and been incorrectly assessed as silt.

8.10 Additional Tests

Although it appears that one or two of the samples tested showed fair dispersion (as in Figures 8.4 and 8.5), none showed unquestionably satisfactory dispersion (i.e. little or no clay-sized material forming agglomerations or associations with other particles). Some showed very poor dispersion (as in Figures 8.12 to 8.16).

Tests at the CUT geotechnical research laboratory were carried out to assess the effect of concentration of dispersant, volume of dispersant, length of time of submersion in dispersant and time of mechanical agitation. The GR3 procedure includes only minimum times for soaking and agitation, so this served only to check the adequacy of this aspect of the specification.

SANS 3001 GR3 calls for a minimum of 16 hours' submersion in the dispersing agent. Various periods from 16 hours to 2 weeks were tested. No visible improvement in dispersion was observed. SANS 3001 GR3 calls for a minimum mechanical stirring time of 15 minutes at 1500 rpm. Various periods from 15 minutes to 24 hours were tried with no visible improvement in dispersion observed. This tends to confirm that the specified minimum times are adequate. No visible improvement in dispersion was observed by doubling the quantity of dispersant used to treat the samples or by increasing the concentration of dispersant from 40 g/l to 60 g/l. This was not unexpected, since the UFS samples used 50g/l and showed no visibly significant improvement in dispersion from the GR3 samples.

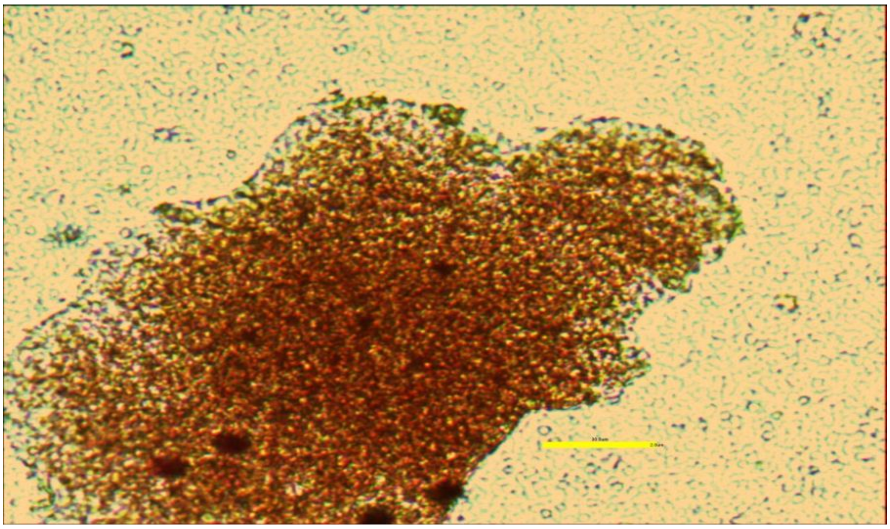


Figure 8.18: Dark brown sandy clay from a road project at Thaba Nchu in the central Free State after preparation for hydrometer analysis by soaking in dispersant and mechanical agitation according to SANS 3001 GR3

Samples taken from suspensions allowing little time for settlement allowed assessment of the dispersion of clay particles from sand-size particles. Dispersion appeared to be no better than from silt as can be seen in Figure 8.18

This sand grain is of the order of 200 microns long and 100 microns wide. It appears to be completely covered with clay size particles. Many clay particles around it are dispersed, although most are out of focus and cannot be clearly seen. With particles as large as this the depth of the suspension between slide and cover slip is so great that very little of the suspended material is in focus. It appears that treatment with dispersant and subsequent mechanical agitation has failed to dislodge clay particles from the sand grain.

8.11 Discussion

All of the clays tested showed some lack of dispersion. Every sample showed instances of clay remaining attached to larger particles or forming agglomerations with other clay particles. In some cases, the lack of dispersion was fairly small but in some cases a substantial fraction of the clay particles appeared to be undispersed. This suggests that it will not be reasonable to look for some universal factor by which hydrometer results could be corrected. It appears that predictions based on clay fraction determined by the procedure of SANS 3001 GR3 may be very unreliable for some soils. Since the SANS 3001 procedure is very similar to that of many other countries it is likely that this problem may be widespread. The samples prepared at the UFS Soil Science laboratory using soil science procedures with some differences to those of SANS 3001 GR3 showed visibly similar results to those prepared at the CUT laboratory using the GR3 procedure.

It might be reasonable to consider specifying different de-flocculants for different types of soil. Rodriguez *et al.* (2011) found that Lithium hydroxide is very efficient for high-CEC soils but is not efficient for dispersing low-CEC electropositive soils. Rolfe *et al.* (1960) found considerable difference in the clay yield given by a number of dispersants across different types of clay in hydrometer tests. Perhaps it is not surprising that the single dispersant specified for all South African soils appears to be reasonably adequate for some soils and completely inadequate for others. Changing dispersant may be futile however, since most of the soils tested showed mixtures of clay ranging from small, high CEC particles (much of it probably montmorillonite) to large, low CEC particles (much of it probably Kaolinite). If a dispersant is not efficient for several types of clay it will not give reliable results for these soils.

The methods of quantitatively assessing the efficiency of de-flocculants for geotechnical and soil-science purposes (Rodriguez *et al.*, 2011; Rolfe *et al.*,1960) take hydrometer yield as the standard of comparison. There appears to have been no attempt to assess how much of the clay remains undispersed. The use of even the most efficient dispersant for any particular clay may therefore give poor results. There is also the question of mechanical agitation. Rodriguez *et al.* (2011) noted that horizontal mechanical shaking in helicoidal motion, with the addition of coarse sand as an abrasive, is more effective for dispersion than the conventional method. They did note, however, that its efficiency is not the same for all soils.

8.12 Conclusion

It appears that Savage's suspicion that "De flocculation of many types of clay is seldom fully completed at the time of testing" is well founded. None of the clays tested reached good dispersion even when all aspects of the dispersion procedure were extended substantially. Clay coating of large (silt/sand) particles was observed to some extent in all samples which contained silt and sand particles - in some cases to a very considerable extent. Such particles will probably settle at the rate of silt/sand particles and their clay coating will not be assessed with the clay fraction. It may be advisable to consider the hydrometer unreliable for any critical analysis such as heave prediction. Work in progress at the CUT soil mechanics research group is attempting to find more reliable methods of assessing clay fraction but is at a very early stage and is unlikely to be able to give reliable quantitative results for at least two years. The quest for better methods of assessing clay content should perhaps become a priority on a wide scale.

CHAPTER 9: PARTICLE SIZE DISTRIBUTION FOR GEOTECHNICAL PURPOSES: TRADITIONAL AND AUTOMATIC PARTICLE SIZE ANALYSIS ASSESSED BY OPTICAL EXAMINATION

9.1 Abstract

A combination of sieving and hydraulic settlement is widely specified throughout the world to determine particle size distribution for geotechnical purposes. In the past three decades several kinds of apparatus have become available for automatic determination of particle sizes for grains and powders, particularly for the pharmaceutical industry. The speed and convenience of these instruments have attracted much interest for possible use in particle size analysis of soils. Agreement between traditional and new methods has generally been found to be very promising for coarse soils, but poor agreement for soils with significant fines fraction has prevented their general acceptance for geotechnical use. This paper examines possible reasons for the difficulty of analysing the fines fraction of soils, by using optical microscope imaging. It is concluded that fundamental problems, including dispersion and agglomeration, need to be overcome before any currently available particle sizing method can be reliably used for assessment of natural soils containing a significant clay fraction.

Keywords: Particle size distribution, PSD of soils, PSD of clays, Laser Diffraction, Hydrometer analysis, dispersion of clays.

9.2 Introduction

Particle size analysis forms the most basic and vital part of soils analysis. A combination of sieving and sedimentation techniques have traditionally been used in geotechnical soils laboratories for this purpose using standardised techniques as set out in for example ASTM D422-63 (ASTM 2002) and BS 1377-2 (British Standards Institution 1990). These procedures typically involve sieve analysis for the coarse fraction (sand and gravel size particles) and hydraulic sedimentation analysis for the fine (silt and clay-size) fractions. Despite many years of standardisation and reliance on the hydrometer or pipette, doubts have frequently arisen about the reliability of fines fraction determination. Perhaps this is one of the reasons for many months of absence of ASTM D422-63 after withdrawal in 2016. Large discrepancies have been noted. Kubota (1972) for example found very large differences in clay fraction for some soils depending on the details of sample preparation for hydrometer analysis. Keller and Gee (2006) noted that *particle size data must be treated at best as an empirical estimate*, and that *hydraulic methods work best for coarse-textured (low clay) structure-less soils.* It appears that it is the clay fraction which poses the greatest problem for particle size analysis of natural soils.

Much of our understanding of clays comes from scanning electron microscope (SEM) images. Preparation of samples for SEM imaging involves processes such as drying and gold plating. However, clay fraction is usually assessed by sedimentation procedures in aqueous suspension. In this form, clay particles are surrounded by vast numbers of hydrated ions. The hydrated diameter is several times that of the ion itself. A clay particle in aqueous suspension, closely surrounded by such structures held captive by electrostatic fields may look different from a SEM image and it may behave in a different way to that expected by reference to a SEM image.

In the 1970's several methods for inferring particle size distribution (PSD) by the scattering of light were proposed. In the 1980s instruments were commercially produced based on these and related ideas and have now become readily available. Such designs have reached a high degree of sophistication. The possibility of their use for PSD analysis of soils is attractive since they are convenient to use, require only a small sample size, allow a PSD analysis to be completed in a short time and generally give repeatable results with very fine grading (of the order 100 divisions).

9.3 Automatic Particle Sizing

Automatic particle sizing instruments employ a wide variety of techniques. One popular technique involves the evaluation of occlusions (shadows) cast against an optical detector illuminated with laser light as a stream of single particles crosses the laser beam. The merits of the single particle size (SPOS) technique have been assessed by White (2003) and Clayton, Abbireddy, and Schiebel (2009). White showed that the SPOS technique can accurately assess dry sand-size particles down to about 100µm. Clayton, *et al.* (2009) showed that the SPOS technique has promise for evaluating the form of sand-size particles but is not suitable for assessment of finer material. It seems rather unlikely that any dry powder technique will be successful for assessing clay fraction. Clay particles agglomerate on drying. In aqueous suspension chemical and mechanical dispersion can be applied to break down clay agglomerations. It therefore seems likely that techniques using aqueous suspensions would stand a better chance of assessing clays.

Possibly the most popular such technique is based on the principle of laser diffraction, which evaluates multiple particles as they pass in aqueous suspension through a laser beam. Instruments using laser scattering (LS) techniques have been assessed by a considerable number of researchers. Konert and Vandenberghe (1997) found that LS works well for particles of certified size, but for natural soils correspondence with traditional methods breaks down. They suggested ad-hoc re-classification of sizes by up to 8 size classes to achieve tolerable agreement. Eshel *et al.* (2004) assessed the LS technique with a range of natural soils from sand through silt to clay. They found significant discrepancies between LS and hydraulic PSD which did not follow a systematic pattern compared to sedimentation. LS under-estimated clay fraction but over-estimated silt fraction. Adjusting results to improve correlation in one particle size category made discrepancies worse in the other categories. They could not devise a meaningful correlation between the two approaches.

Abbireddy, Clayton and Huvenne (2009) assessed the use of the LS technique for evaluating the form of fine particles. They compared particle-sizer results against particles which had been restricted to a small size range by sieving, and whose form had been assessed by examining scanning electron microscope (SEM) images. They used manual settings to adjust the analysis accordingly. While this may be a valuable method of determining the shape or form, it also suggests that the sizer may give misleading results for particle sizes where SEM images are not available to suggest suitable correction factors for lack of sphericity. Di Stefano, Ferro and Mirabile (2010) did comparisons between PSD results from hydrometer and LS tests for 228 natural soils from Sicily. Their results were comparable to those found by Eshel *et al.* (2004), whose correlation between sedimentation and laser diffraction showed relative errors up to 104% for sand, 93% for silt and 144% for clay-size particles. Di Stefano *et al.* (2010) presented their results graphically, rather than numerically. Their graphs suggest even worse correlation for clay but better correlation for sand.

Goosens, (2008) tested ten different instruments for measuring hydrated PSD. He analysed four sediments with different median grain size and with no particles > 90µm. The instruments were based on laser diffraction, sedimentation, impedance, electrical sensing zone and optical techniques. His results showed disagreement between the various methods of at least 4:1 and often greater than 10:1 with little or no consistency across size categories or across the four sediments. He expressed the opinion that the LS technique appeared more credible than the others, but noted that without defining the exact criteria for comparison, none could be assessed as reliable. Abbireddy and Clayton (2009) reviewed 7 methods of automated PSD measurement techniques and found that for glass spheres, and clean, bulky sand all gave similar results. For small and medium chips of glass, flakes of mica and classified gold tailings results showed less good, but still fair agreement. For kaolinite results disagreed *by an order of magnitude*. Kaolinite is found in most natural soils. Their results, therefore, again throw doubt on the suitability of automated PSD equipment currently available for PSD of natural soils.

Bah, Kravchyuk and Kirchof (2009) suggested that the discrepancies between LS and hydraulic methods might be attributable to the variability of soil particle density as well as non-sphericity. Sedimentation methods assume a single particle density, whereas LS measurements are independent of particle density. Lack of sphericity affects both methods. LS should on average assign an irregular shaped soil particle a cross-sectional area greater than that of a sphere having the same volume and therefore inflate larger size fractions and underestimate the clay fraction. Non-spherical particles, on the other hand, have longer settling times than equivalent spheres, which should lead to overestimation of the clay fraction in hydraulic methods. The need for an agreed definition of size arises in all of these investigations. The traditional definition, based on particles passing through the square apertures of a sieve, cannot be applied in optical methods which assess other aspects such as a two dimensional projection (SPOS) or a scattering potential (LS). It is therefore difficult to compare results with conventional tests. Nevertheless, it would appear that for inert particles (grains of rock of any size from a few millimetres to a few tenths of a µm) optical methods hold out potential for rapid, repeatable, reliable and accurate PSD with very fine gradation of results. The problem remains, however, that the determination of clay fraction is a very important part of PSD assessment and automatic optical methods do not appear to have shown their suitability for this task.

9.4 Suitability for Geotechnical PSD

For geotechnical purposes it is more important to be able to assess the fractions of the major soil divisions: clay, silt and sand, than to assess fine gradations within these categories. Concerning the pipette method specified in BS 1377-2, Abbireddy and Clayton (2009) pointed out, *the standard test is configured only to determine the silt and clay fractions of materials, because these are the parameters most often required in geotechnics*. The clay fraction is of particular importance for many geotechnical considerations. The above review suggests that no automatic PSD technique has demonstrated the ability to give convincing estimates of clay fraction of natural soils. The standard hydraulic methods are also questionable for high clay content soils. This investigation seeks to assess the reliability of both automatic and traditional hydraulic methods of PSD.

9.5 The Uniqueness of Clays

The reason for clays presenting such a challenge is of great importance to the question of what techniques can be expected to assess them reliably. Clays generally consist, in principle, of silicon atoms, each surrounded by four oxygen atoms forming silicon-centred tetrahedral, and aluminium atoms, each surrounded by six hydroxyl groups forming aluminium-centred octahedra. The silica tetrahedral pack together to form extended sheets. The alumina octahedra also pack together to form extended sheets. These sheets can stack together in two distinct patterns. When silica and alumina sheets are stacked alternately they form a 1-1 mineral structure. When one alumina sheet is sandwiched between two silica sheets, a 2-1 structure results. In the 1-1 form, the pairs of sheets are held together by hydrogen bonds and form stable, non-expansive stacks which can be a few thousand sheets thick. Such particles tend to be large (~2µm) and bulky, with a low surface to volume ratio, or specific surface area (SSA). Kaolinite is the most common 1-1 clay mineral and occurs in almost all soils. In 2-1 structures each three-sheet sandwich is held to neighbouring sandwiches by interlayers of positively charged particles. Cations of various species like potassium or iron may fill the interlayer, producing various types of clay such as mica and illite. These are usually stable and only slightly expansive. Where hydrated cations fill the interlayer, smectites are produced, one of the most common being montmorillonite. In this case, where the interlayer is formed by hydrated cations, the structure is far less stable, the number of sheets stacked together to form a clay particle may be quite limited; they tend to be small and flaky with large SSA. These particles are expansive; the spacing between the three-sheet sandwiches depends on the number of hydrated cations in the interlayer. This simple picture is complicated by exchanging of ions in the clay structure. Clay properties are therefore complex, they depend not only on patterns of ion exchange, but on the nature of the underlying geology and the balance between chemical, physical and biological processes to which it has been subjected. These, in turn, depend on highly variable factors such as rainfall and temperature.

9.6 Cation Exchange Capacity

Clay particles at normal pH are negatively charged. Soil contains a variety of ions, both positively and negatively charged. The positively charged cations are attracted to the clay particles and are adsorbed in vast numbers on the exposed edges of the silica and alumina sheets. Soil in natural conditions contains water to some degree. When an ion is surrounded by a sufficient number of water molecules, it becomes a stable hydrated unit of size characteristic for the specific ion (several times larger than the ion itself). Hydrated ions are mobile and are attracted to the edges of clay particles. There is an order of preferential adsorption onto clay surfaces known as the Hofmeister series. When hydrated cations of higher rank in the series are available, they tend to displace those of lower rank, although any cation can displace the others if present in high enough concentration. The total charge which can be displaced by cations is known as the cation exchange capacity (CEC). For large, bulky, low SSA clay particles like kaolin CEC is low, typically 3-4 cmol/kg, while for small, high SSA particles like montmorillonite, CEC is far higher, typically 70-120 cmol/kg (Borden and Giese, 2001).

9.7 Dispersion

When clay is agitated in water it tends to disperse into small agglomerations in suspension. For estimation of clay fraction, it is necessary to break down the small agglomerations so that they can be assessed as individual clay particles. Certain chemicals have been found to be effective for increasing clay yield in settlement procedures like hydrometer and pipette analysis, particularly when pH is raised to 10 or above, at which stage the charge at the exposed edges of the silica and alumina sheets changes from negative to positive. A mixture of Sodium Hexametaphosphate $(NaPO_3)_6$ and Sodium Carbonate $(NaCO_3)$ is one such dispersant specified in many engineering standards. Attractive, but over-simplified theoretical explanations have been put forward to explain this dispersion. It is implied that when sufficient dispersant has been applied, then dispersion is effectively complete. This can be seen, for example in Mishra *et al.* (2011). On finding that ASTM D422-63 (which specifies the use of more dispersant than the Japanese standard JIS A1204-JGS 0131) gave a higher yield in the hydrometer, they concluded that the ASTM procedure *gave complete dispersion for all samples*. Experiments by other researchers e.g. Rolfe *et al.* (1960) have shown that there is an optimum quantity of dispersant for particular clay samples and adding more dispersant than the optimum leads to flocculation and fall in yield, contrary to the expectations of popular, simple theories. The assumption that maximum hydrometer or pipette yield signifies complete dispersion may be unjustified.

In attempting to explain dispersion, attention has turned to double-layer theory. The double layer theory was proposed by Helmholtz more than a century ago to explain the behaviour of an electrolyte around a charged rod. It is a continuum description which tries to account for the interaction of the charge densities, the charged mineral surfaces and the thermodynamics of ion interaction. It has been successful in qualitatively explaining reduction of dispersant efficiency at high concentration of dispersant, but has had to rely on untested assumptions for its attempts at quantitative analysis. Two points about double layer theory raise questions about its applicability to clays. Firstly, the theory was proposed for electrodes producing moderate electric fields. Measurements by Hu *et al.* (2015) suggest enormous field strengths at clay surfaces (in the region of 10^8 V m^{-1}) under certain circumstances. Secondly, the theory was proposed for particular electrolytes. In clays there are mixtures of a not necessarily easy to determine number of not necessarily easily identified electrolytes. Double layer theory relies on a number of assumptions which make its predictions uncertain in many respects. It does not appear to imply that maximum yield signifies complete dispersion. It does not predict the findings by (Hu *et al.*, 2015) that at low electrolyte concentrations, repulsion increases substantially. Hu *et al*. deduced that this may be due to non-classical (i.e. quantum mechanical) effects. It appears that the theory is as yet not able to reliably predict the effectiveness of any technique for the dispersal of clay suspensions.

9.8 Optical Assessment of Dispersion

The majority of images of clay particles are made with the scanning electron microscope (SEM). Such images give an indication of the appearance of clays typically at high magnification. Preparation for electron micrographs involves processes like drying and gold plating. SEM images may therefore not be relevant to clays in suspension, where clay particles are probably surrounded by multiple layers of closely bound hydrated ions to a thickness of several nanometres.

The optical microscope is well suited to examining aqueous suspensions. There are, however, objections which make the use of optical microscopes largely rejected. Suspended particles of the order 1µm and smaller are subject to Brownian motion; impacts by groups of water molecules impart random impulses leading to erratic motion in random directions. The high-powered objective lenses required to study particles as small as clay have very little depth of focus and tracking small, erratically moving particles can be challenging. It can also be difficult to differentiate between clay-size silt particles and particles of clay. These disadvantages can, to some extent, be overcome in two ways. Firstly, a digital camera can add digital magnification to produce useable images with less powerful objective lenses (which have greater depth of focus). Secondly, clay can be labelled with $C_{16}H_{18}N_3SCl$, generally known as “Methylene Blue” (MB), which preferentially replaces exchangeable cations in clay while leaving silt and sand unstained (Turoz and Tosun, 2011). These considerations led the authors to a qualitative assessment of the reliability of the LS technique for assessing the clay fraction of natural soils. For this investigation the particle size distributions of clayey soils were determined using an instrument which employs laser diffraction and is equipped with a facility to apply ultra-sound for dispersion purposes. This equipment is predominantly used for the determination of particle size distributions of industrial powders. The particular instrument for the tests here described is housed in the soil mechanics laboratory of the University of Pretoria, South Africa, where it is generally used for soils research purposes.

For correct particle size distribution to be determined, sufficient dispersion, either by means of mechanical agitation and/or chemical de-flocculation, needs to be achieved. The use of ultrasound may provide a means of increasing dispersion. Franco; Perez-Maqueda and Perez-Rodriguez (2004) found that sonication increased the structural disorder (synonymous with dispersion) in kaolinite. They found that prolonged treatment with ultrasound led to splitting of the crystal structure. The possibility that exposure to ultrasound for a limited period may increase structural disorder by de-flocculating and dispersing clays could be of value in estimating the clay fraction of soil more reliably. This possibility was investigated by Di Stefano *et al.* (2010). They did not go into detail but gave the impression that success was limited. This paper deals with attempts to assess the effectiveness of PSD by laser diffraction (with and without ultrasound dispersion) for natural clayey soils and compares the results to values obtained by traditional means. The effectiveness was assessed qualitatively using an optical microscope.

9.9 Materials and Methods

This investigation employed a Malvern Mastersizer 2000, an optical microscope equipped with 10x, 40x, 60x and 100x objective lenses and a 9 Megapixel digital microscope camera. The magnification achieved by the combined optical and digital contributions of objective and camera were assessed by measurement on a diffraction grating having 100 lines per mm (i.e. a line spacing of 10µm). The total magnification depends on the device on which the camera’s image is projected. The most meaningful expression of magnification or resolution is given by the number of pixels per µm (e.g. 37.7 pixels per µm for the 100x objective). The most convenient way of indicating scale is by superimposing an object of known size on the image produced. In this investigation a rectangle is superimposed on photographs and the dimensions of the rectangle are stated in the caption.

All photographs were taken of samples suspended in water. The depth of water depends on the size of the largest particles, which act as spacers between the microscope slide and its cover slip. Since all samples contained silt-sized particles, the depth of water was far greater than the depth of focus of the microscope’s high-power objectives. Much of the field of view is therefore out of focus in most of the photographs. Brownian motion causes rapid movement of the smallest particles. Extremely small particles could, however, be photographed where they formed part of agglomerations too large to be subjected to Brownian motion. In many instances minute particles appear to form nebular structures which often contain silt and coarse clay.

Samples of natural soils of different clay-content were tested. Three soils are of particular interest for their problematic behaviour on construction projects and most of the photographs shown deal with these three soils. Samples were sent for testing to accredited laboratories and hydrometer results from these laboratories are compared with LS results.

Normal procedure at the laboratory where the tests took place involves sizing without chemical dispersant. The first test compared this procedure against soaking for 24 hours in chemical dispersant before testing. The dispersant was a solution of 33g Sodium Hexametaphosphate together with 7g Sodium Carbonate dissolved in 1 litre of water. This is the dispersant specified in BS 1377 part 2 (British Standards 1990), SANS 3001 (SABS 2011) and engineering standards of several other countries. Some samples had a small quantity of methylene blue ($C_{16}H_{18}N_3SCl$) added after sizing to aid positive identification of clay particles. All tests with the Malvern Mastersizer 2000, took place at the soil mechanics laboratory of the University of Pretoria.

9.10 Results of the PSD Tests

9.10.1 Steelpoort clay

Samples of a red-brown soil with very high swell potential from Steelpoort in North-Eastern South Africa were examined. For this soil: 90% passed the 425µm sieve, LL = 78, PI = 43, LS = 18.0, Gs = 2.46. LS analysis was performed both with and without pre-treatment in chemical dispersant. PSD curves with and without ultrasound are shown in Figure 9.1. The curves for samples with no chemical dispersion show a large difference with and without ultrasound treatment. Those with chemical dispersion show very little difference with ultrasound treatment. The curves from the LS device are in marked disagreement with the hydrometer. The most necessary particle size information for most geotechnical puposes is the clay/silt/sand make-up of the soil. Figure 9.2 shows the results as bar charts divided into three classes: sand (>75µm), silt (2µm to 75µm) and clay (< 2µm). If automatic particle sizing cannot give reasonable values for these important groupings then fine grading into 100 intervals may be of little relevance, and may simply give a false impression of accuracy and reliability.

Figures 9.1 and 9.2 sugges that use of the apparatus without chemical dipersant may give unreliable results. This soil is so expansive and caused so much damage to structures founded on it that the clay fraction does not appear to be reasonsable. Ultrasound appears to disperse clay from the slt and sand components, but too little to give a credible clay fraction.

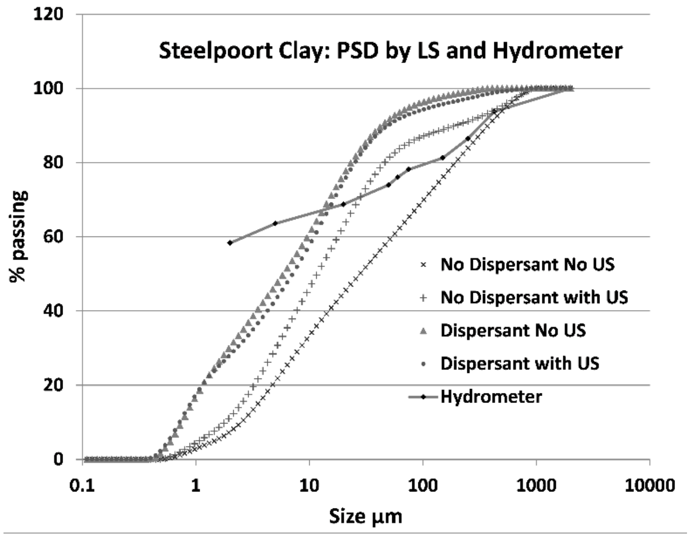


Figure 9.1: PSD curves for Steelpoort Clay by Laser Scattering.

Figure 9.1 shows PSD curves for Steelpoort Clay by Laser Scattering with no chemical dispersion with and without ultrasound, by Laser Scattering with chemical dispersion with and without ultrasound, and by hydrometer with chemical dispersion and mechanical stirring.

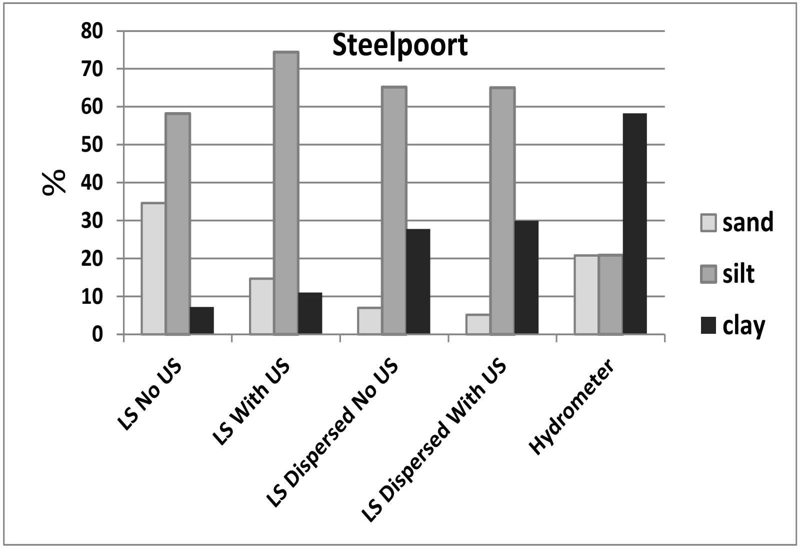


Figure 9.2: Broad particle size categories for Steelpoort Clay.

Figure 9.2 shows division of results for Steelpoort Clay into broad particle size categories of sand (> 75µm), silt (2µm to 75µm) and clay(< 2µm). Results are from Laser Scattering PSD, with and without chemical dispersion, both with and without Ultrasound dispersion, and also PSD by hydrometer.

It is clear from the two figures that after chemical dipersion the LS value for clay fraction is almost three times as much as without dispersant, and about half that of the hydrometer. With dispersant the silt fraction is about three times more than the hydrometer and the sand fraction about one third. This tends to support the observation by Eschel et al (2004) that it may be difficult to devise a meaningful correlation between the LS and Hydrometer results.

9.10.2 Botshabelo clay

Samples of a dark olive-brown clay soil from a roads project near Botshabelo in Central South Africa were examined. For this soil: 95% passed the 425µm sieve, LL = 66, PI = 36, LS = 12.2 and G_s = 2.47. This material was considered unsuitable for a road foundation and was removed from the site. LS analysis was performed after pre-treatment in chemical dispersant (with and without ultrasound). PSD curves are shown in Figure 9.3. There is again considerable disagreement between the LS and hydraulic results.The fact that this material was considered only suitable for removal rather than treatment suggests that the hydrometer results are more likely to reflect a credible PSD. There is little significant difference between applying and not applying ultrasound; both give a clay fraction less than one third of the hydrometer's value. Figure 9.4 shows the sand, silt, clay fractions of the clay.

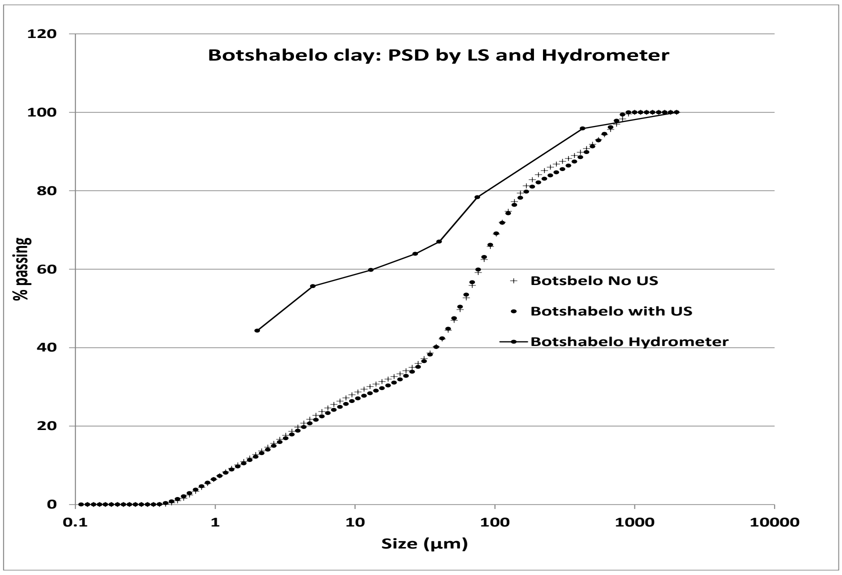


Figure 9.3: PSD curves for Botshabelo clay with dispersant

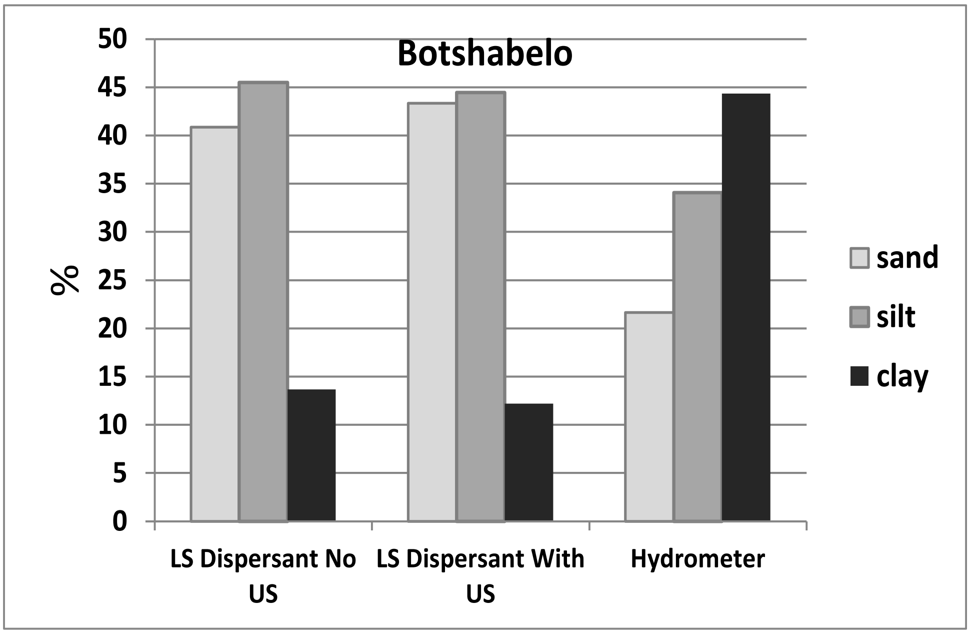


Figure 9.4: Broad particle size categories for Bosthabelo Clay.

9.10.3 Kimberley clay

Figures 9.5 and 9.6 give results for a light reddish-brownsoil from a housing development in Kimberley, central South Africa. For this soil: 60% passed the

425µm sieve, LL = 44, PI = 19, LS = 9.0 and G_S = 2.32. PSD results from the geotechnical investigation indicated a very low clay content, and consequently a low shrink/swell potential.

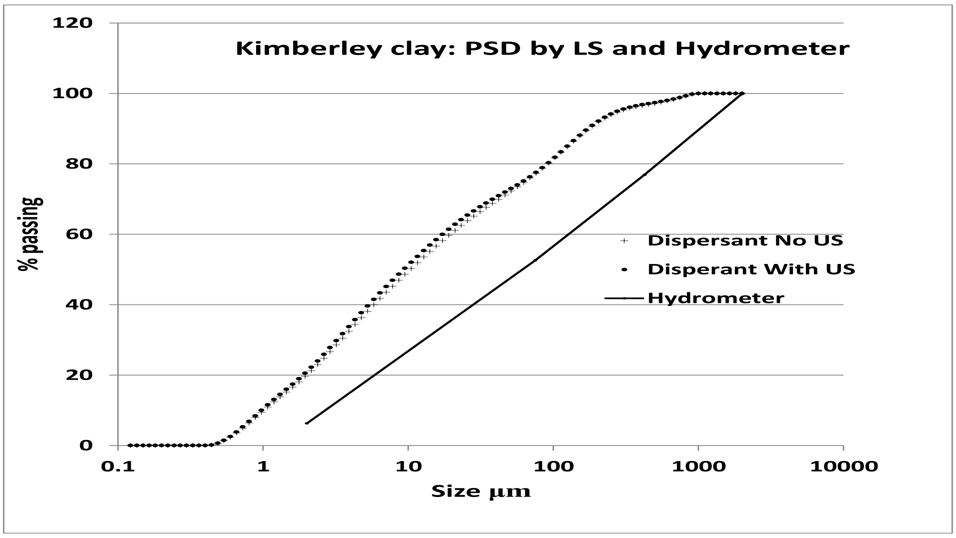


Figure 9.5: PSD for a soil from a Kimberley housing project.

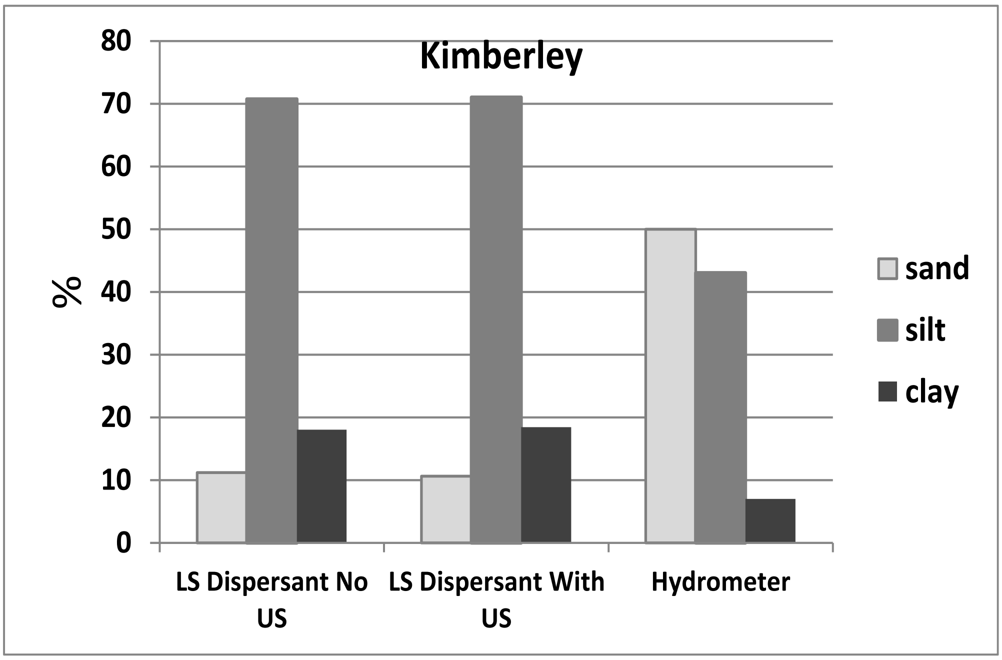


Figure 9.6: Broad particle size categories of Kimberley soil.

However, severe heave damage occurred within a short time after completion of the project. This soil represents the only sample of 9 soils tested in the investigation, where LS PSD indicated a higher clay content than the hydrometer.

The investigations considered by Eshel *et al.* (2004), Di Stefano *et al.* (2010) and Abbireddy and Clayton (2009) found LS methods gave consistently lower values for clay content than hydraulic methods. It would appear that this is an unusual soil, and the damage to the housing project would indicate that the true clay content may be considerably higher than any of these results suggest.

9.11 Results of Microscopic Examination

9.11.1 Steelpoort clay

Figure 9.7 shows typical low-power views after PSD with ultrasound. The sample is untreated Steelpoort clay (a) after PSD with ultrasound and (b) after PSD with ultrasound and subsequent addition of methylene blue. The depth of focus is of the same order as the depth of water between slide and cover slip and everything is inacceptable focus. Many small individual particles are visible together with some silt particles and some agglomerations which could be silt particles with clay adhering to them. In (b) practically all of the silt-size particles are stained, suggesting that they are either agglomerations of clay, or silt particles with clay adhering to them. Although low-power views give a good impression of the sample as a whole, higher powered views are more suitable for showing salient details - which are not necessarily typical, and much of the field of view is out of focus. The larger perspective (scale bar 80µm x 4µm), as in Figure 9.7 should be borne in mind.

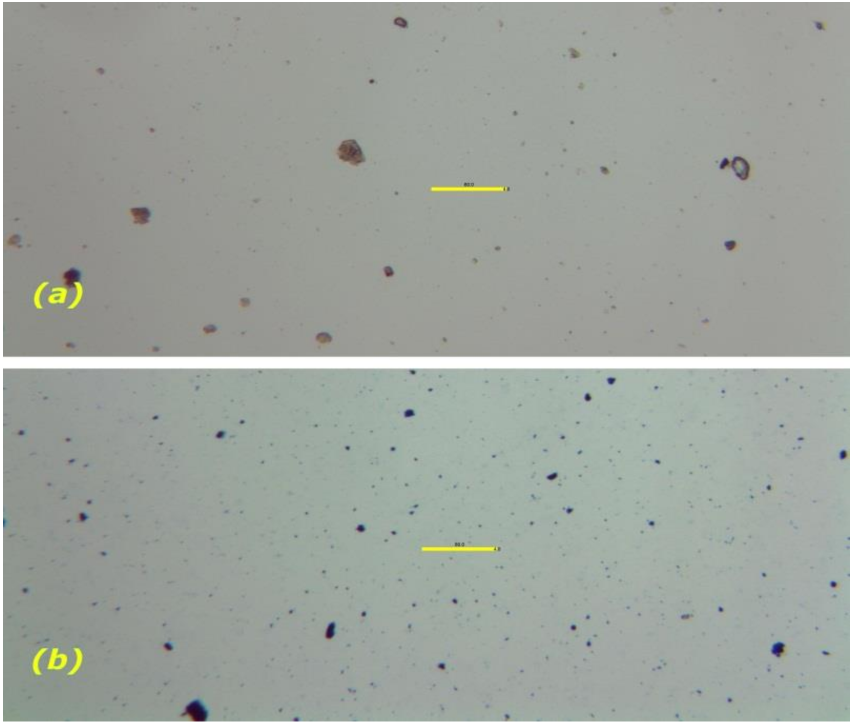


Figure 9.7: PSD of untreated Steelpoort clay sample.

Figure 9.8 (a) shows silt-sized particles after PSD with ultrasound. The two large particles both appear to be either agglomerations of clay-sized particles or silt completely covered with clay-sized particles. Not every silt particle appears to be coated with clay. A few apparently clean, uncoated particles are visible, but more clay particles appear to be in agglomerations than not. It appears that more effective dispersion is required than is provided by the mechanical dispersion and ultrasound applied by the apparatus. Figure 9.8 (b) shows a chemically dispersed sample which has gone through the PSD procedure without ultrasound. A number of particles appear to be dispersed, but the agglomeration in this photograph seems to consist largely of clay-sized particles held together by a tenuous nebula which is probably composed of particles too small for resolution at this magnification. (c) shows a similarly treated sample after PSD and the application of ultrasound. Again there are a few well dispersed particles but the main feature of the photograph appears to be a grain of silt largely covered by clay-size particles.

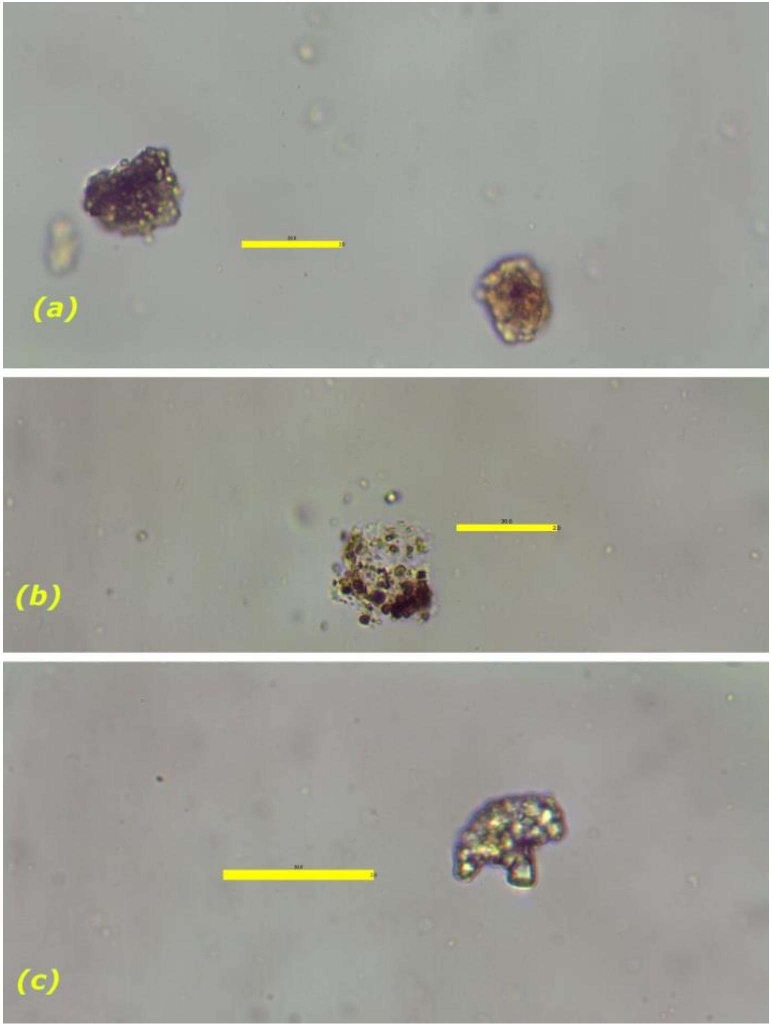


Figure 9.8: Steelpoort clay; (scale bar 30µm x 2µm).

(a) After no chemical dispersion, PSD without ultrasound: (40x objective)

(b) After chemical dispersion and PSD without ultrasound: (40x objective).

(c) After chemical dispersion and PSD with ultrasound: (60x objective).

The addition of methylene blue effectively labels the clay, as can be seen in Figure 9.9. In (a) many clay particles between 1 and 2µm are dispersed, but others appear to have tenuous associations with small clouds of very small particles. The larger agglomerations could be silt with a complete coating of clay particles which are so deeply stained that their form is not discernable. The silt-sized agglomerations will probably be counted by the particle-sizer as silt and the clay fraction will therefore be under-estimated. It appears that this is the case whether ultrasound is used or not. Figure 9.9 (b) shows the same sample after chemical dispersion and PSD with ultrasound. It shows a tenuous silt-size agglomeration which appears to be almost entirely made up of clay particles, most of which are very small. Figure 9.9 (c) is an enlargement of part of Figure 9.9 (b). The particles making up the blue-stained nebula appear to be smaller than 0.1µm.

It could be expected that for this particular clay, neither treatment with ultrasound, nor with the standard chemical dispersant, nor a combination of both will lead to an accurate estimate of clay fraction. It could be expected that LS might register such particles incorrectly as silt-sized, whereas the hydrometer might possibly register them correctly as clay, since the settling speed would probably be considerably slower than a single, solid particle of this size.

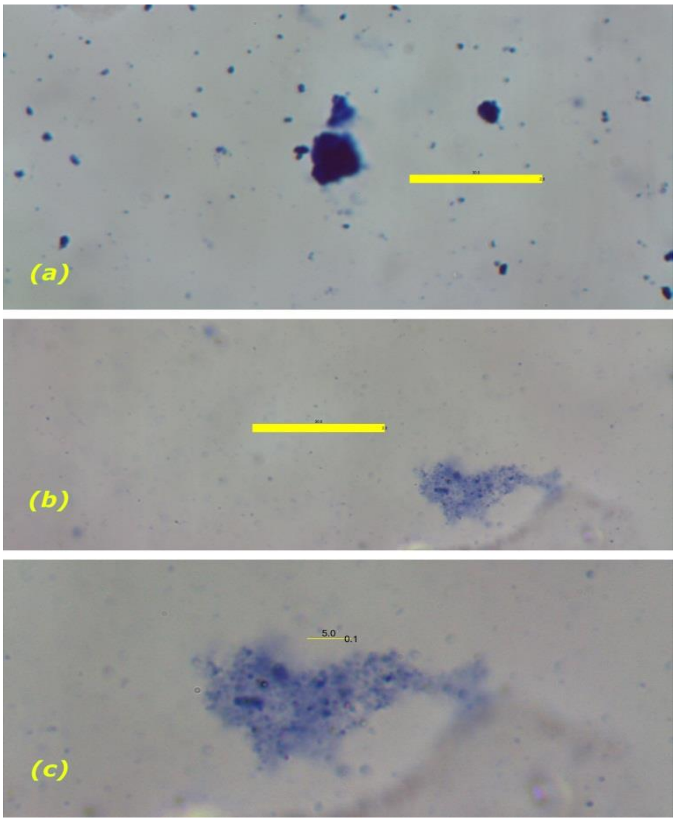


Figure 9.9: Steelpoort clay, after addition of methylene blue following PSD.
(a) No chemical dispersant with ultrasound: scale bar 30µm x 2µm.
(b) Chemical dispersant with ultrasound: scale bar 30µm x 2µm.
(c) Enlargement of (b): scale bar 5µm x 0.1µm.

The question of the reliability of the Hydrometer as the standard of reference remains. A sample of Steelpoort clay (Figure 9.10) was prepared for hydrometer analysis by soaking in dispersant for 24 hours and mechanically stirring for 15 minutes at 1500 rpm before examining under the microscope. The result can be seen in Figure 9.10 (scale bar 30µm x 2µm). Note that concentration of particles is far higher in the hydrometer than in the LS sizer. Although many particles of 2µm and smaller are dispersed, it appears that a large proportion of visible clay has formed groupings which are larger than clay-size and are not likely to be counted by hydrometer analysis as clay. The true clay fraction may therefore be considerably greater than any of the results found above.

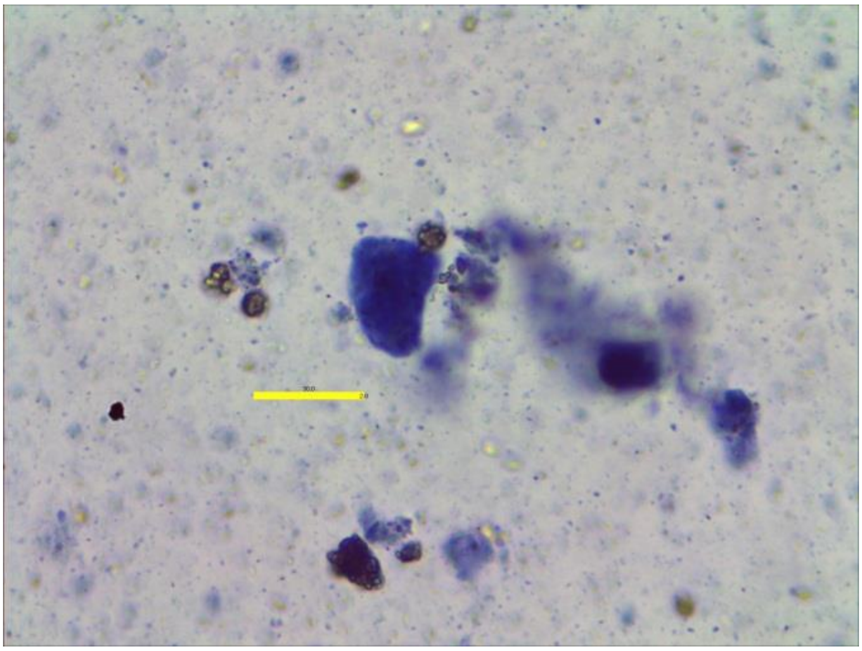


Figure 9.10: Sample after dispersion and agitation for hydrometer analysis.

9.11.2 Botshabelo clay

Samples of the soil from Botshabelo which had been immersed for 24 hours in dispersant and then analysed using the particle sizer with and without the application of ultrasound were examined under the microscope. Figure 9.11 (a) shows a large particle which is apparently completely covered with clay-sized particles. Figure 9.11 (b) shows a coarse silt-size particle covered by small, high CEC, deeply blue-stained particles, silt particles covered with larger clay-size, low CEC, lightly stained particles, and also scattered, well dispersed clay-sized particles. Many, possibly the majority, of the clay particles have not been dispersed. For comparison with the Hydrometer standard, a sample of this soil was prepared accordingly. Figure 9.11 (c) suggests that while a large number of silt and clay size particles can be seen to be well dispersed, there are many clay particles forming an agglomeration of about 40µm diameter. These clay particles may be covering a silt particle. They will probably not be assessed as clay in the hydrometer analysis. It would again appear that the hydrometer is not an adequate standard by which to compare clay fraction determination and it is likely to under-estimate clay fraction quite significantly for this soil.

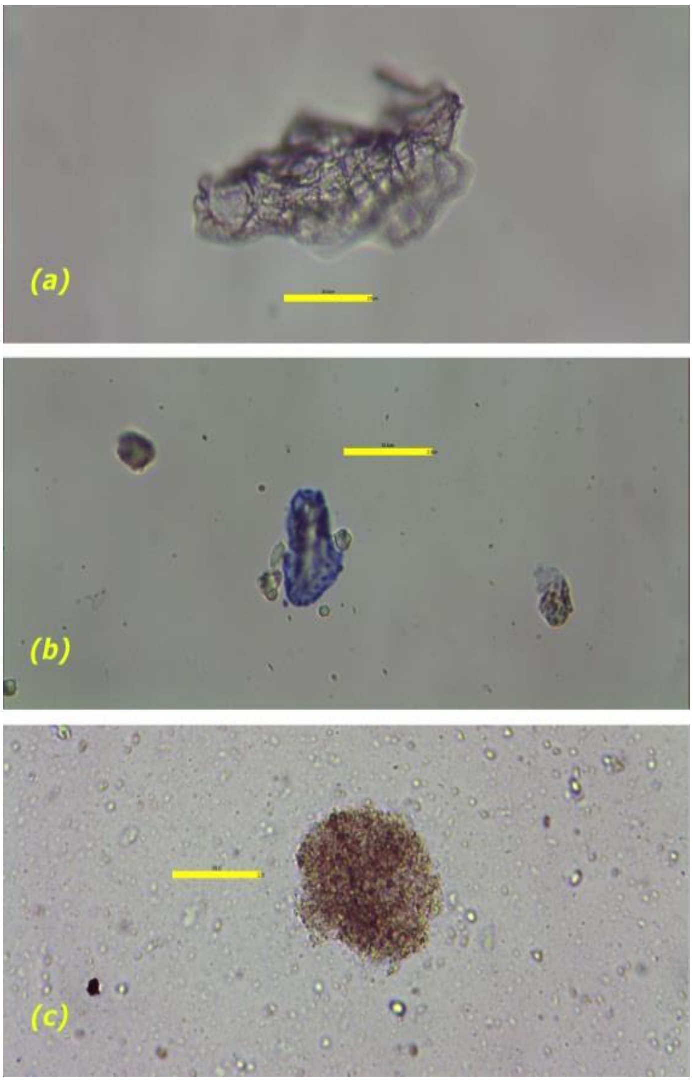


Figure 9.11: Botshabelo clay after chemical and mechanical dispersion.

(a) Laser particle sizing (no ultrasound).

(b) Laser particle sizing with ultrasound and methylene blue.

(c) Chemically and mechanically dispersed for Hydrometer analysis.

9.11.3 Kimberley clay

Figure 9.12. Shows a sample from a housing project in Kimberley in central South Africa. Laboratory results from the geotechnical investigation had indicated the

clay content to be very small over the whole site. Expansive potential was therefore assessed as low and no significant shrink/swell was designed for. One of the houses became structurally unsound due to heave and had to be demolished before construction was even completed. In the course of one year many of the houses became seriously damaged and required major repairs.

Figure 9.12 (a) shows a sample treated with dispersant and subjected to laser diffraction analysis without ultrasound. Dispersed particles are visible, but there is a large conglomeration of clay-sized particles which are not dispersed. Figure 9.12 (b) shows the same sample after ultrasound. Again there are many dispersed clay-sized particles, but others appear to be attached to a silt particle and others form fine-silt-size associations of a few clay-size particles. Figure 9.12 (c) shows the same sample after addition of MB. It is difficult to find any particles at all which are not stained. They are therefore of high CEC clay or covered with high CEC particles. The majority of particles are extremely small. The nebula at lower left is approximately 30µm in diameter and appears to consist entirely of high CEC / high SSA clay particles.

Most of the visible particles are agglomerations larger than 2µm and would probably be analysed by the particle-sizer as particles larger than clay. This would probably be the case with hydrometer analysis also and could explain why the estimate of clay fraction was deceptively small in the geotechnical investigation’s particle size analysis. The output from the LS particle-sizer indicated the clay-size fraction to be a little under 20%. Visual assessment of the methylene blue-stained samples typified by this photograph suggests a considerably higher value for the actual clay content. The extensive damage to the building project also suggests high clay content. Figure 9.12 is a sample from Kimberley housing project after dispersant treatment and laser scattering PSD.

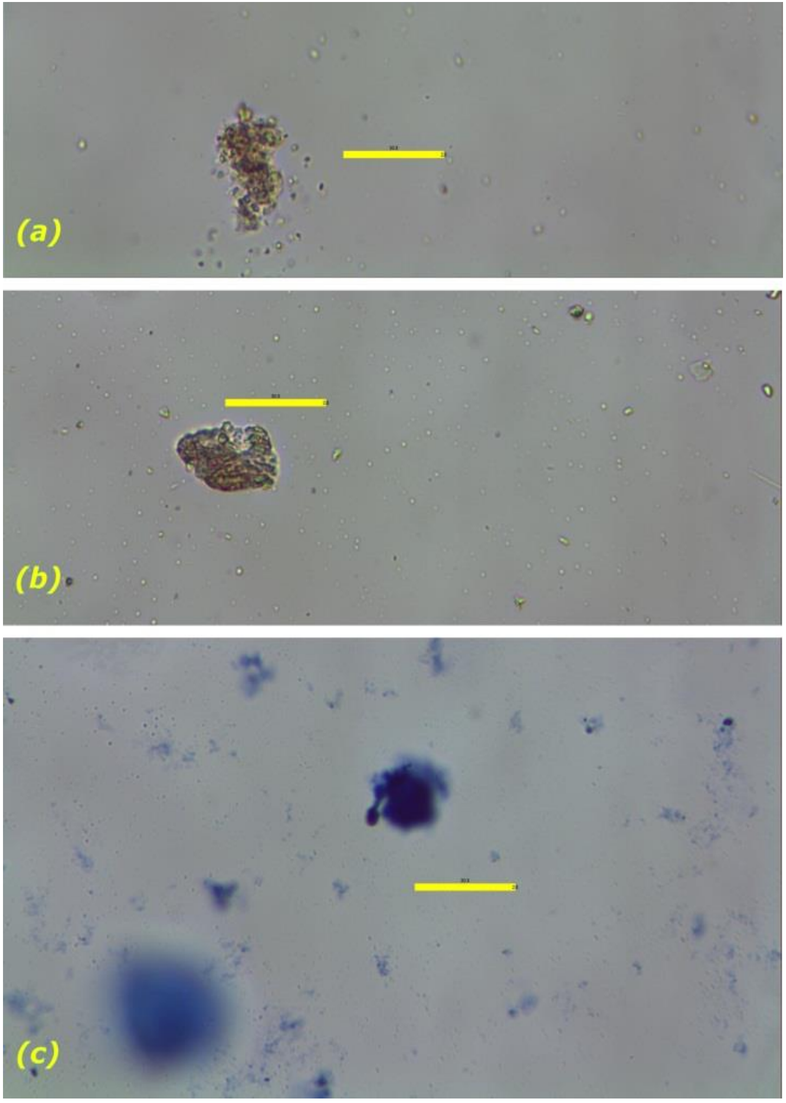


Figure 9.12: Sample of Kimberley soil (scale bars 30µm x 2µm).

(a) No ultrasound;

(b) with ultrasound;

(c) With ultrasound and methylene blue labelling.

Several other clayey samples were tested; PSD results gave poor correlation in all cases. One sample, from Virginia in central South Africa is shown in Figure 9.13 (a) is a high power image (100x objective) of a nebula of small, high CEC/high SSA particles containing many larger particles. This follows chemical and mechanical dispersion in preparation for hydrometer analysis. Part of this image is enlarged in Figure 9.13 (b), where the scale bar measures 2µm by 0.1µm, which covers the range of sizes normally considered typical for most species of clay particles. It appears that the individual particles making up the nebula are smaller than 0.1µm. Clay particles smaller than 0.1µm would be typically smectites.

However, some organic colloids can also be smaller than 0.1µm (Dur *et al.*, 2006). Organic colloids are widely distributed in soils and this raises the possibility that these features may possibly be not clay, but organic colloids.

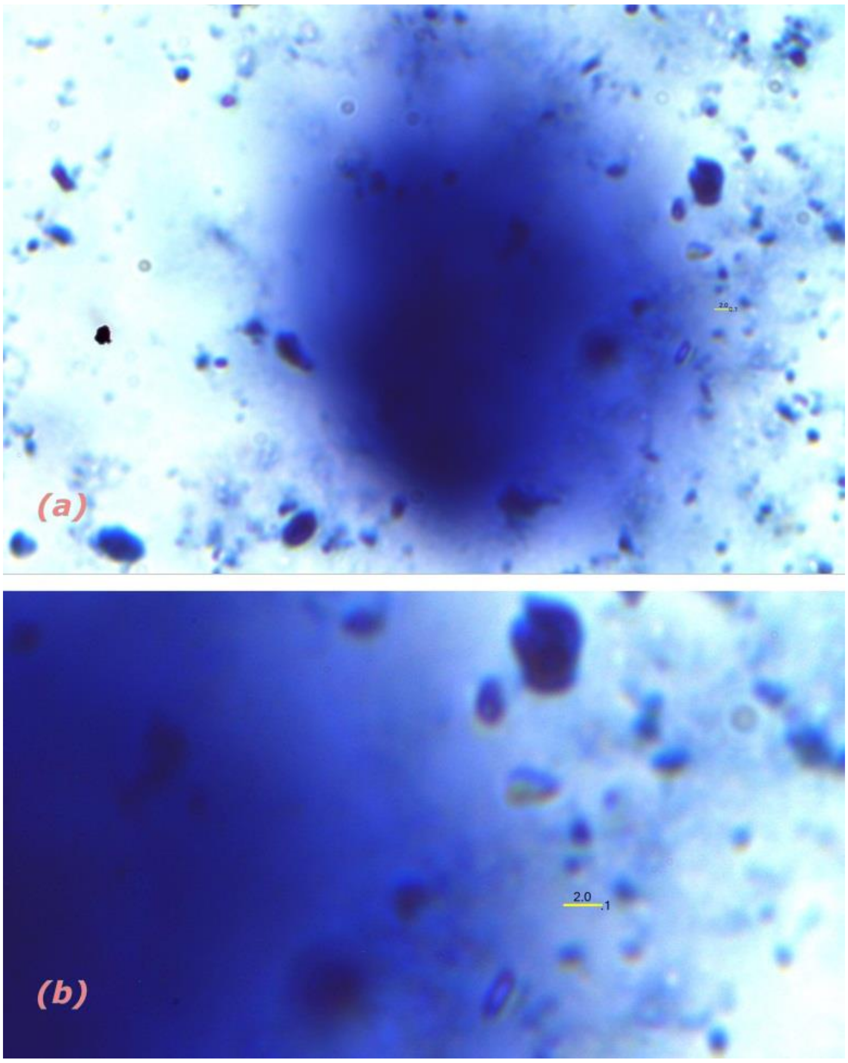


Figure 9.13: Methylene blue-stained clay-size and silt-size particles

9.12 Discussion of Results

The laser and ultrasound scattering particle-sizer is quick and convenient to use. It appears to give satisfactory results for analysing inert powders and its use in the pharmaceutical industry is widespread. Its use for analysis of natural soils, while tempting from the point of view of its speed and simplicity appears to be dubious. Even after prolonged treatment with chemical dispersant there appears to be significant flocculation of clay particles whether ultrasound is employed or not. It would appear likely that much of the clay in many samples would be assessed as larger particles, since they form part of silt or even sand-size agglomerations. This would give an under-estimate of clay fraction. Examination of sediment following settlement of samples prepared for hydrometer analysis has shown that the agglomerations visible in microscopic inspection tend to settle more slowly than solid particle of similar size (Stott, Monye, and Theron, 2016). Sand-size agglomerations tend to settle with the coarse silt, large silt-size agglomerations tend to settle with the fine silt and small agglomeration tend to settle with the clay. It appears that Laser Scattering PSD should under-estimate clay fraction even more seriously since particle density is not thought to influence its assessment.

The extremely small size of many particles suggested by, for example Figures 9.7 (b), 9.10 (c) and 9.11 may be beyond the resolution of current LS devices. Their apparent propensity to form extended nebular structures may explain the somewhat surprising appearance of electron micrographs of smectites. The overall size of the petal-like structures often found in SEM images of smectites is comparable to that of the nebular structures of Figures 9.7 (b), 9.10 (c) and 9.11. It might be possible that drying in preparation for SEM imaging may cause extremely small clay particles in such nebulae to condense into very thin, undulating sheets, making the individual particles difficult to distinguish, especially after gold plating. On the other hand, it is possible that some or all of these very small particles could be organic colloids (which commonly form part of soils to considerable depth). PSD for geotechnical purposes does not usually consider the effect of organic colloids. In view of the apparent role of the observed nebular structures in binding clay particles into large groups and preventing the entrapped particles being counted as clay, it may be necessary to consider removal of organic colloids to allow meaningful PSD. The removal of organic content has been a subject of much research by soil scientists primarily interested in determining total organic content. Mikutta *et al.* (2005) have reviewed the soil science research on this subject and found that it is a complex process which can produce permanent and major changes in the make-up of clay minerals, precipitation of metals, changes in SSA of up to 90% and replacement of organic particles by inorganic particles in the interlayers of 2-1 clay structures. Efficiency and results depend on the elements in the soil, the pH, types of clay present, depth of sample and other factors. Its effect on PSD would require detailed investigation. BS 1377 part 2 specifies an H_2O_2 treatment for high organic content soils like peat, but warns against its application for most soils because of its effects on clay minerals. Di Stefano *et al.* (2010) applied an unspecified H_2O_2 pre-treatment to their samples for removal of organic content but their results (as noted above) suggest that its effect, although *recognizable*, did not lead to good correlation for the clay fraction. Abbireddy and Clayton (2009) also treated their samples with H_2O_2 but also found that results differed by *an order of magnitude* between the automatic and hydraulic methods.

9.13 Conclusion

It appears that the laser-scattering particle-sizer is not a suitable instrument for assessing natural soils which contain a significant clay fraction. The deficiencies seem to be attributable largely to difficulty in dispersing the materials tested. It also appears that the hydrometer is not a reliable standard by which to compare performance in this regard. It is possible that difficulty of dispersion could be peculiar to the soils tested, all of which could, possibly, be classed as tropical soils. None are from within the tropics, but all are from latitudes less than 30˚S. Tropical soils are known to have certain peculiarities. However, salient aspects of these findings are similar to those of other researchers Eshel *et al.* (2004), Abbireddy and Clayton (2009), Di Stefano *et al.* (2010) who worked with soils from California, UK and Sicily respectively. All of these researchers noted poor agreement but did not suggest a plausible reason. It would seem that a reliable method of assessing clay fraction remains elusive and automatic PSD methods in their current form are unlikely to give acceptable analysis of soils containing significant clay fractions. Currently available chemical dispersants, mechanical agitation and ultrasound all appear to be ineffective, particularly for dispersing extremely small, high CEC / high SSA clay particles. None of the current methods appear likely to reliably assess natural soils with significant clay content. It would appear that a search for a reliable method of clay fraction determination should be given high priority. Until such a method is found it might be advisable to examine critically important samples using optical microscope procedures as described here to guard against gross error.

CHAPTER 10: ASSESSMENT OF RELIABILITY OF THE HYDROMETER BY EXAMINATION OF SEDIMENT

10.1 Abstract

A fundamental aspect of the characterization of any soil is the assessment of its particle size distribution. While this is relatively easy for the coarse fraction it remains problematic for soil fines particularly for the fraction less than 2µm. Hydrometer analysis has been the standard tool for fines assessment for many years but there may be serious shortcomings. Nettleship *et al.* (1997); Savage, (2007); Rodrigues *et al.* (2011) and many others have pointed to a number of problems facing the hydrometer. Some of the questions have been addressed by laser scattering techniques (e.g. Eshel *et al.,* 2004), but others, including completeness of dispersion and the amount of clay carried down with coarser fractions remain problematic. This investigation assesses some aspects of the reliability of the hydrometer by isolating and testing the sand and silt fractions after settlement. Microscopic examination is used to compare the composition of sediment layers with that expected according to hydrometer theory.

10.2 Introduction

Hydrometer analysis is widely used internationally for determination of the clay and silt fraction of soils in both engineering and soil science practice. The standard procedure used in South Africa for Civil Engineering purposes is detailed in SANS 3001 GR3 (SABS 2012). The test has a number of theoretical weaknesses. It monitors the change in density of a settling suspension - theoretically at one level in the suspension. It has been pointed out, by e.g. Rolfe *et al.* (1960) that it averages the specific gravity over the submerged part of the instrument and therefore depends on the shape and depth of hydrometer submergence, which may bring errors into the analysis.

Others e.g. Savage (2007) have noted that hydrometer analysis relies on Stokes Law (which assumes all particles to be spherical); it assumes that all soil particles have the same density; it assumes complete dispersion of clay particles at the time of testing; and it assumes that fine particles are not carried down by coarse particles – all of which assumptions are dubious. Nettleship *et al.* (1997) suggested that hydrolysis of polyphosphate dispersants may cause underestimation of clay fraction. Rodrigues *et al.* (2011) proposed that soil mineralogy should be taken into account and probably different treatments and different dispersants are required for different clay types. In addition, Keller and Gee, (2006) noted that hydraulic soil property estimates work best for coarse-textured, structure-less soils of low clay content.

One of the most problematic aspects of South African soils (from an insurance claims point of view, from the point of view of frustration to the government's attempts to provide durable low-cost housing to the poor, and from the point of view of several aspects of the performance of roads, water supply conduits, etc.), is the behaviour of expansive clay. Most of the methods for predicting the severity of heave which may be expected on a site underlain by fine-textured soil rely on an estimate of clay fraction. As noted above by Keller and Gee (2006), clay is the soil material which is least amenable to hydraulic methods of analysis, and hydrometer results can be expected to be particularly unreliable for these most problematic soils.

Critical aspects of soil behavior depend on clay mineral content rather than particle size as given by the hydrometer test. It is assumed that clay particles range from 2 microns downward, and that particles larger than 2 microns are silt. There are, however, some clays whose particles can be considerably larger than 2 microns (e.g. kaolinite, illite and halloysite) also silt particle sizes may range down to 1 micron. The range of sizes between approximately 1 and 2 microns may therefore be mineralogically either clay or silt.

Stott and Theron (2016) examined suspensions after preparation for hydrometer testing following the procedures of SANS 3001 GR3. The suspensions were examined using a light microscope / digital camera combination. They found three aspects of incomplete dispersion which suggested that unreliable results might be obtained by hydrometer analysis. The first was adhesion of clay to larger-sized particles, the second was agglomeration of clay particles among themselves and the third was agglomeration of clay and silt particles into groups of appreciable size. They speculated that these agglomerations would not precipitate at the rate expected for clay and would therefore give misleading results in hydrometer analysis. The objective of the investigation here described is to evaluate the hydrometer test for assessing typical South African clayey soils in light of the above objections.

10.3 Methods and Materials

10.3.1 Soil samples and treatments

A range of plastic clays from typical roads and housing projects were tested. Samples were treated with Sodium Hexametaphosphate and Sodium Carbonate as per SANS 3001-GR3. For control purposes examination of untreated samples were considered to serve as a check on the effectiveness of the dispersant for each soil. Preliminary tests, however, showed that without treatment in dispersant, agglomerations are so large that meaningful information would not be obtained. Samples were prepared in accordance with SANS 3001-GR3:2012. After being soaked in dispersant solution for not less than the prescribed minimum of 16 hours they were mechanically stirred at 1570 rpm for a minimum of 15 minutes and transferred to a settlement container specifically designed for this investigation. Distilled water was added to reach the dilution indicated in SANS 3001 GR3.

Samples were left to settle for approximately two days, excess water was carefully removed by a suction pipe. Containers were then transferred to an oven and dried at 45°C until the sediment was solid enough to be removed from the containers. The various fractions, which were usually visually distinct, were separated. SANS 3001 GR3 is concerned primarily with determining sand and silt fraction and the last reading which is given as normal is 12 minutes. The sub 2µm fraction can be deduced by subtraction. An optional final reading at 12h is mentioned if a reading is required for the sub 2µm fraction. The clay topping was treated simply as a protective layer to shield the silt layers from disturbance while removing water from the container and was not examined. The separated layers were prepared for microscopic examination by the addition of de-ionised water.

10.3.2 Settlement container

A settlement container was designed to be waterproof while having one side removable for the extraction of settled samples. Figure 10.1 shows two of these containers, one before closure, the other with water seal and front cover in place.

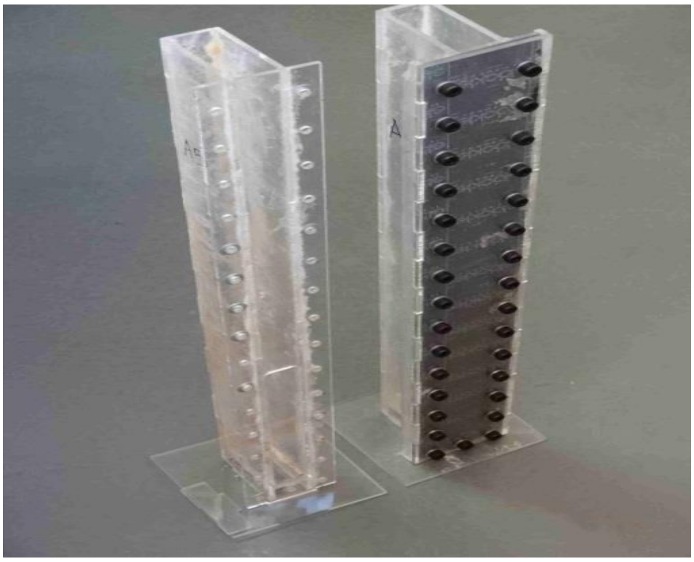


Figure 10.1: Settlement containers: 430x50x20mm: 0.43ℓ.

10.3.3 Microscopic examination

An optical microscope with 10x, 40x, 60x and 100x objectives was used for this investigation. The microscope was equipped with a 9 mega-pixel digital camera. The combined optical and digital magnification was assessed by measurements on a micro ruler. Magnification can be expressed in various ways. For example, the spacing between 10 micron graduations on the micro ruler was 60mm on the viewing screen when using the 60x objective, indicating a magnification of 6000 times. Alternatively, the magnification can be expressed as the number of pixels per micron (22.48 for the same objective). The most useful way of indicating magnification is by the inclusion of a scale on photographic results. All photographs shown here have a scale rectangle 30µm x 2µm.

After removal of sediment from the container and separation of the visible layers, small samples of each layer were suspended in de-ionised water. A drop of that suspension was placed on a meticulously cleaned microscope slide and covered with a similarly cleaned cover slip. Photographs were taken of samples before and after addition of methylene blue.

10.3.4 Methylene blue

Methylene blue - C16H18 N3SCl - (MB) has been found to be effective in labeling clay minerals by replacement of exchangeable cations. Clay minerals have cation exchange capacity (CEC) which depends on their atomic structure and specific surface area (SSA). The more active clays, like the smectites, have high CEC and SSA. Such clays exchange cations with MB readily and become stained easily. Less active clays like kaolinite and halloysite have low CEC and SSA and require higher concentration of MB to become visibly coloured. MB staining techniques have been found useful in evaluating possible clay activity and estimation of CEC. Procedures are specified by the *Association Française de Normalization* (AFNOR) and by the American Society of Testing Materials (ASTM). Both standards describe procedures for obtaining a semi-quantitative evaluation of the activity of a soil and indications of the probable type of clay minerals contained in the soil. AFNOR (1993) derives the “*valeur de bleu*” (V_B) and ASTM (1984) derives a comparable “methylene blue index” (MBI). Chiappone *et al.*, (2004) came to the conclusion that both procedures can give good estimates of clay content in certain circumstances. For the purpose of this investigation a MB solution with a concentration of 0.5g/ℓ was used for staining purposes. After examining unstained samples, small quantities of this MB solution were progressively added to the soil suspensions and samples were examined microscopically after each addition.

10.4 Photographs and Findings

10.4.1 Sand layer

The sand layers were expected to contain small quantities of silt and clay present at the lower part of the containers at the start of precipitation. This expectation was met, as can be seen in Figure 10.2, a photograph of material from the sand layer from an active clayey soil from Bloemfontein. A small quantity of Methylene Blue has been added. A scale-rectangle of 30 x 2 microns has been included in all of the microscope images. Some photographs show conditions before, others after, addition of MB. A few silt and clay particles are visible in Figure 10.2. The silt-size particle within the white circle is completely covered by particles of approximately 2 microns which show very little Methylene Blue staining and are probably kaolinite. Almost all of the sand particles have a thin covering of extremely small, deeply blue-stained particles which are probably montmorillonite. The two sand grains at upper left appear to be joined by a solid bridge of these clay particles (within the white rectangle) and at the lower left there is a silt-size arc (within the white pentagon) which appears to be entirely made up of the same minute clay particles.

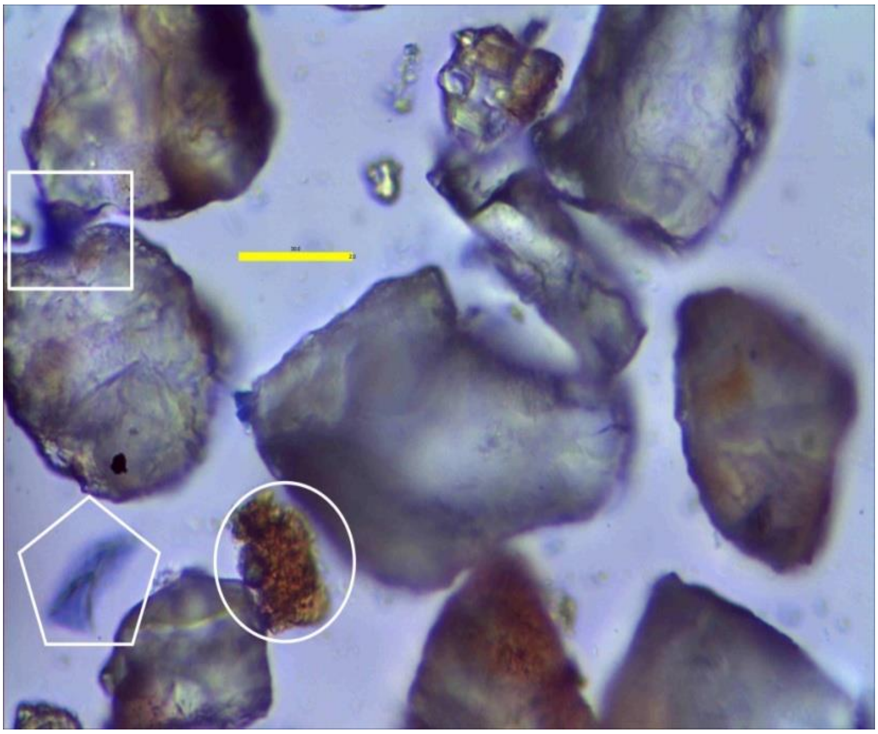


Figure 10.2: Sand layer of Bloemfontein soil.

Figure 10.3 is a view of a grain of coarse sand from the lowest deposition layer of a clayey soil from a roads project in Thaba Nchu. The sand grains visible in this photograph show fuzzy blue-stained edges consisting of very small, deeply stained clay particles (probably montmorillonite). The larger sand grain has a marked projection of these particles near the upper right corner and a more tenuous projection on the upper left side.

Figure 10.3: Sand layer of Thaba Nchu soil.

Figure 10.4 is from the sand layer of a soil from a housing project in Kimberley. The hydrometer analysis performed as part of the geotechnical investigation for the project indicated very little clay. Heave damage, however, indicated significant clay.

Figure 10.4: Sand layer of Kimberley soil (1).

The agglomeration shown consists of silt particles of various sizes. Each silt particle has a coating of deeply blue-stained fine clay particles, and it appears to be these clay particles which hold the silt together in a large, sand-sized agglomeration. Since it is found in the sand layer it seems probable that it precipitated out of suspension at a rate suitable for a sand grain, rather than at the rate expected for silt.

Figure 10.5 is from the same sand layer of the Kimberley soil shown in Figure 10.4. The structure visible appears to be an agglomeration, possibly of silt, (possibly also sand) or possibly part of a root-hair, within a mass of particles with size close to 2µm. These show only slight staining from the very small quantity of Methylene Blue added and are probably low-CEC clay. At top right and along the left side smaller, deeper stained particles are visible.

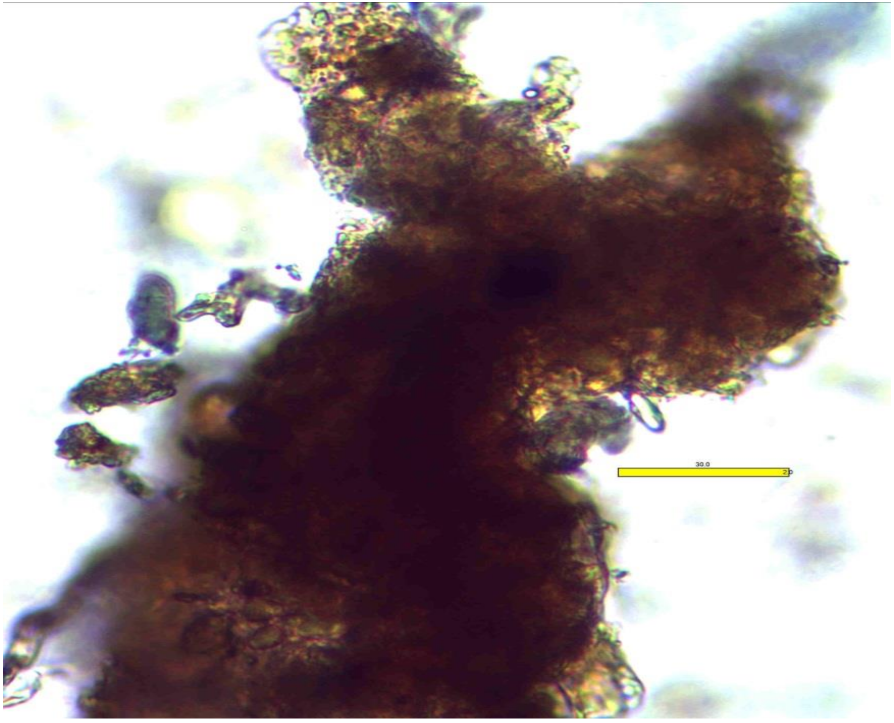


Figure 10.5: Sand layer of Kimberley soil (2).

They are high-CEC / high SSA clay. The size of this agglomeration is that of a large sand grain, and it appears to have precipitated with the coarse sand particles. Little, if any of it appears to be sand. A significant fraction is clay. One of the clays (from a Bloemfontein housing project) showed good dispersion of clay from the sand particles.

Figure 10.6 shows grains from the coarse sand layer. Very few clay particles appear to be attached to the sand grains at the right and centre. The sand grain on the left has a partial coating of very small clay particles visible as a fuzzy covering, particularly at the top. Dispersion for the sand grains of this soil appears to be more successful than for the other soils examined.

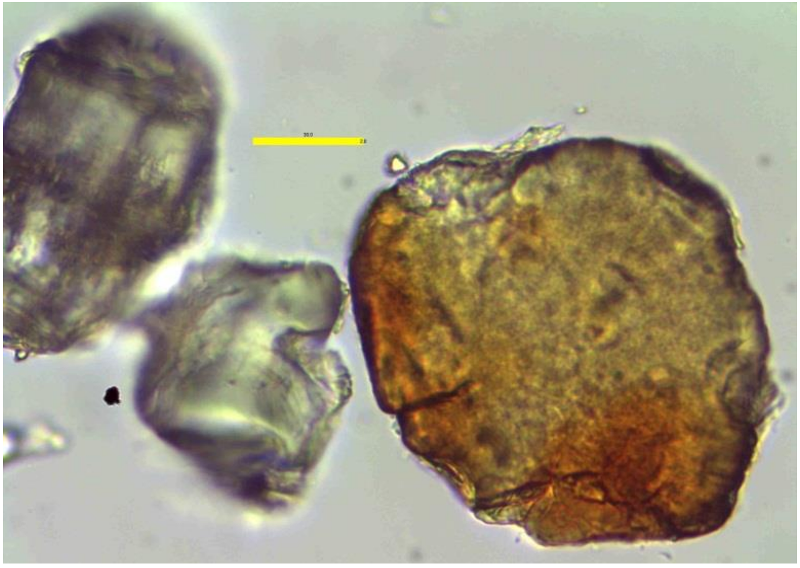


Figure 10.6: Sand layer of Bloemfontein soil.

10.4.2 Coarse silt layer

Figure 10.7 is a photograph from the coarse silt layer of the soil in Figure 10.6.

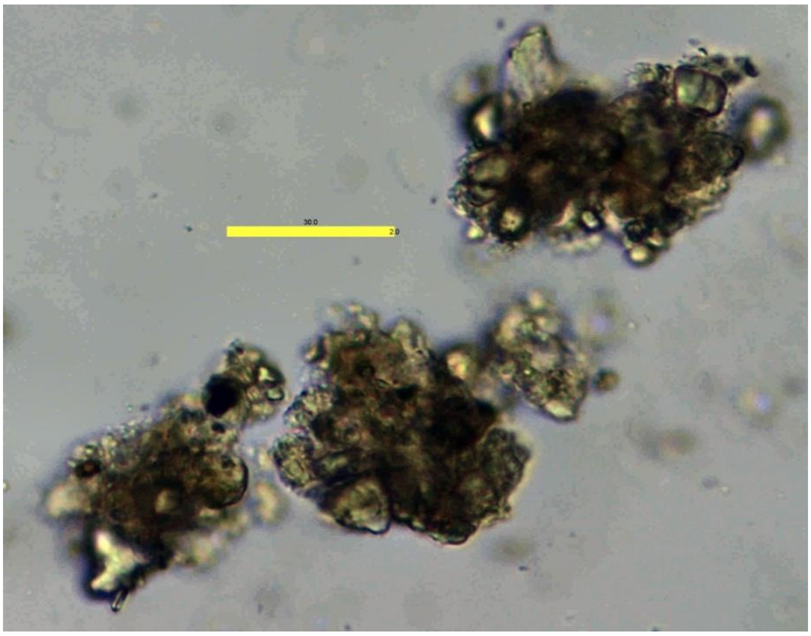


Figure 10.7: Coarse silt layer of Bloemfontein soil (1).

The structures visible are of coarse-silt size, but they appear to contain fine silt sized particles bound together and covered by clay particles. A similar situation is shown in Figure 10-8. Where the largest silt particle (which is only partially visible) could be coarse silt, but the agglomeration is of sand size. The coating obscuring the details of the structure appears to be about half of fine, high CEC/SSA clay and half of larger, low CEC/SSA clay.

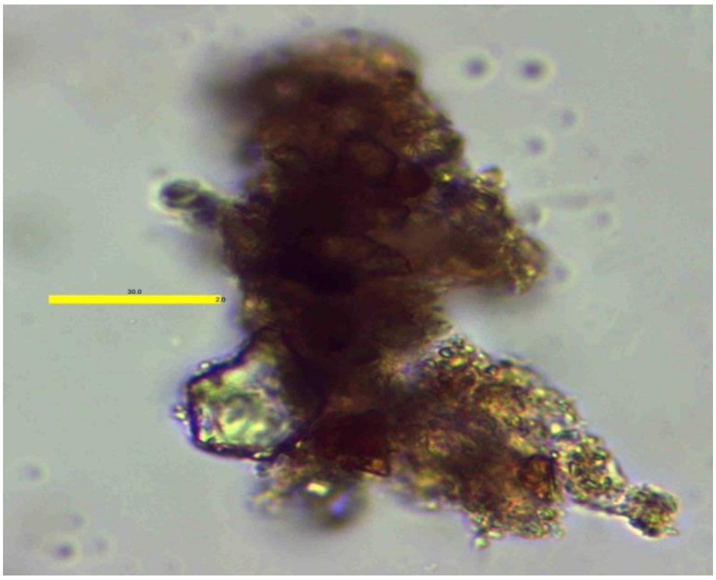


Figure 10.8: Coarse slit layer of Bloemfontein soil (2).

Figure 10.9 is from the coarse silt layer of the Thaba Nchu soil shown in Figure 10.3.

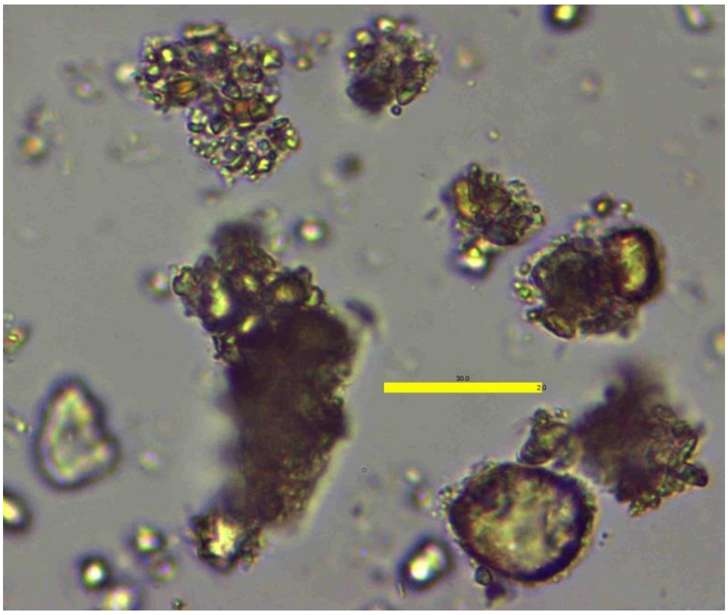


Figure 10.9: Coarse silt layer of Thaba Nchu soil.

There are agglomerations of finer material together with the silt. Again the agglomerations tend to be larger than the clean members of the group. This tends to confirm the impression gained from most of the photographs; that agglomerations precipitate more slowly than single particles of similar size.

10.4.3 Fine silt layer

Figure 10.10 shows the fine silt layer from the Bloemfontein soil of Figures 10.6, 10.7 and 10.8. The agglomerations are again larger than the clean silt particles. Some fine silt particles appear to be fully coated with very fine, deeply stained clay particles. Some of the agglomerations appear to be clay particles with no silt core; the largest agglomerations probably do have silt cores. Genuine fine silt particles do not appear to make up the majority of the material present.

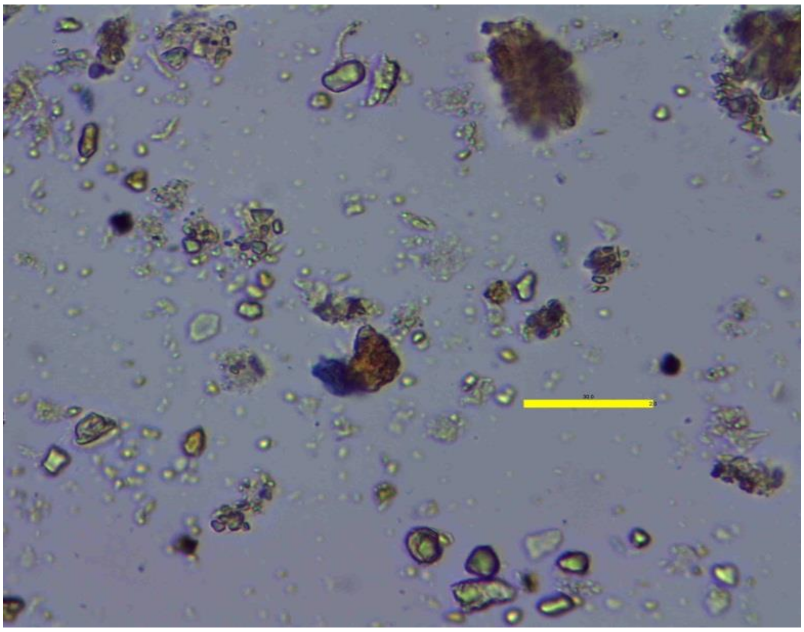


Figure 10.10: Fine silt layer of Bloemfontein soil.

Figure 10.11 is a photograph from the fine silt layer of a soil from Wepener in the Free State. A large part of this layer appears to be made up of agglomerations of clay without silt cores. Many of these agglomerations appear to be tenuous sting-like structures, in some cases up to 100μm long. Observation of their movement under the microscope suggests that their bonds, though flexible enough to allow them to bend easily, are strong enough to hold the structures together robustly. The silt particles present are completely coated by fine, high CEC/SSA clay which is deeply stained by the small quantity of methylene blue added to the suspension. A large proportion of the material appears to be clay.

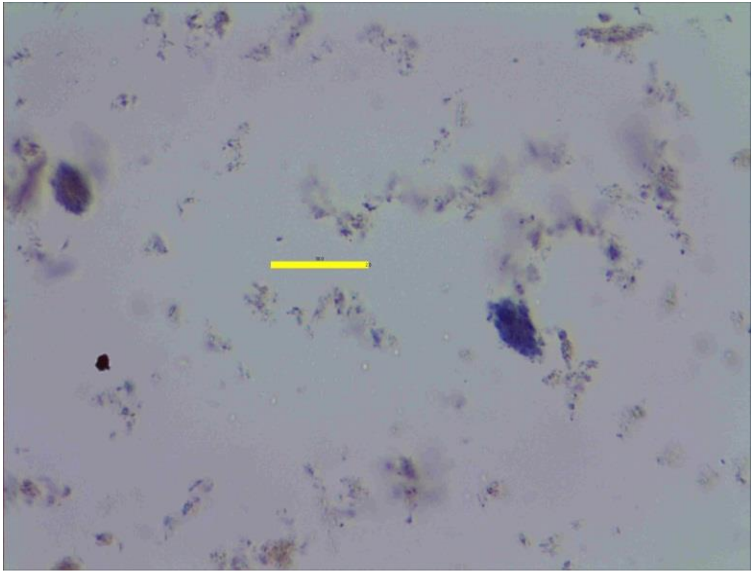


Figure 10.11: Fine silt layer of Wepener soil.

10.5 Conclusion

The investigation tends to confirm the widely held view that hydrometer analysis is not always reliable, particularly for high clay-content soils. A number of the queries raised by Savage (2007) can be seen to be justified. The photographs generally indicate relatively few free clay particles in the sand layer but a noticeable increase in the silt layers. The fine silt layer has generally more free clay particles than the coarser layers (Figures 10.10 and 10.11), suggesting that smaller particles may be more effective in carrying clay particles down with them than large particles. Almost all sand and silt particles were seen to have at least a few attached clay particles. The coating of clay noted in the sand-layer was generally quite thin in comparison with the size of the sand grains and therefore unlikely to make up a large fraction of the material in this layer. Hydrometer analysis might therefore be expected to give an acceptable assessment of the sand fraction in many cases. The silt layers, on the other hand were frequently found to be made up of agglomerations of fine silt and clay, or in some cases, of clay alone. These agglomerations were frequently larger than the bare particles of the host layers, suggesting that the agglomerations settle more slowly than single particles of their aggregate size, but more quickly than the individual small particles of which they are made. In the analysis of high clay content soils, it seems likely that the hydrometer will over-estimate the amount of silt to a considerable extent. Consequently, the clay content will almost always be under-estimated.

CHAPTER 11: THE MOVEMENT OF SOIL MOISTURE UNDER A GOVERNMENT SUBSIDY HOUSE

11.1 Abstract

The South African Government's attempts to provide affordable, subsidised housing for the very poor has suffered from a large number of structural failures, many due to heaving foundations. These houses are particularly susceptible to damage by heaving clay because they are exceptionally light and clay can lift them very easily. Rational design requires knowledge of the pattern of heave which will occur under the foundation. The pattern of heave depends on the pattern of moisture movement. Currently available methods of rational design rely on assumptions about the shape of the mound which will develop due to moisture movement under the foundation. The shape assumed is largely guided by measurements made on test foundations. Instrumentation has been installed under a Government Subsidy house in the Free State and moisture movement is being monitored. The actual pattern of moisture movement observed is substantially different to what is normally assumed and could point to more reliable estimates of the heave which needs to be designed for.

11.2 Introduction

Since 1994 the South African government has built more than 2.68 million subsidised houses throughout the country (Government Communication and Information System 2015). The construction of raft foundations for Government Subsidy housing has become common in areas affected by expansive soils as is the case in much of the Free State and Northern Cape. The intention of raft foundations on active soils is to limit the differential movements of the underlying soils to a level which can be tolerated by the superstructure (Day, 1991). The large number of failures suggests that the problem of providing sufficient stiffness to the foundation has not yet been solved. This impression is reinforced by the observation that many houses are being built on "stock design" rafts which bear only general correspondence with the likely heave potential of the soils involved. There is the impression that there may be inadequate understanding of the actions which need to be designed for. In many cases, when structures have become structurally unsound due to heaving foundations, it is more economical to demolish than to attempt repair. The seriousness of this problem therefore merits a search for a solution. Current raft design relies on assumptions about the shape of the mound or dome which will develop due to moisture movement underneath the foundation. The shape which is assumed is often based on heave measurements on simulated foundations (Pidgeon, 1987; Pidgeon and Pellissier, 1987) or foundations simulated by sheet covers (Fityus *et al.*, 2004; Miller *et al.*, 1995; De Bruijn, 1973). Such test foundations do not take account of two important factors concerning the influence of a building constructed on the foundation – the influence of the building on the temperature regime under the slab and the influence of the building on solar radiation reaching the soil surrounding the raft. Both of these factors can have a profound effect on moisture movement under the foundation. Fityus *et al.* (2004) found that over a period of measurement lasting seven years, temperature appeared to have a greater effect than rainfall on moisture movement and consequent heave of clayey soils at their test site.

11.3 Site Description and Instrumentation

The location of the study area is in Botshabelo section K, which forms part of the Mangaung Metropolitan area and is situated 45° km east of Bloemfontein. Each year many government subsidy houses are built throughout Botshabelo. A significant number of them experience structural distress well short of their design lifetime. Botshabelo is underlain by mudstones, shales and sandstones of the Beaufort Group, with frequent intrusions of dolerite. All of these rocks frequently produce expansive clays when weathered in the semi-arid conditions of the central Free State. The area is therefore likely to be very suitable for the study being undertaken. A Government subsidy house (GSH) was selected based on the soil conditions and the fact that it has a raft foundation of a very common “stock” design. Continuous Logging Soil Moisture (CLSM) probes were installed to measure water content at various depths under the house. Measurements are being taken automatically at hourly intervals. The installation layout is from east to west and north to south in direct alignment with the house.

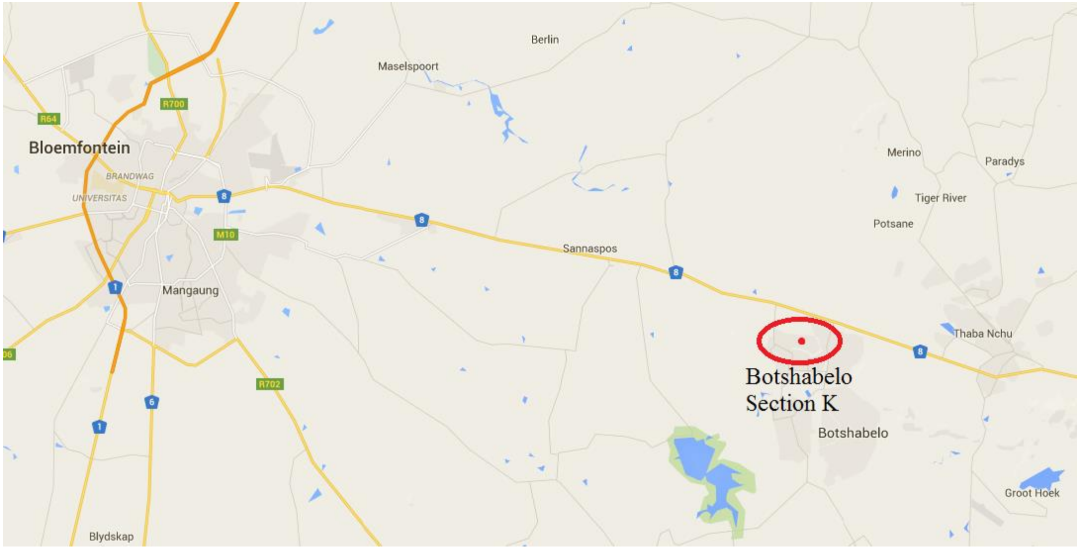


Figure 11.1: Study site: Botshabelo Section K.

The CLSM probes allow measurement of temperature and water content at depths of 150mm, 300mm, 450mm, 600mm, 800mm and 1000mm. The soil profile underneath the house has a thin layer of dark brown clayey sand with a thickness of 150mm. This is underlain by a layer of black transported clay and a layer of olive residual clay. Both clays were assessed by Van der Merwe's method (1964) as having medium expansiveness. Rock is found at a depth of approximately 1.1 meter. The first clay layer is from 150 - 900mm, the second clay layer is from 900 - 1100mm. Figures 11.2 and 11.3 illustrate the layout of the CLSM probes inside and below the house.

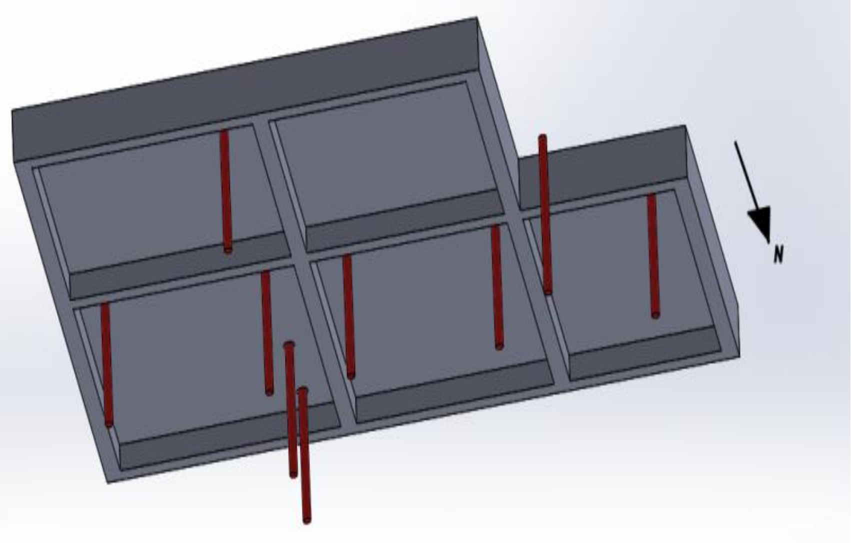


Figure 11.2: 3D layout of CLSM probes.

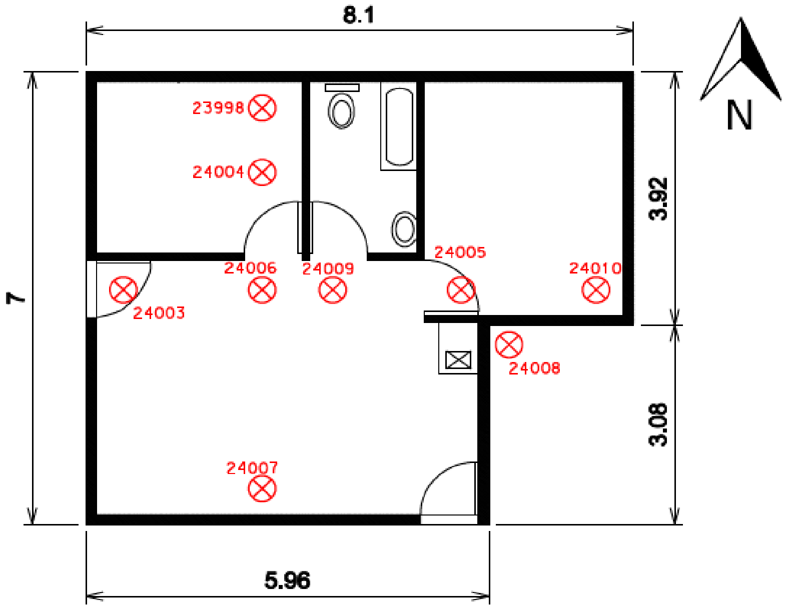


Figure 11.3: CLSM probe layout in raft foundation: dimensions (m).

11.4 Results and Discussion

11.4.1 Moisture content seasonal change models

Results of hourly measurement of water content taken over a period of two years were analysed. The following figures illustrate moisture content values measured in summer and winter at each depth recorded by the probes. These graphical representations of the data were produced using the programme “3D field”. The scale was selected as a best fit for each specific model. Areas of dark blue have higher moisture content. A colour scale is shown next to each figure. From the figures it can be observed that the north side consistently shows the lowest moisture content compared to the rest of the house. This is almost certainly due to the fact that solar energy reaching the ground on the northern side is more intense than that reaching the other sides of the building. When rainfall occurs the north side is prone to rapid moisture change. This is probably due to the fact that the dry, cracked soil allows immediate access to rainwater. At the north side water contents range from less than 5% to 25%. In contrast, water content near the south east re-entrant corner remains within the range 31% to 34% at all depths in both summer and winter. This re-entrant corner never receives any significant solar radiation

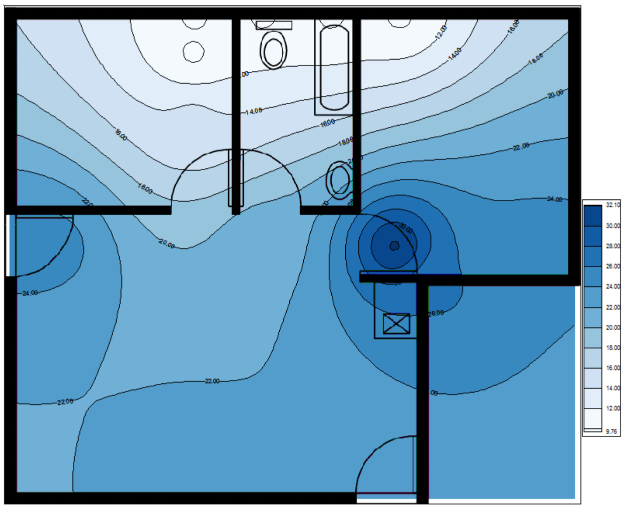


Figure 11.4: Summer: 150mm w = 10%-32%.

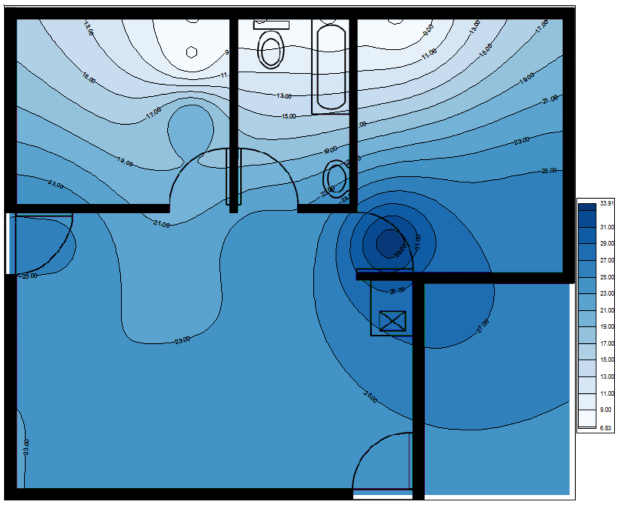


Figure 11.5: Summer: 300mm w = 7%-33%.

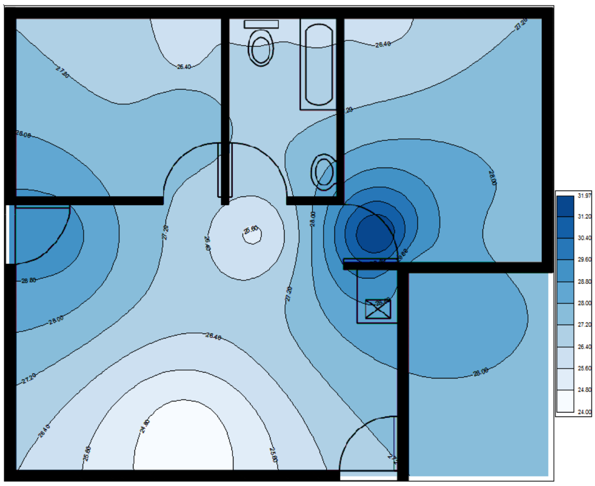


Figure 11.6: Summer: 450mm w = 24%-33%.

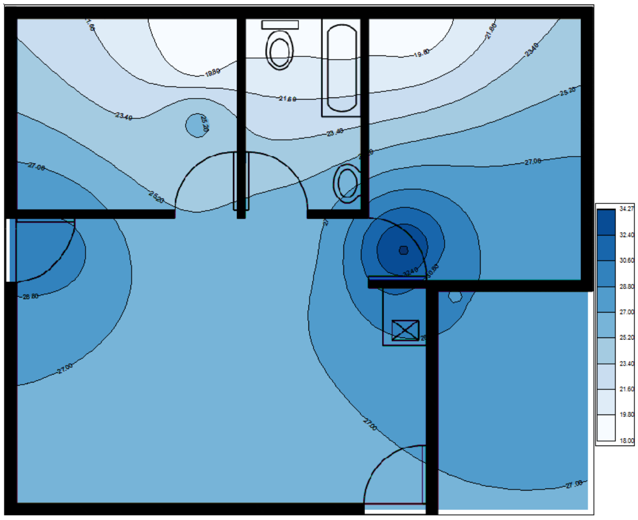


Figure 11.7: Summer: 600mm w = 20%-35%.

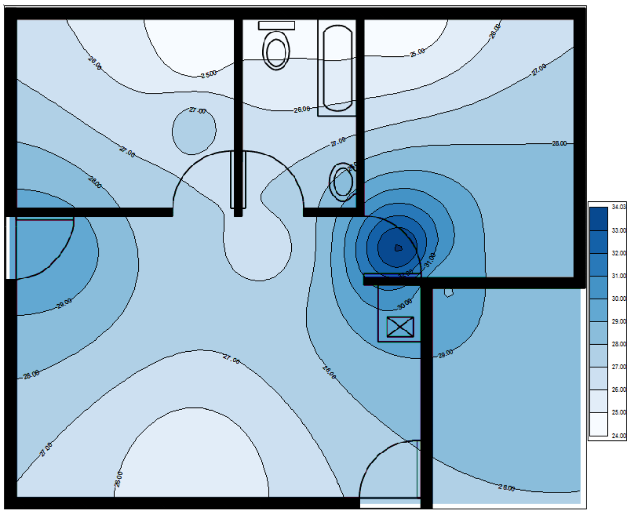


Figure 11.8: Summer: 800mm w = 25%-35%.

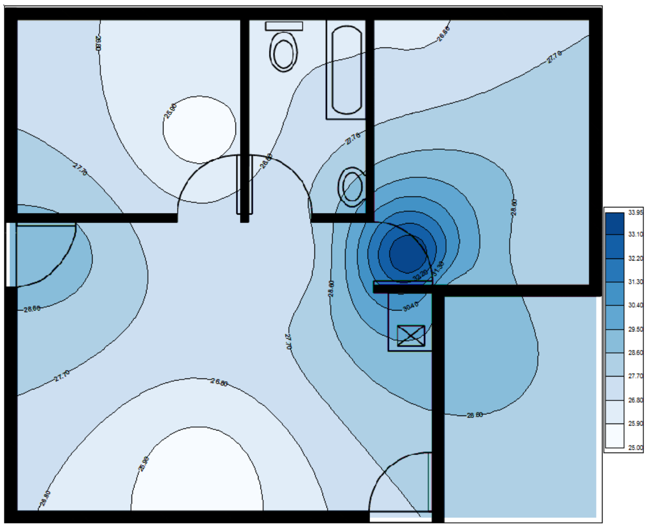


Figure 11.9: Summer 1000mm w = 25%-34%.

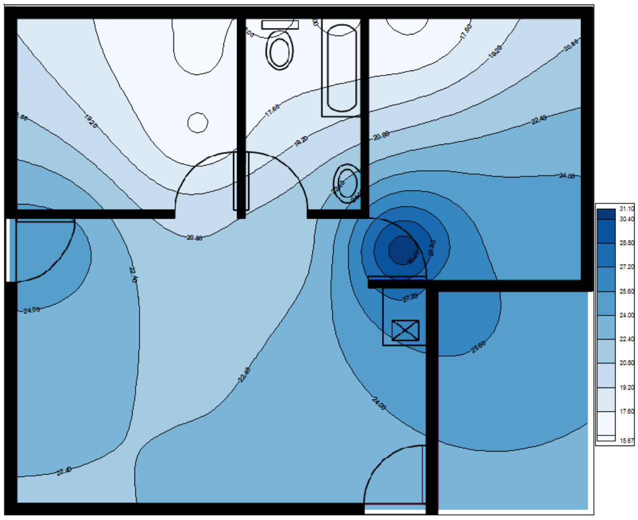


Figure 11.10: Winter: 150mm w = 15%-31%.

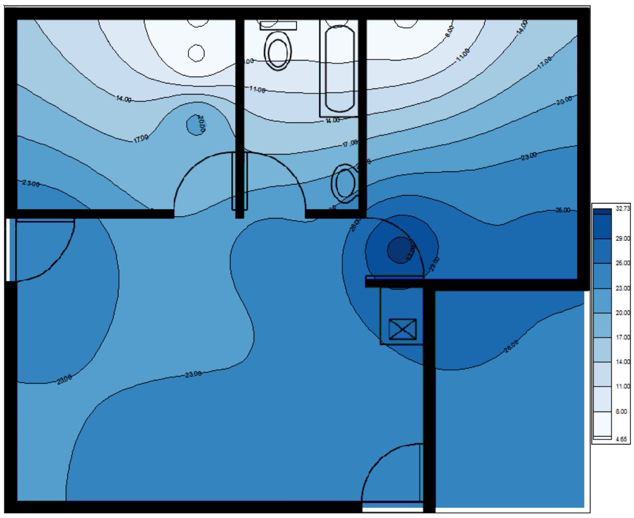


Figure 11.11: Winter: 300mm w = 5%-33%.

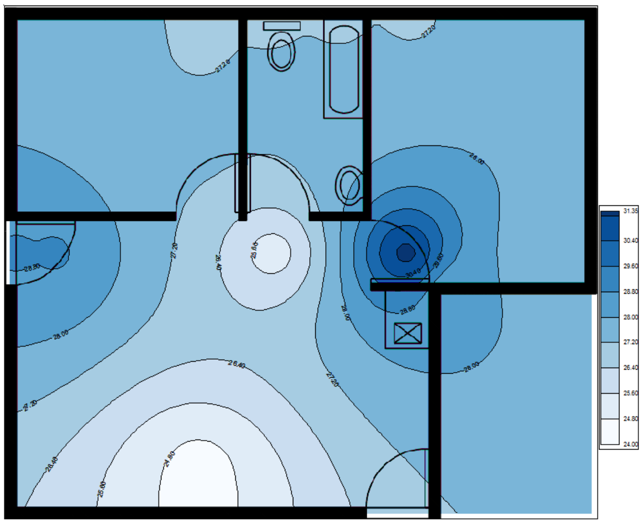


Figure 11.12: Winter: 450mm w = 24%-31%.

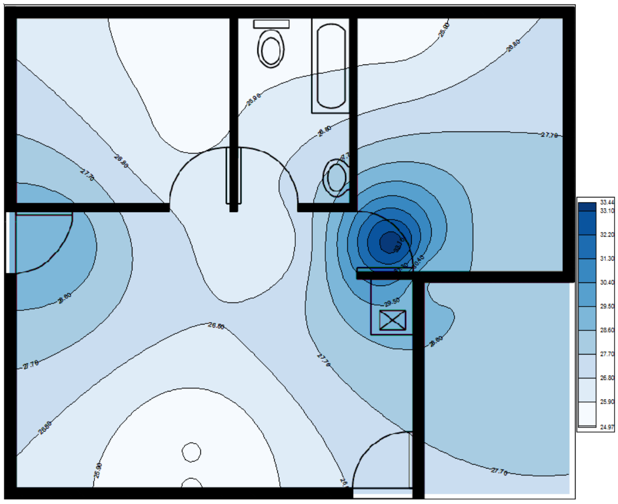


Figure 11.13: Winter: 600mm w = 25%-33%.

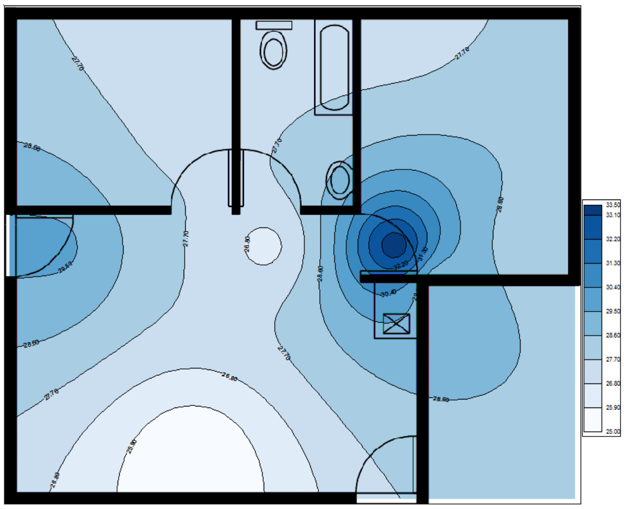


Figure 11.14: Winter: 800mm w = 25%-35%.

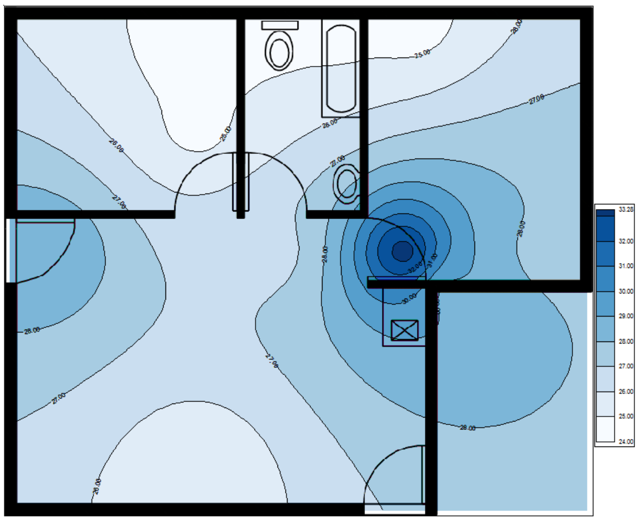


Figure 11.15: Winter: 1000mm w = 24%-33%.

The architecture of the building guarantees that this re-entrant corner is practically always in the shadow of the building. Its moisture content did not change by more than 3% in any of the different layers of soil. Figure 11.16 illustrates the seasonal changes from March 2014 to February 2015 of the north side (CLSM probe 23998) that consistently has the lowest moisture content compared to the rest of the house. While Figure 17 shows the south east side (CLSM probe 24005) seasonal moisture changes for the same time that has the most consistently high moisture content compared to the rest of the house.

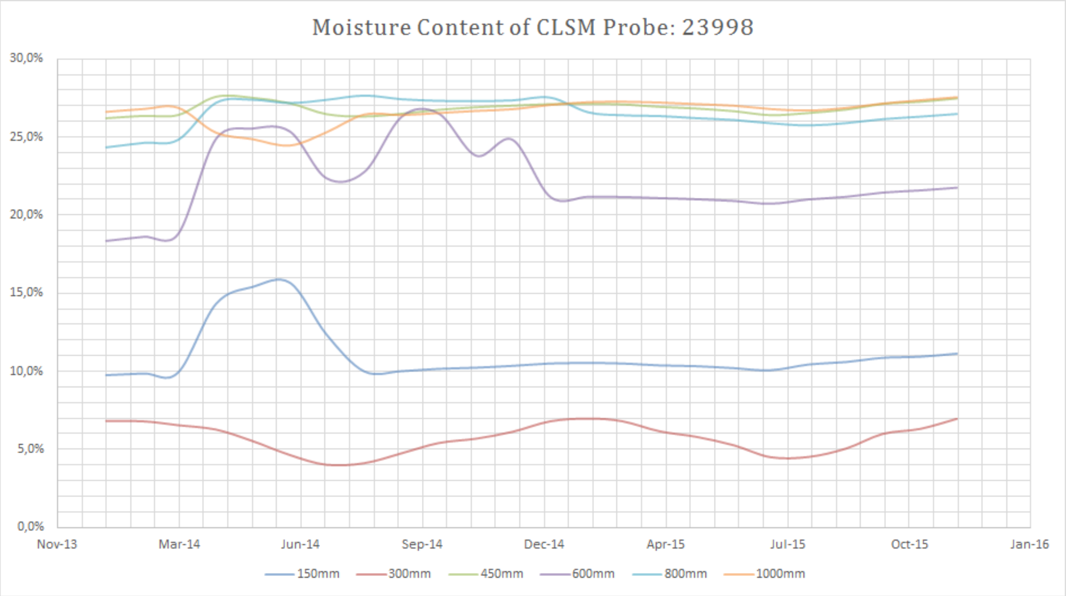


Figure 11.16: Probe 23998 moisture content 5% to 25%.

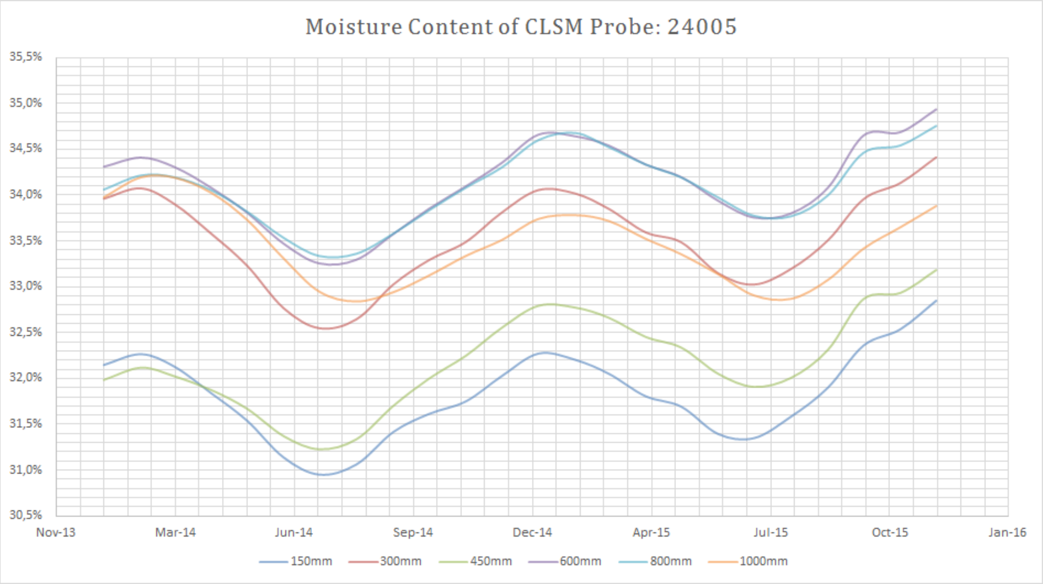


Figure 11.17: Probe 24005 moisture content 31% to 34%.

The observed moisture variations suggest that an approximately symmetrical dome-shaped heave pattern, centred roughly on the centre of the foundation, as found by Pidgeon (1987), Pidgeon and Pellissier (1987), Fityus *et al.* (2004) etc. in simulated foundation tests is not likely to develop. Simulated foundation tests take no account of the influence of shadows cast by the building. It appears that the south east re-entrant corner will remain in a high water-content, highly expanded state (varying by only about 3%) whereas the northern wall will assume a lower water-content, less expanded state with far more variability. This tends to confirm the findings of Fityus *et al.* (2004) that temperature can have a more pronounced effect than rainfall on swelling patterns in active soils.

11.4.2 Further considerations

It has been noted (Fityus *et al.*, 2004) that heave predictions commonly ignore the effect of the loads applied by the building, though this may not be true in all cases. The load applied to the soil directly under a raft foundation by a completed GSH house is approximately 10 kPa. The three layers of material under the instrumented foundation were tested to assess their ability to heave against constraining pressures. The clayey sand of the thin upper layer was found to be able to expand against a pressure of 11 kPa up to a water content of slightly above 28%. The transported clay, of the second layer, was found to be able to expand against a pressure of 14 kPa up to a water content of 35% and the residual clay of the third layer could expand against a pressure of 14 kPa up to a water content exceeding 45%. These pressures were measured by apparatus under development by the CUT Soil Mechanics Research Group. At lower water content far higher pressures can develop. Assuming a unit weight of 2 000 kg/m^3 for all layers, the maximum pressure exerted at the base of the lowest layer would be approximately 21 kPa. It would therefore appear that the loads applied by overburden and structure might cause a restriction to heave in the thin upper layer in the wettest area only (around the south-east re-entrant corner). The lowest layer of clay would probably experience no restriction to heave, and the lower part of the middle layer would probably be restrained in the area of maximum water content.

11.5 Conclusion

The measurements of moisture content under a very common type of Government subsidy house built on clayey soil suggest that currently accepted patterns of heave are unlikely to provide good guidance for foundation design. The instrumentation used in this investigation has proved itself convenient and reliable. It is hoped that it will be possible to instrument several other light structures in order to work towards a general modelling procedure. This should enable reliable predictions of the moisture conditions which need to be designed for in the general case. This in turn should allow reliable and economic design of a wide range of raft foundations with the prospect of fewer failures. In the meanwhile, by applying the findings of this investigation it should be possible to at least obtain better estimates of moisture pattern development and to estimate more realistic heave patterns than are currently being employed.

CHAPTER 12: RAPID SHRINKAGE CURVE GENERATION USING SMALL SAMPLES AND HIGHLY VOLATILE NON-POLAR LIQUIDS

12.1 Abstract

The application of unsaturated soil mechanics would benefit from quicker and simpler procedures for producing essential tools including the shrinkage curve. Determination of the shrinkage curve for expansive clay usually takes several days. A method is presented for producing accurate and consistent shrinkage curves from full saturation to complete desiccation in a few hours using highly volatile non-polar liquids and samples of small size.

Keywords: Rapid soils testing, shrinkage, soil shrinkage curves, unsaturated soil mechanics.

12.2 Introduction

The development of unsaturated soil mechanics over the last few decades has provided the engineer with powerful tools for coping with a wide range of problems. The use of these methods by the engineering community has been somewhat limited, particularly for smaller projects. Fredlund (2006) noted “Greatly increased costs and time were required for the testing of unsaturated soils. As well, laboratory equipment for measuring unsaturated soil properties has proven to be technically demanding and quite complex to operate.” In the same paper, he emphasized this problem by noting “One of the ‘roadblocks’ standing in the way of implementation of unsaturated soil mechanics has been the excessive cost and demanding laboratory testing techniques associated with the direct experimental assessment of unsaturated soil properties.”

To overcome the problem that some of the tests take several weeks (and cost several thousand dollars) to perform methods have been devised for estimation from the results of quicker, easier, cheaper tests such as particle size analysis, or from "data-base mining". Such estimates are less reliable than direct measurement, and they are suitable primarily for preliminary analysis. There is therefore a need to produce essential unsaturated soil mechanics parameters such as the soil water characteristic curve and the shrinkage curve cheaply and accurately if the advances in unsaturated soil mechanics are to be widely adopted by the majority of Engineers. This paper deals with a technique for the production of the shrinkage curve with good accuracy in one day, even for clays with high expansive potential.

12.3 The Soil Shrinkage Curve

Soils which shrink and swell do not have a "rigid" structure, i.e. they do not have stable relations between their solid and pore volumes and experience significant bulk density variations during water content variation Taboada (2003), Peng, and Horn (2005) noted that understanding and prediction of water transport processes in non-rigid soils requires a knowledge of the dynamics of soil shrinkage. Chertkov (2003) noted that the shrinkage curve is one of the physical characteristics of soils and it is necessary for the prediction of soil behaviour when a soil interacts with water. The shrinkage curve is then, an important tool for dealing with shrinking/swelling soils. In principle, it demonstrates a relationship between volume and water content. A typical representation of the shrinkage curve from Fredlund (1999) is shown in Figure 12.1. Unfortunately, there is no universally accepted way to illustrate this dependence. Among soil scientists, two popular ways to show this dependence are a) specific volume (volume per unit mass) versus water content, and b) void ratio (volume of voids: volume of solids) versus moisture ratio (volume of water: volume of solids). Among engineers, common ways to show this dependence are void ratio versus water content and specific volume versus water content. Even more unfortunately, there is no consistency in the use of the term "water content"; sometimes it refers to volumetric, sometimes gravimetric measure. In this paper, "water content" is always gravimetric and the pattern of Figure 12.1 is followed. This has the advantage that specific volume has a definite base line value (the inverse of density of the soil solids) and a definite reference line (the zero air voids line) which allows immediate and convenient comparisons between different soils.

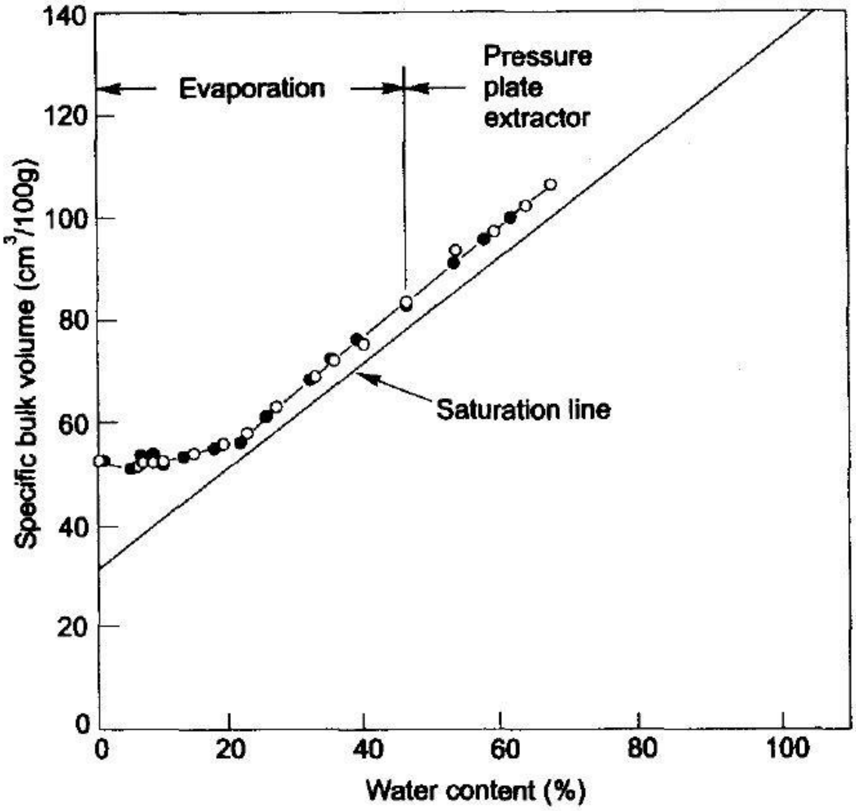


Figure 12.1: Shrinkage curve for Regina Clay (Fredlund, DG.1999).

12.4 Review of Currently Popular Methods

Shrinkage curves have been produced by soil scientist since before engineers appear to have appreciated their value. A major concern of soil scientists is agriculture and hence with soils containing enough moisture for crop production. Soil scientists have, for many years, been producing shrinkage curves for the wetter part of the range by immersion in paraffin (kerosene), toluene or other non-polar liquids. Figure 12.2 shows a typical arrangement (Taboada, 2003) in which a container of non-polar liquid is placed on a balance and a sample is immersed by hanging from a support. When a soil is saturated it does not absorb non-polar liquids and the method gives a shrinkage curve very conveniently up to the air entry point. When a soil is drier than the air-entry point, however, it does absorb non-polar liquids.

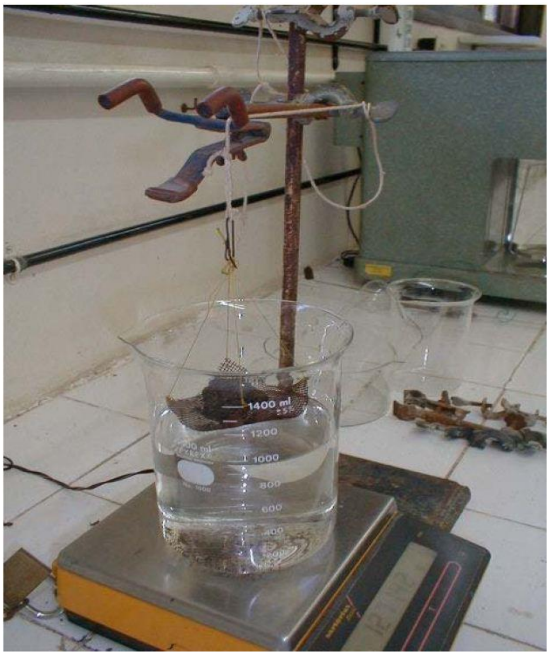


Figure 12.2: Container of non-polar liquid.

Unsaturated soil mechanics deals with soils drier than the air-entry point and to overcome this problem the technique of coating soil samples with Saran resin dissolved in methyl ethyl ketone was developed (Brasher *et al.*,1966), and became widely accepted as the standard method for shrinkage curve generation. The essential feature of the Saran resin coating is impermeability to liquid water (allowing volume determination by immersion), combined with permeability to water vapour (allowing the sample to undergo a drying or wetting regime). While this method is widespread it suffers from the disadvantages that about six weeks are needed to produce a shrinkage curve and methyl ethyl ketone is hazardous to human health and living environment. The test should be performed under a fume hood exhausting above the level of the local building. Alternative, less hazardous encasement materials, have been sought (Krosley *et al.*, 2003) apparently with some degree of success.

It is also possible to determine the volume of a soil clod by coating with a totally impermeable material before immersing in liquid. Paraffin-wax is the standard coating material, but it suffers from a number of disadvantages. The temperature of melting, and the high latent heat of fusion cause considerable heat input to the sample being coated. There is usually a significant expulsion of air and/or water vapour from any sample which is not completely desiccated. This leads to small blow-holes in the coating. These may be so small that they are not easily seen and are only detected when the sample is found to gain weight on immersion. Paraffin-wax also contracts significantly on solidification, and tension cracking of the coating may occur, particularly in cool conditions.

The production of a shrinkage curve by wax coating is not very convenient. An individual sample must be prepared for each data point on the curve. Due to the difficulty of ensuring a completely waterproof sample (which is confirmed only on immersion) more than one sample may be required for each data point. Moisture content determined by comparison with an initially similar sample may be questionable due to the expulsion of water vapour during immersion in the hot wax. Determination of moisture content after immersion may be problematic due to the difficulty of complete removal of the coating in a sufficiently short time to prevent significant evaporation or absorption of moisture from the air. Cornelis, *et al.* (2006) noted an advantage of this method in that samples for testing can be broken from clods along lines of cracking for specimens in the dryer range of the shrinkage curve, eliminating the inclusion of voids from the test sample. This is not normally considered for clod-test specimens, and the results of these tests may be significantly affected by included voids.

Another approach to the production of shrinkage curves is the direct measurement of dimension changes in a sample during drying. This requires a large enough sample to allow acceptable accuracy in dimension measurements. This in turn requires a considerable time for evaporation, particularly for clays, which have low permeability both to water and to water vapour. Bensallam *et al.* (2012) determined shrinkage curves using such a method. Their apparatus allowed shrinkage curve readings to be taken at 6 hourly intervals, the final stage requiring three days of oven drying. The total time required to produce a shrinkage curve was not stated.

A rather simple method of shrinkage determination was devised by Tariq and Durnford, (1993). A soil sample is enclosed in a normal rubber balloon. The balloon provides a waterproof coating; its flexibility allows volume determination.

Drying of the soil sample is achieved by pumping air into the balloon between mass and volume readings. Cornelis *et al.* (2006) reported favourably on this method. They noted that using an air pump delivering 50 to 100 L per hour they were able to take measurements at 12h intervals. Shrinkage curve production took two weeks. They noted that it should be possible to reduce the time by using a more powerful pump.

Another simple and inexpensive method has been investigated by Cerato and Lutenegger (2006). Shrinkage trough is filled as for a normal linear shrinkage determination. The sample is dried progressively and measurements of linear dimensions taken at suitable intervals. The moisture content is determined by weighing the trough and its contents. The procedure has been found to give good results for determining the shrinkage limit, and gives a fair indication of the shape of the shrinkage curve, but these curves tend to show considerable variability between individual samples of the same material. This may be due to two problems. Firstly, expansive clay samples arch and shatter to an unpredictable extent even when dried slowly; this makes linear measurement uncertain. Secondly it is difficult to exclude all voids when filling the shrinkage trough; this may lead to uncertainties in the estimation of volume. A shrinkage curve can be obtained in about one week by this method.

12.5 General Consideration for this Investigation

Several alternative coatings were tried following the methods of Krosley *et al.* (2003). It was soon realised that any such coating, which can give impermeability to water, but permeability to water vapour, cannot give a shrinkage curve in a short period of time. The limiting factor for a rapid shrinkage curve is rapid drying. This implies a small sample and all-round exposure to the air. To maintain acceptable accuracy this in turn demands a sensitive balance. Sensitive balances have a relatively small measuring capacity, which necessitates modification of the normal approach to volume determination by Archimedes principle. Dispensing with a water-tight coating requires that volume determination be done with a non-polar liquid. Immersion at moisture contents below the air entry point results in liquid entering the sample. Volume determination must therefore account for absorbed liquid.

12.6 Materials and Methods

An analytical balance capable of reading to 0.00001g was used. To utilise this accuracy a balance requires a very stable operating environment free from air currents and vibration as well as temperature and humidity fluctuations. Relatively few measurements were taken at this accuracy. When set to read to 0.0001g the balance performed well in normal laboratory conditions. Most of the measurements were done at this sensitivity, which is considered ideal for shrinkage curve production. The balance was mounted on a small table closed on three sides to give protection from air currents. A hole in the centre of the table top allowed a hook to hang from an attachment point in the balance to the shielded area below. Cradles made from 1mm galvanised wire were used to carry the soil samples. The cradles and their samples were suspended from the hook attached to the balance (Figure 12.3 (a)). A manual lifting device was made to raise a container of liquid to immerse the suspended sample. To minimise disturbance of the fluid and allow rapid stabilisation of the balance reading, the lifting must be smooth and vertical. It should also always be to the same height. A rubber stopper was provided against which the lifting handle was brought to a halt in each test (Figure 12.3(b)).

Figure 12.3: Submerged sample.

Figure 12.4 shows the graph produced by balance software of weight (g) against time (s) for a 2.1879g cradle and 3.3245g dry-mass sample at water content 27.0% (above air entry). The reading stabilised to a constant value of 2.9462g about 5 seconds after immersion. The vertical lines mark immersion and extraction.

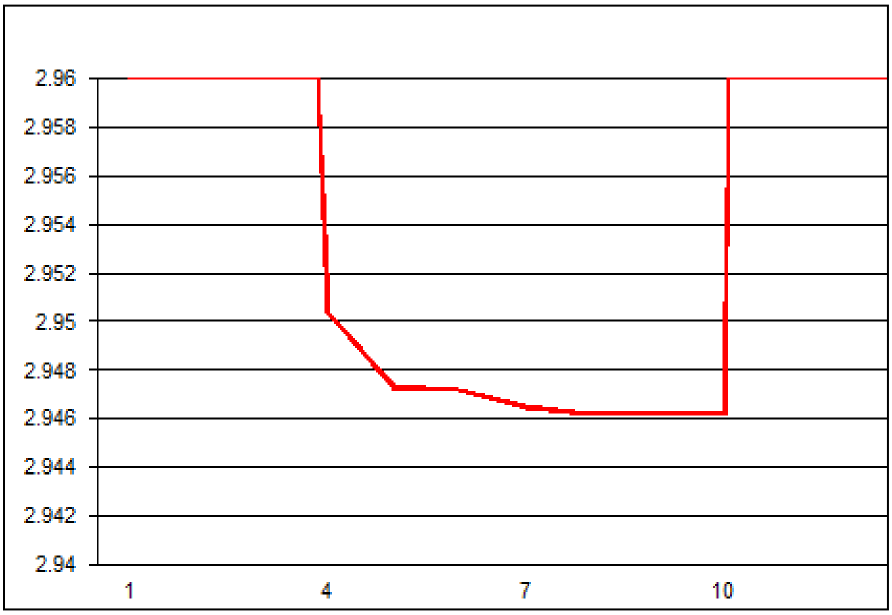


Figure 12.4: Graph of weight (g) against time (s).

Some turbulence is inevitable however smooth the immersion of the sample, and stable readings can only be obtained when the turbulence has largely dissipated. It was found that a time of about 5 to 10 seconds was usually sufficient for readings to become stable when the sensitivity of the balance was set to 0.0001g and saturated samples were used. A major problem with absorption of fluid by the sample is its removal before the next mass and volume determination can be made. This necessitates a very volatile liquid. A number of volatile liquids were considered. Some were discounted because of health hazard; others were found to have damaging effects on soil samples. Two were chosen as suitable for the purpose: chloroform and benzene (petroleum ether). Both have the advantages of reasonable price and ready availability. Both are widely used as solvents. Chloroform is also commonly used as an adhesive for plexi-glass (Perspex). Chloroform has the advantage of posing no fire hazard – it is non-inflammable, but it has the disadvantage that it will produce toxic gasses when exposed to fire. Benzene is highly inflammable and poses a significant fire risk. Chloroform has the disadvantage that it has anaesthetic properties and could cause lack of concentration or worse if inhaled at high enough concentration. No such problems were noted in several months of use for shrinkage curve production. Chloroform appears to have two major advantages. It is more volatile and its density is far greater (more than double). This means that for any sample size the time between data readings can be shorter, and for any required degree of accuracy of volume determination the sample size can be smaller.

Evaporation tests were performed by saturating filter papers in various non-polar fluids and monitoring evaporation by weighing at five second intervals. Two liquids used in soil-science tests, paraffin (kerosene) and toluene, were tested in addition to Chloroform and Benzene. Figure 12.5 shows the fraction of original mass remaining over time. Two curves were produced for each liquid. The exact shape of each curve depends on varying factors including temperature, humidity and local air movement, but the trends are evident.

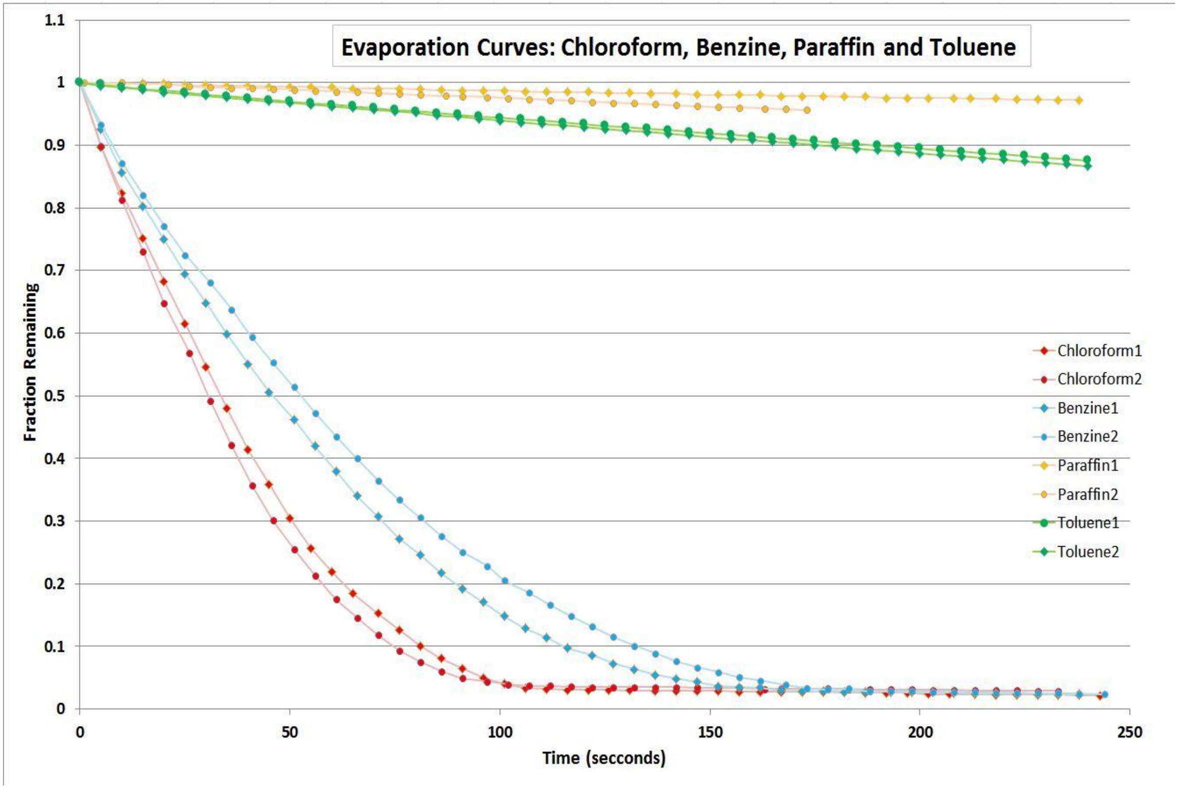


Figure 12.5: Normalised evaporation curves.

Chloroform reaches its residual value in about 60% of the time required for benzene and in a very small fraction of that required for paraffin or toluene. A point of note is that a small residual (of the order 2% of the initially absorbed mass for both liquids) remained held by the filter paper. This residual can be expelled by oven heating. Control filter papers were allowed to reach equilibrium with prevailing air conditions during the evaporation tests. The filter paper used (Whatman's 42) held an equilibrium water content of approximately 7.4% by mass at the prevailing temperature and humidity at the time of the experiment. The retained content of chloroform and benzene at the same conditions were approximately 1.0% and 1.5% by mass of filter paper respectively. Since soils may retain comparable water content in similar temperature and humidity conditions it can be expected that comparable percentages of these liquids may be retained by desiccated soils after immersion. The normalised evaporation curves from Whatman's 42 filter paper for four non-polar fluids subject to air drying is shown in the figure. Evaporation tests were performed on several clayey soils, with PI ranging from 7 to 42, in order to assess evaporation times and residual retention.

Figure 12.6 shows drying curves for three samples of a saturated clay soil. The weight of each of the three samples was approximately 5g.

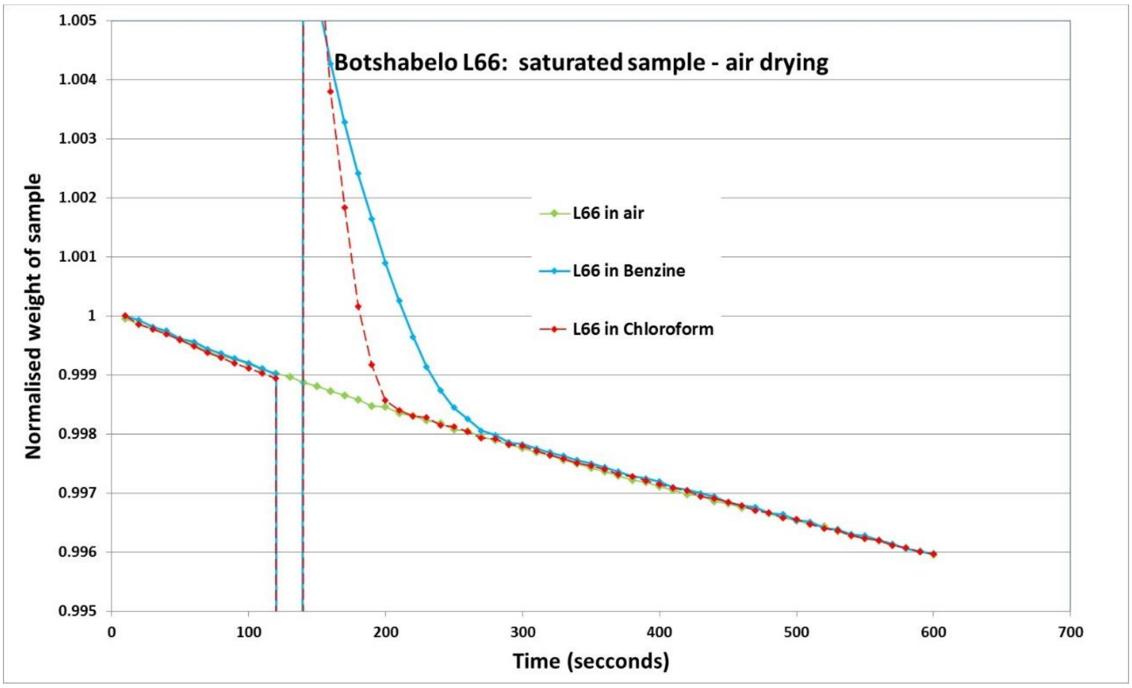


Figure 12.6: Normalised drying curves of 3 samples of a saturated clay soil. Weights have been normalised and superimposed for meaningful comparison. One sample was simply air-dried, the other two were immersed for 20 seconds commencing 120 seconds after the start of the test. The sample immersed in chloroform took approximately 3 minutes to reach the air drying curve, the sample immersed in benzene took approximately 5 minutes. Water content remained above air entry level throughout the tests. For dry samples, liquid is absorbed into the soil on immersion. Evaporation from the fabric of the soil takes longer than from the surface only and some of the liquid is retained in the sample by capillary suction.

Figure 12.7 shows curves of gravimetric moisture content against time for samples of a clay soil on exposure to the air after oven drying. Four samples with mass of approximately 2g each were oven dried. Two were then exposed to the air and weighed at 10 second intervals. One of the samples was immersed in chloroform for 30 seconds, 40 seconds after the start of the test. The other two samples were treated in the same way but with immersion in benzene. Soil samples absorb moisture from the air when removed from the oven; the times taken for the samples to cool and be transferred to the scale were a little different for each pair of samples, so the starting moisture contents are not identical.

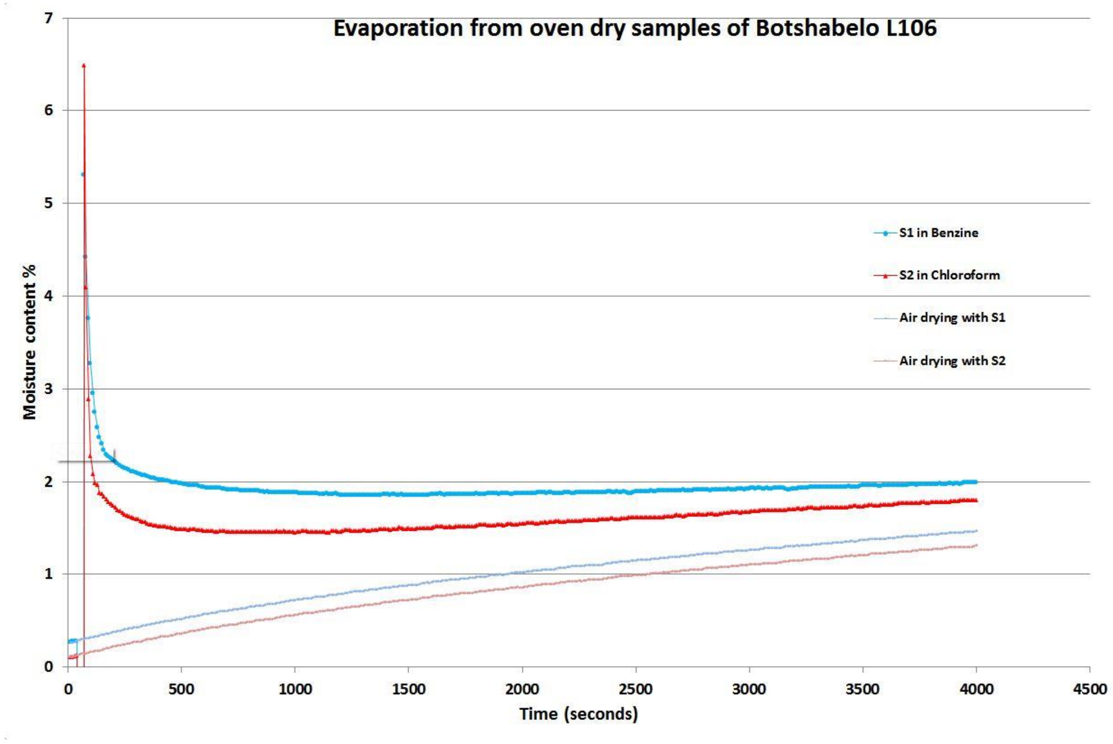


Figure 12.7: Moisture content: Time(s).

The form of the curves is similar and similar conclusions can be drawn. The curves for immersed and non-immersed samples do not meet, but do tend to become parallel. This was as expected. The gap between the final parallel curves is the retained non-polar liquid content. The chloroform-dipped curve becomes parallel to the un-dipped curve after about 50 minutes. The curve for benzene is not quite parallel after 70 minutes. Both curves tend to a retained liquid content of approximately 0.5% of the weight of the sample. 1.0% retention was reached after about 12 minutes following immersion in chloroform and after about 23 minutes following immersion in benzene. A point to note is that chloroform has more than twice the density of benzene. A similar mass retained implies that more than twice the volume of benzene is retained compared to that of chloroform. Unsaturated samples draw in fluid at a rate dependent on their state of desiccation. Sufficient time of immersion is necessary to establish a curve suitable for extrapolation back to the time of immersion. This extrapolation allows an estimate to be made of the immersed weight of the sample before the ingress of fluid. It was found that an immersion of 20 to 30 seconds was usually required to establish an absorption curve for this purpose.

Figure 12.8 shows two curves given by proprietary software immediately after a series of readings. They are for the same active clay sample as Figure 12.4, at moisture contents below air entry, viz. (a) 8.14% (top) and (b) 2.42% (bottom).

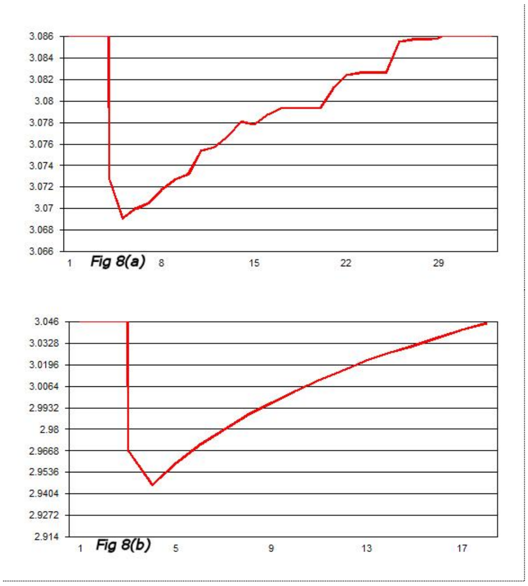


Figure 12.8: Proprietary balance software curves (balance reading: time)

Extrapolation by eye suggests a balance reading of approximately 3.069 at immersion for (a) and 2.927 at immersion for (b). Balance output can be exported to a spread-sheet if more precise extrapolation is required. Figure 12.9 shows the same results plotted in a spread-sheet, with extrapolation curves superimposed

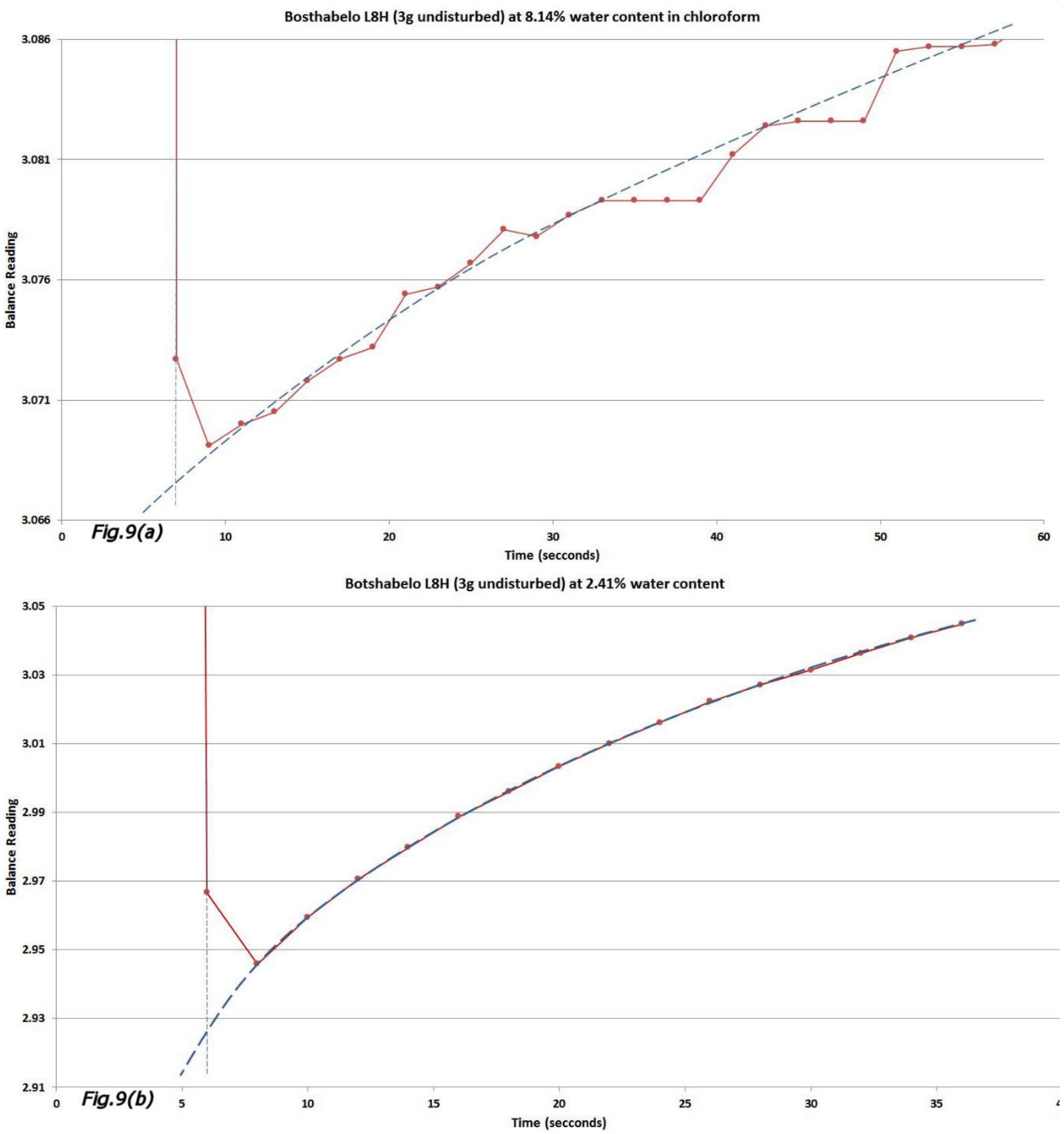


Figure 12.9: Results plotted with extrapolation curves superimposed.

Extrapolation for Figure 12.9 (a) suggests balance reading 3.068 at immersion and for (b) 2.926. These values are very close to those obtained by inspection of curves (available immediately after recording the balance readings) shown in Figure 12.8. The steepness of the curve increases and probable accuracy of the extrapolation decreases as the sample approaches complete desiccation. Immediately after oven drying the probable error appears to be of the order of 1% of the sample mass for small samples (1g-2g). In all cases of shrinkage curves which follow, extrapolation was done by inspection of the immediately available curves given by the balance software. Occasional spot checks were made by transfer to a spread-sheet.

Shrinkage curves of three samples of an active clay (PI 36) are shown in Figure 12.10 (a), (b) and (c). Chloroform was used as the liquid for volume determination. Air drying at approximately 23°C and 35% relative humidity required 8 hours for completion of the curve. Figure 12.10 (a) shows the curve with no allowance for liquid retention, Figure 12.10 (b) shows the shrinkage curve allowing for 1% chloroform retention below the air entry point. Since the time between immersions was in the region of 30 minutes this is probably a generous allowance. It shows a small change from the “uncorrected” curve. Since the correction is for moisture content only, the effect of the adjustment is to move data points in the direction of water content rather than in the direction of specific volume. The curves close to the shrinkage limit are therefore altered very little and the most noticeable change is to give a transition curve with slightly greater radius of curvature.

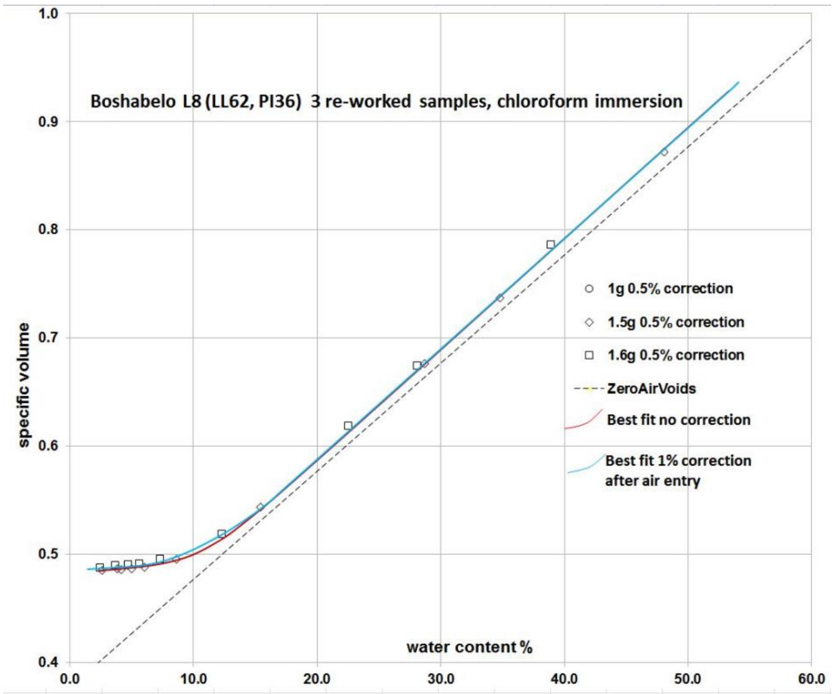


Figure 12.10 (a): No correction for liquid retention.

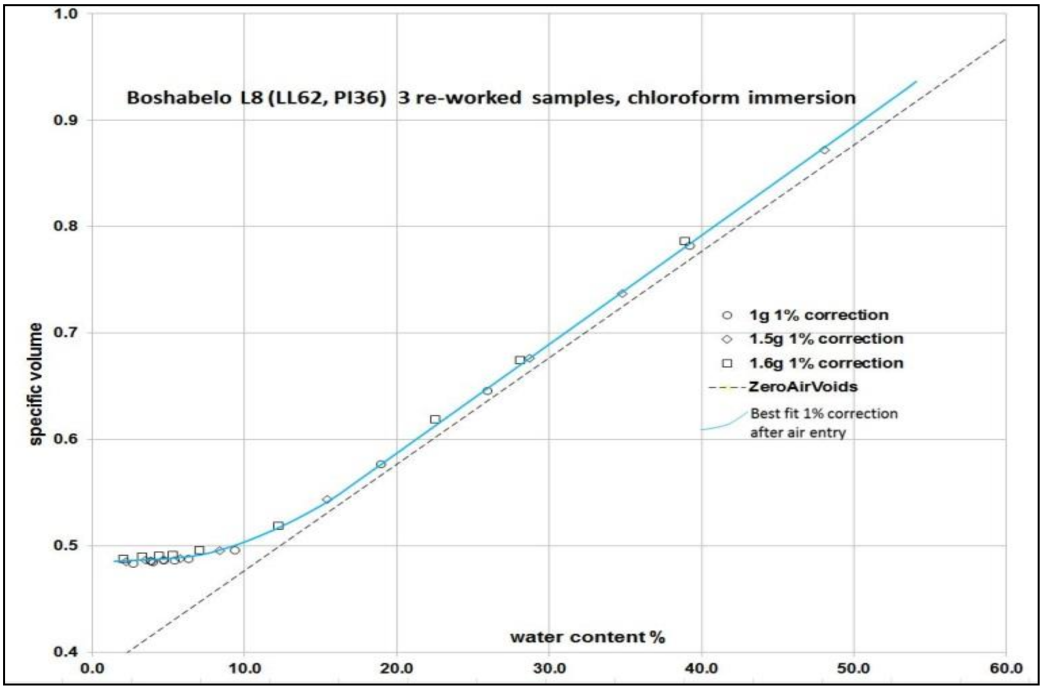


Figure 12.10 (b): Allowance made for 1% liquid retention.

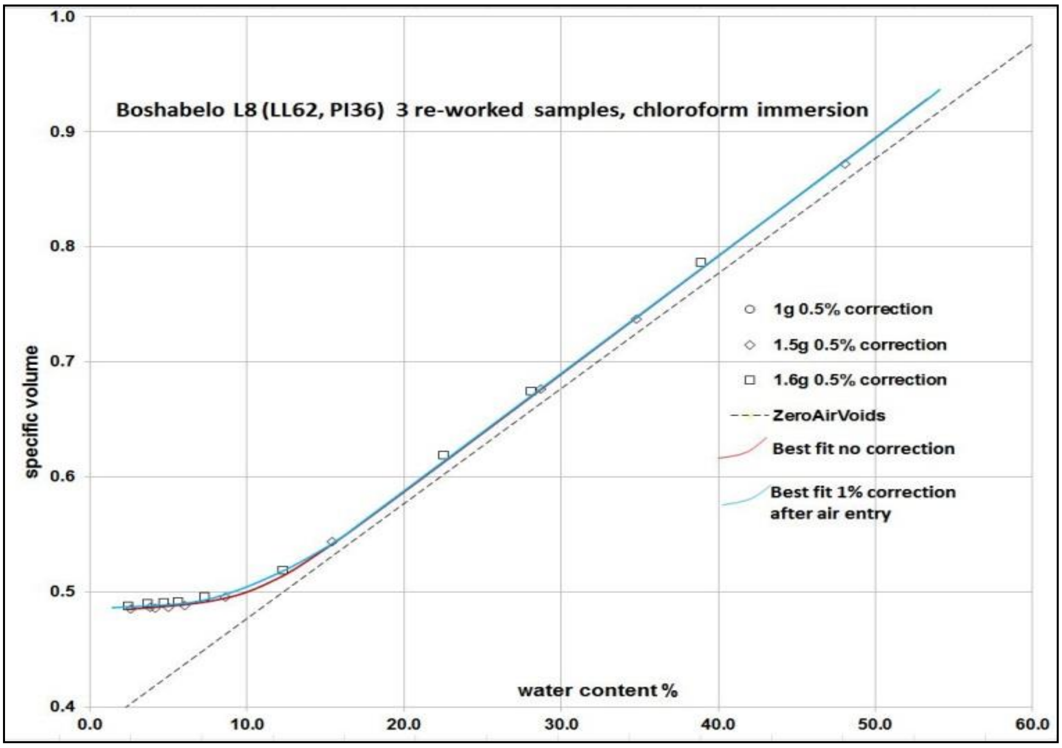


Figure 12.10 (c): Superimposed curves with data points corrected.

Figure 12.10 (c) shows the “uncorrected” and the “1% corrected” curve together with the data points for 0.5% correction. It appears that the scatter of the data is of the same order of magnitude as the divergence of the curves. It appears that a very good approximation to the shrinkage curve can be made using any value less than 1%. Figure 12.11 shows the shrinkage curve for an active clay obtained using six undisturbed samples and 3 remoulded samples of sizes ranging from 2g to 10g. Immersion was in benzene for three undisturbed samples and two disturbed samples, in chloroform for the remainder. Air drying took approximately 8 hours at approximately 24°C and 35% relative humidity.

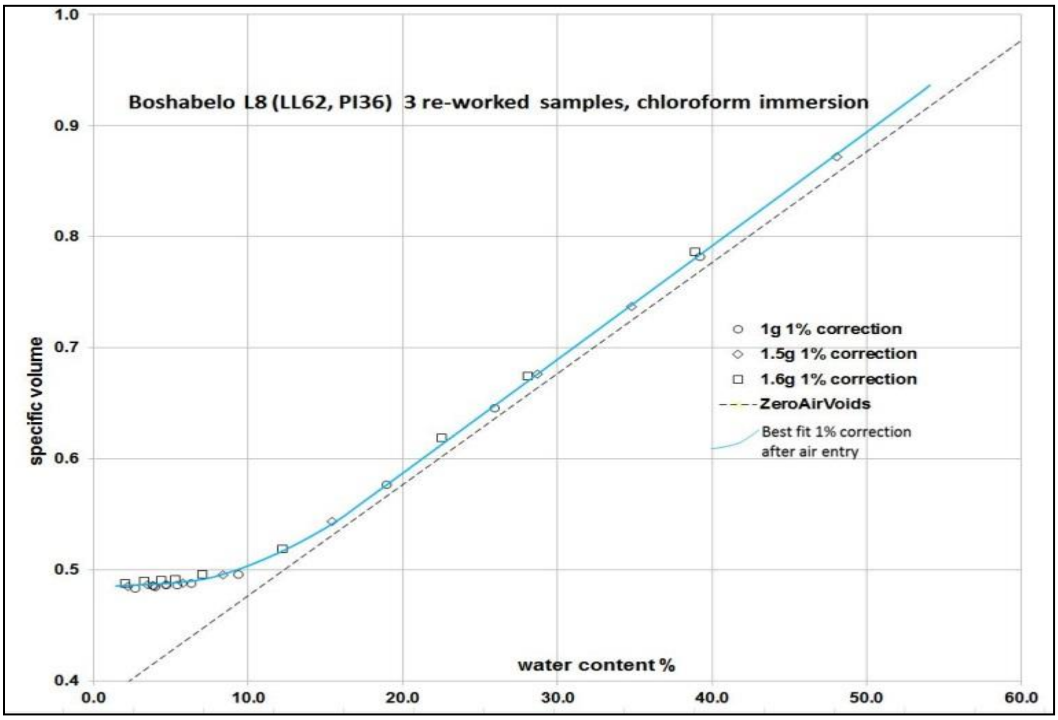


Figure 12.10: Composite shrinkage curve.

12.7 Conclusion

Shrinkage curves can be produced quickly and easily for small samples using volatile non-polar fluids. Chloroform appears to be an ideal liquid for this purpose, requiring little time for removal from samples between mass and volume determinations and requiring little correction for retention by desiccated samples. Drying at slightly elevated temperature in an incubator can reduce the total time required for a full shrinkage curve to below 8 hours even for highly active clays.

CHAPTER 13: CONCLUSIONS

This research project has taken a very different direction from that originally intended. The original intention entailed checking the reliability of standard soil mechanics tests and finding ways to improve reliability of inputs to existing heave prediction methods (particularly the Atterberg limits). Intentions also involved finding quicker ways to generate unsaturated soil mechanics parameters like the shrinkage curve and the soil water characteristic curve, as well as more reliable input parameters for finite element analysis of foundations.

The project uncovered a major set-back for this approach, in finding that no reliable procedure seems to be able to accurately assess clay fraction. The most popular methods of assessing heave-potential rely on clay fraction as a critical input. It appears that cases of large-scale and costly failure can be attributed to large and apparently not easily recognisable errors in clay fraction determination by the most trusted and long-established methods, performed in well-established, reputable laboratories.

13.1 The Potential Value of the Research Presented in this Thesis.

But the most significant aspect of the research presented is the demonstration that variability of heave potential may be a specific property of individual clayey soils, and that this variability can be very significant. This fact throws doubt on the effectiveness of any design method which does not take variability into account. For many years engineering pioneers like Phoon (2009) have been pointing out the value of considering variability in inputs for all aspects of geotechnical design. Reliability Based design appears to be a very powerful way to incorporate variability, but relatively little attention has been paid by most practicing engineers. Perhaps one of the reasons for this has been the ad-hoc estimates of COV to be employed. For PI of clay, for example, the recommendation of assuming 3% to 12% of the PI for the COV (Phoon and Ching, 2013) appears to be too much like a guess to inspire confidence. The value Phoon and Ching (2013) suggest is based on considerations of operator error in testing, variations between clay types etc. This thesis has presented cases of measured values of COV for particular clays well above 20%. Failure to recognise this variability for specific soils has led to costly damage. It would probably have led to damage even if Reliability Based design had been applied if the proposed ad hoc estimates had been used instead of measured values. With the simple and inexpensive method presented in this thesis it is possible to obtain a good estimate of mean and COV for expansive potential in one test which requires relatively little cost in both equipment and skilled labour content.

The discovery of specific variability in properties of the expansive soils concerned in this project has brought the realisation that all attempts to solve the problem of expansive clays must be viewed in a different light. Any attempt to assess heave, or to find practical procedures to deal with it, must take specific variability of the soil concerned into account. No currently popular method of heave prediction does so.

13.2 Outlook for Future Studies

Several lines of investigation were being undertaken by the candidate when the problem of variability was revealed. One of these possibilities was that of using the fall cone for Liquid Limit determination. The fall cone has been adopted in many countries, but not in South Africa. This is somewhat surprising in the light of widespread dissatisfaction with the current method which uses the Casagrande cup. Several prominent South African Soil Mechanics practitioners, including Derek Sparks and Geoffrey Blight suggested in personal communications that the fall cone test is superior to the Casagrande cup and should be investigated. It appears that a major reason for the fall cone being rejected for use in South Africa is an assessment by Sampson and Netterberg (1984) in which they found several reasons for not adopting the fall cone. One of these reasons was the difficulty of finding consistent correlation between values of LL given by the fall cone with those given by the traditional method. The finding of a substantial Coefficient of Variation (COV) for many clayey soils suggests possible reasons for such lack of correlation. In fact, the CUT research group had been attempting to find correlations between several different heave indicators and had found difficulty identifying any such correlation before the influence of COV was appreciated. Comparing single values from different methods can only be valid for soils with very low COV. It would, for example, be unhelpful to compare isolated values of PI taken at random from the upper part of Figure 7.21 (shown below as Figure 13.1) against isolated values of water retention taken at random from the lower part of the figure. Meaningful comparison must deal with sufficient samples to enable comparison of mean and COV.

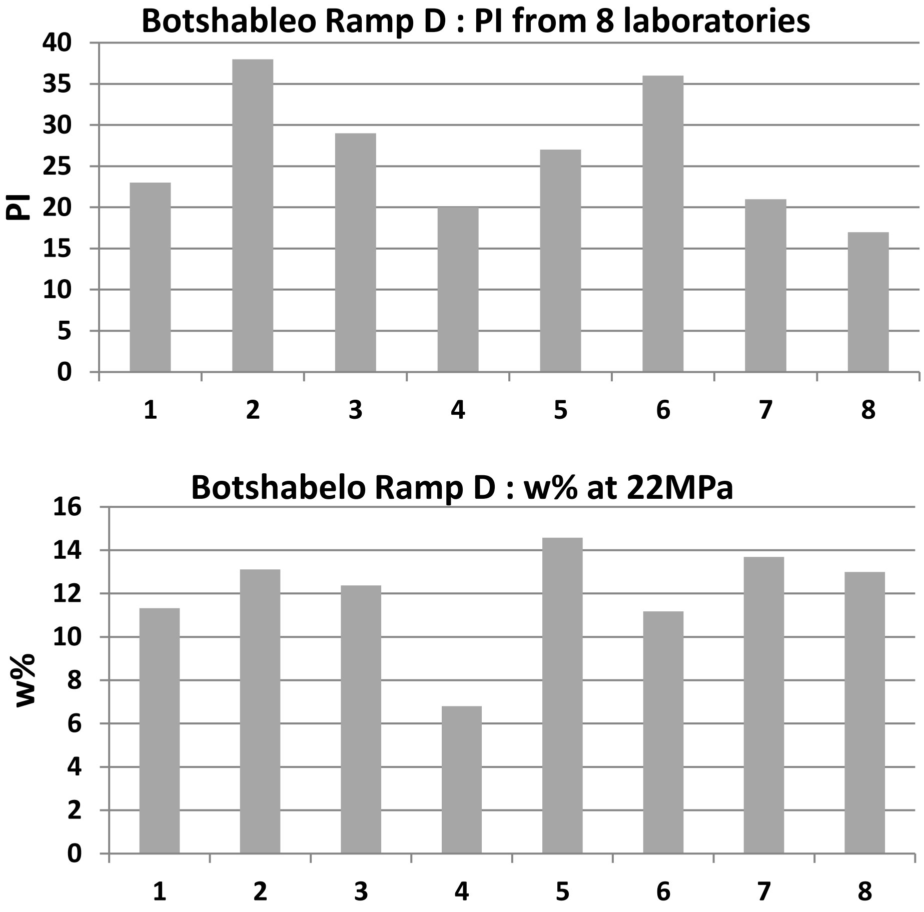


Figure 13.1: PI and water retention values.

Figure 13.1 shows PI and water retention values for a clayey soil from a Central South African road project. PI from 8 laboratories (top) and water retention from 8 suction tests (bottom).

The investigations at the CUT Soil Mechanics Research Group have now changed focus to take account of variability in the soils involved in all of its investigations.

All tests, including the fall cone investigation, are now being done with the understanding that large enough numbers of tests on different samples of the same soil must be conducted to enable comparisons of mean and COV rather than only isolated values. As a consequence, some of the possibilities being examined by the candidate as part of this investigation are being taken over by students as individual research projects.

Apparatus has been developed for several promising ways of assessing aspects of expansive clay behaviour. Most of the apparatus developed for these investigations is made from Perspex or wood, since they can be cut and assembled to fair accuracy quickly and cheaply. This has proved more successful than more glamorous procedures, such as 3D printing, which was found to be expensive, slow, and difficult to modify. When designs have been brought to the stage where useful results can be obtained then it is worthwhile to make working prototypes in stronger material.

One of these avenues of investigation is a rapid way to assess heave against pressures of the magnitude probable under the foundations of light structures. This method was used for assessing the potential movement under the government subsidy house of section 11.4.2 in Chapter 11. A narrow sample of soil can be dosed with water in its central plane. There is only a short path for water to penetrate the clay structure and swelling response is very quick. The surface of the sample lifts a cover plate as the soil expands and the movement is monitored. This apparatus enables some questions to be studied in far less time than, for example, by the oedometer. The kind of output to be produced can be seen in Figure 13.2. This figure shows the expansion of a clayey sample against pressures of 7 kPa and 11 kPa from initial water content 19% after addition of sufficient water to bring the final water content to 25%, and from the same sample after further addition of water sufficient to bring the final water content to 30% (Note: apparently not all could be absorbed).

From the curves it can be deduced that this soil can probably lift a government subsidy house (where pressure under a typical raft foundation is about 10 kPa) from typical natural moisture content for highly expansive clay (19%) to moisture content 25%. It appears that it should be able to lift the house by 8% of the thickness of the clay layer. Of the water made available to the clay when at 25% water content, it would appear that it could only draw in about one third of the water supplied, say 2% and could heave only another 2.4% of the thickness of the clay layer. One might therefore expect a maximum uplift from natural moisture content of about 10% of the thickness of the clay layer. The total testing time needed to be able to make these deductions is about 12 hours.

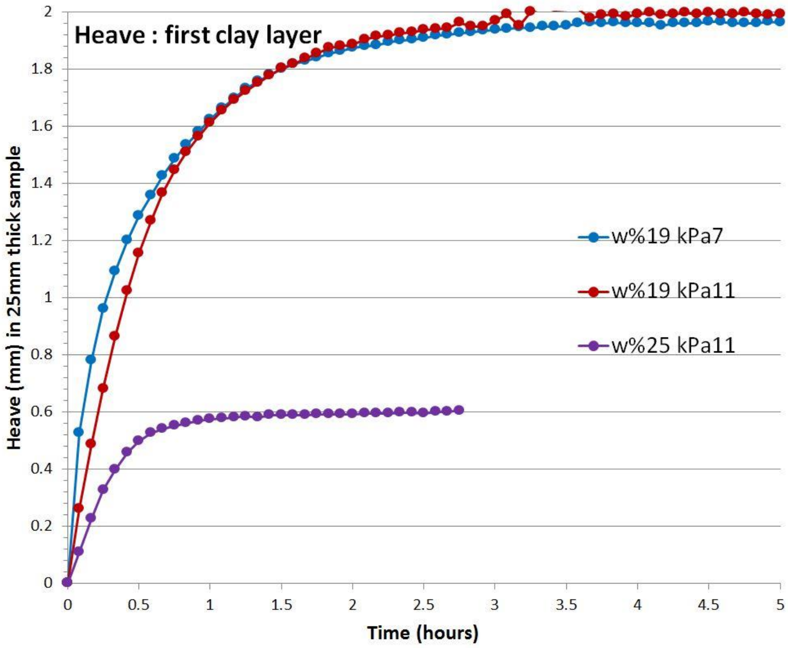


Figure 13.2: Expansion of a clayey sample.

Another set of readings from the apparatus is shown in Figure 13.3, where the general heave potential of a sample is rapidly assessed and full testing is only continued when the sample is deemed to be close to the limits of its heave potential. This can save considerable time for assessing powerfully expansive clay.

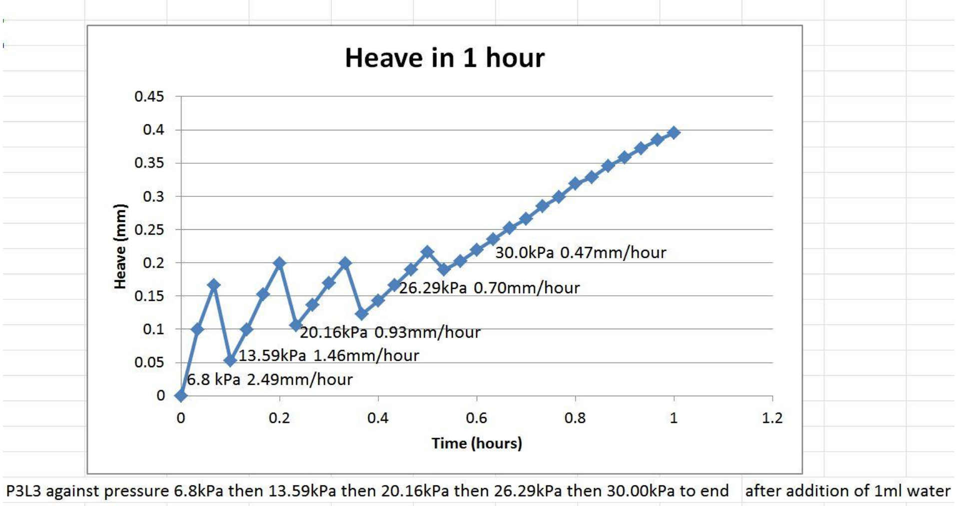
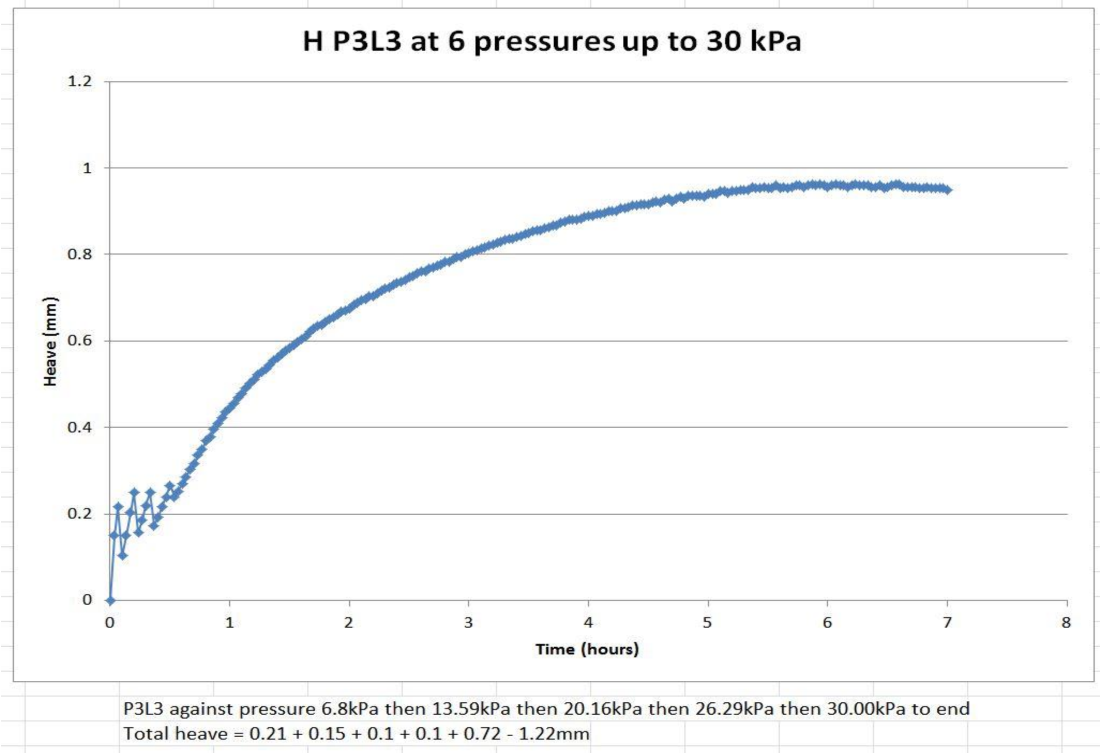


Figure 13.3: Heave progress of a clayey soil subjected to incremental loading.

Initially 6.8 kPa was applied. After 6 minutes it was seen that the sample was heaving at the rapid rate of 2.49mm per hour and was probably capable of expanding against a much higher pressure. Load was increased to 13.59 kPa and after another 6 minutes the expansion rate was seen to be 1.46mm per hour. Further increments were made until at 30 kPa the expansion rate had fallen to 0.47mm per hour, indicating that the soil was probably close to its full heave potential and expansion was allowed to continue at this load until all of the water supplied had been absorbed, as shown in Figure 13.4.

It can be seen that it took 6 hours to reach the maximum expansion possible with the amount of water added (1ml). The disadvantage of making apparatus out of Perspex is illustrated in these graphs. Each addition of load causes significant compression of the thin Perspex structure and leads to a step down in the curve at each load increment. The full heave therefore has to sum the individual heave contributions at each loading step, showing a total heave against 30 kPa pressure of 1.22mm i.e. 4.9% of the clay layer thickness. 30 kPa corresponds to the pressure under the raft foundation of a typical two storey building.



P3L3 against pressure 6.8kPa then 13.59kPa then 20.16kPa then 26.29kPa then 30.00kPa to end
Total heave = 0.21 + 0.15 + 0.1 + 0.1 + 0.72 - 1.22mm

Figure 13.4: Continuation of the heave curve.

Having reached the stage of providing interesting results the apparatus has now been manufactured from stainless steel and has been made available for a student to conduct further research. Any testing programs for this apparatus will take into account the fact that variability is a vital factor to be considered and will allow broadening the scope of investigation into possible fractal distribution features of expansive clays.

Investigation of this type of heave indication method seems to hold out far more hope of success than any procedure which relies on clay fraction determination - until a reliable method of particle size determination can be devised. Possible methods of PSD are being investigated at the present time, but success is not likely to be achieved in the very near future.

It appears that a wide range of assessment is needed to examine the full ramifications of the issues raised in this thesis. It is hoped that this will lead to a significant change in Fredlund’s assessment that volume change of clays is a field “*where if you get involved you usually lose money; it is not a good problem”.*

CHAPTER 14: REFERENCES

Abbireddy, CO R.2008. Clayton, C.R.I., and Huvenne, V.A.I. (2009). A method of estimating the form of fine particulates. *Geotechnique* 59, No. 6, 503–511 [doi: 10.1680/geot.2008. P.009]

Abbireddy, COR. and Clayton CRI.2009. A review of modern particle sizing methods, *Geotechnical Engineering 162*, August 2009 Issue GE4 pp193-201. Doi: 10.1680/geng 2009.162, 4.193

AMERICAN SOCIETY FOR TESTING AND MATERIALS .2002. Standard Test Method for Particle-Size Analysis of Soils. ASTM International, West Conshohocken, PA, 2002, ASTM D422-63.

Al-Lami, HS.2008. Evaluation of Some Water-Soluble Dispersants for Different Types of Kaolin Powders. *AL-TAQANI,* Volume: 21, 2008, pp 95-100. Available at Iraqi Academic Scientific Journals: www.iasj.net.

Balkema.1987. The results of a large scale field experiment aimed at studying the interaction of raft foundations and expansive soils. *International Conference on soil structure Interactions. Paris. May 1987.* Paris: ENPC Press.

Bah, AR; Kravchyuk, O and Kirchof, G. 2009. Fitting performance of particle-size distribution models on data derived by conventional and laser diffraction techniques. *Soil Science Society of America Journal,* 73,1101- 107. doi:10.2136/sssaj2007.0433

Badenhorst, W; Theron, E and Stott, P. 2015. Duplicate testing conducted on the input parameters for the estimation of potential expansiveness of clay. *Proceedings of the 16th African Regional Conference on Soil Mechanics and Geotechnical Engineering. Tunisia 27-30 April 2015.* Tunis: ATMS.

Bensallam, S; Bahi, L; Ejjaaouani, H and Shakhirev, V. 2012. A New Shrinkage Curve Model, Applied to Moroccan Clayey Soil. *International Journal of Geosciences*. July 2012 (http://www.SciRP.org/journal/ijg)

Bester, DM; Stott, PR and Theron, E.2016. “The movement of soil moisture under a government subsidy house”. Proceedings of the first Southern African Geotechnical Conference – Jacobsz (Ed). Taylor and Francis Group, London.

Blight, GE. 2012. Microstructure, mineralogy and classification of residual soils.*Mechanics of Residual Soils*. Blight GE. and Leong. EC (Eds.), CRC Press, London, New York, Leiden. 2012, pp. 50-54.

Blight, GE. 2012. Microstructure, mineralogy and classification of residual soils. In Blight GE. & Leong EC. (Eds), *Mechanics of Residual Soils.* London, New York, Leiden: CRC Press, Taylor and Francis.

Blight, GE. 2013. Unsaturated Soil Mechanics in Geotechnical Practice. CRC Press Boca Raton, London, New York, Leiden, 2013. p.48

Blight, GE. 2013. Unsaturated Soil Mechanics in Geotechnical Practice. Boca Raton, London, New York, Leiden: CRC Press, Taylor and Francis.

Borden, D and Giese R F.2001. Baseline studies of the clay minerals society source clays: cation exchange capacity measurements by the Ammonia-electrode method. *Clays and Clay Minerals,* Vol. 49, No. 5, 444–445.

Brasher, BR; Franzmeier, DP; Valassis, V and Davidson, SE.1966. “Use of saran resin to coat natural soil clods for bulk density and water retention measurements,” *Soil Science* 1966. 101,108.

BRITISH STANDARDS INSTITUTION.1990. Methods of Test for Soils for Civil Engineering Purposes. Classification Tests. BSI, Milton Keynes, 1990, BS 1377: Part 2.

BSI 1990. Methods of testing soils for civil engineering purposes, BS 1377. Milton Keynes: British Standards Institution.

Buhmann C and Kirsten WFA.1991. The mineralogy of five weathering profiles, developed from Archaean granite in the eastern Transvaal, Republic of South Africa, *South African Journal of Plant and Soil,* *:3, 126-152, DOI: 10.1080/02571862.1991.10634823.

Bulut, R; Lytton RL and Wray, WK. 2001. “Suction Measurements by Filter Paper,” *Expansive Clay Soils and Vegetative Influence on Shallow Foundations*, ASCE Geotechnical Special Publications No 115 (eds. C. Vipulanandan, M.B. Addison, and M Hasen), ASCE, Reston, Viginia, pp 243-261.

Cerato, AB and Lutenegger AJ, 2006, Shrinkage of Clays, *Proceedings of the Fourth International Conference on Unsaturated Soils,* Carefree Arizona, 2006, pp 1097-1108.

Chertkov, VY.2003. “Modelling the shrinkage curve of soil clay pastes.” *Geoderma* 112(2003) p.72

Chiappone, A; Marello, S; Scavia, C and Setti, M.2004. Clay mineral characterization through the methylene blue test: comparison with other experimental techniques and application of the method. Canadian Geotechnical Journal Dec. 2004; 41, 6; ProQuest Central pp 1168-1178

Clayton, CRI; Abbireddy, COR; and Schiebel, R. 2009. “A method of estimating the form of coarse particulates”. *Geotechnique* 59, No.5, 493-501 [doi:10.1680/geor.2007.00195]

Cornelis, WM; Corluy, J; Medina, H; Diaz, J; Hartmann, R; Van Meirvenne, M and Ruiz ME. 2006. “Measuring and modelling the soil shrinkage characteristic curve,” *Geoderma* 137 179-191. 2006.

CSIR. 1986. Technical Methods for Highways, TMH1, Standard Methods of Testing Road Construction Materials, CSIR. Pretoria. 1986

Das, BM. 2006. Principles of Geotechnical Engineering. Toronto: Nelson

Das, BM. 2008. Advanced Soil Mechanics, 3rd Edition, Taylor and Francis, New York, 2008.

Day, PW.1991. Design of Raft foundations for houses on expansive clay using Lytton’s method. *Course on design of foundation to suit various soil conditions. SAICE Structural Division 1991.*

De Boer, GB; de Weerd C; Thoenes, D and Goossens, HW. 1987. Laser diffraction spectrometry: Fraunhofer versus Mie scattering. *Particle Characterization Journal* 4: 14-19.

De Bruijn, CMA.1974. Moisture redistribution in Southern African soils. *Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering.* Moscow. Vol2.2.1973, pp 37-44. (NBRI Reprint No R/ Bou 442, 1974).

Di Stefano, C; Ferro, V; and Mirabile, S. 2010. Comparison between grain-size analyses using laser diffraction and sedimentation methods. *Biosystems Engineeering 106 205-215*. Available online at sciencedirect.com

Dur, JC; Andrades, M; Houot S; Tessier, D and Barriuso, E. 2006. Size Characterization by Laser Granulometry of Colloids Extracted from Compost at Different Temperatures. *Compost Science and Utilization*, 2006, Vol 14, No4, 276-283.

Eshel, G; Levy GJ; Mingelgrin, U and Singer, MJ. 2004. “Critical Evaluation of the Use of Laser Diffraction for Particle-Size Analysis”. *Soil Science Society of America Journal*; May/Jun 2004,68.3: pp.736-743,

Franco, F; Perez-Maqueda, LA and Perez-Rodriguez, JL. 2004. “The effect of ultrasound on the particle size and structural disorder of a well-ordered kaolinite”. *Journal of Colloid and Interface Science* 274 (2004) 107-117. Available online at sciencedirect.com

Fityus, SG; Smith, DW and Allman, MA. 2004. Expansive soil test site near New astle. *Journal of Geotechnical and Geoenvironmental Engineering.* ASCE. July 2004 pp. 686 – 695.

Fredlund, DG and Rehardjo, M.1993. Soil Mechanics for Unsaturated Soils. New York: John Wiley and Sons

Fredlund, DG.1999. Emergemce of Unsaturated Soil Mechanics. NRC Research Press 1999.

Fredlund, DG; Rahardjo, H and Fredlund, MD. 2000. Unsaturated Soil Mechanics in Engineering Practice. John Wiley and Sons. p. 189.

Fredlund, DG.2006. “Unsaturated soil mechanics in engineering practice,” *Journal of Geotechnical and Geoenvironmental Engineering* March 2006
pp 286-320.

Fredlund, DG. 2009. Paradigm Shifts to Facilitate the Practice of Unsaturated Soil Mechanics. *8th C.W. Lovel Lecture.*PerdueGeotechnical Engineering. 26 October 2009.

Fredlund, DG; Rehardjo, M and Fredlund, MD. 2012. Unsaturated Soil Mechanics in Engineering Practice. Heboken NJ: John Wiley and Sons.

Geotechnical Engineering Bureau. 2007. Geotechnical Test Method GTM7 Revision #1. New York State Department of Transportation. New York. April 2007.

Goossens, D. 2008. Techniques to measure grain-size distributions of loamy sediments: a comparative study of ten instruments for wet analysis. *Sedimentology,* 55, 65e96.

Gourley, CS and Schreiner, HD. 1995. Field measurement of soil suction. First International Conference on Unsaturated Soils, UNST'95, Paris, 6-8 September 1995.

Government Communication and Information System. 2015. Housing delivery in South Africa - How have we fared? *Government Communication and Information System* (http://www.sanews.gov.za/south-africa/housing-delivery-sa-how-have-we-fared).

Hu, F; Li, H; Liu, X; Li, S; Ding, W; Xu C; Yue, L and Longhui, Z. 2015. "Quantitative Characterization of Non-Classic Polarization of Cations on Clay Aggregate Stability". *PLoS ONE* 10(4): e0122460. Doi:10.1371 /journal. pone.0122460

Indian Standard IS: 2720 (Part 5) .1985. Reaffirmed 1995 *Determination of Liquid and Plastic Limit*, Indian Standards Institution, New Delhi, August 1985.

Jacobsz, S.W. & Day, P.W. 2008. Are we getting what we pay for from geotechnical laboratories? *Civil Engineering,* 16(4): 8–11.

Jacobsz, SW. 2013. Site Investigation on Dry Clayey Soils, *SAICE Geotechnical Division Course on Site Investigation,19 March 2013*

Jaksa, MB.1995. The influence of spatial variability on the geotechnical design properties of a stiff, over consolidated clay. *Ph.D. Thesis,* University of Adelaide.

Jennings, JEB; Knight, K.1957. The prediction of total heave from the double oedometer test. *Transactions South African Institution of Civil Engineering,* 7*, pp 285-291.*

Jordaan, A. 2014. Easy methods for determination of swell potential for South African clays. M.Sc. Thesis Faculty of Civil Engineering, University of Stellenbosch. April 2014.

Kassa, M.2005. Relationship between Consolidation and Swelling Characteristics of Expansive Soils of Addis Ababa. M.Sc. Thesis Addis Ababa University 2005.

Keller, JM and Gee, GW. 2006. Comparison of ASTM and SSA Methods for Particle-Size Analysis. *Soil Science Society of America Journal;* Jul/Aug 2006; 70, 4; ProQuest Centralpp. 1094-1100.

Kitcher, JSD.1980.*Brief notes on estimates of the cost of structural repairs to houses built on active clay in South Africa between 1980 and 2000*. Proceedings of the South African Geotechnical Conference, 11-13 November 1980, Silverton.

Kleyn, E; Bergh, A and Botha, P. 2009. *Practical Implications of the relation between the clay mineral content and the Plasticity Index of dolerite road construction material.* Journal of the South African Institution of Civil Engineers. Vol 51. No.1. April 2009. pp 2-5

Knappett, JA and Craig, RF. 2012. Craig's Soil Mechanics. Abingdon Oxon: Spon Press, Taylor and Francis

Konert, M and Vandenberghe, J.1997. Comparison of laser grain size analysis with pipette and sieve analysis: a solution for the underestimation of the clay fraction. *Sedimentology.* Vol 44 3 June 1997 pp. 523-535

Krosley, L; Likos, WJ and Lu, N.2003. “Alternative Encasement Materials for Clod Test,” *Geotechnical Testing Journal*, Vol. 26 No.4.

Kubota, T. 1972. “Aggregate-formation of allophanic soils: Effects of drying on the dispersion of the soils”. *Soil Science and Plant Nutrients* 18: pp 79-87.

Lawson, WD.2006. A Survey of Geotechnical Practice for Expansive Soils in Texas.*4th International Conference on Unsaturated Soils,* Carefree, Arizona, April 2006. pp 304-314.

Ma, Z; Merkus, HG; de Smet, JGAE; Heffels, C and Scarlett ,B. 2000. New Developments in particle characterization by laser diffraction: size and shape. *Powder Technology.* Vol 111, Issues 1-2 August 2000 pp.66-78. Elsevier.

Mavroulidou, M; Zhang, X; Kichou, Z and Gunn, MJ. 2013. Hydromechanical properties of lime-treated London Clay. *Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering,* Paris 2013. pp. 1151-1154.

Macek, M; Smolar, J and Petkvsek, A.2013. “Extension of measurement range of dew-point potentiometer and evaporation method”. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, pp. 1137-1142.

McCave, IN; Bryant RJ; Cook, HF and Coughanowr, CA.1986.Evaluation of a laser diffraction-size analyser for use with natural sediments. *Journal of Sedimentary Petrology* Vol. 56 No.4 July 1986 pp. 561-564.

Mendes, J and Toll, DG. 2013. Influence of initial water content on the water retention behaviour of a sandy clay soil. *Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering,* Paris 2013. pp. 1137-1142.

Mikutta, R; Kleber, M; Kaiser, K and Jahn, R. 2005. Review: Organic Matter Removal from Soils using Hydrogen Peroxide, Sodium Hypochlorite, and Disodium Peroxodisulfate. *Soil Science Society of America Journal.* January 2005. DOI: 10.2136/sssaj2005.0120

Miller, DJ; Durkee, DB; Chao, KC and Nelson, JD.1995. Simplified heave prediction for expansive soils. *Unsaturated soils/sols Non Satures, Alonso & Delage (eds).* Rotterdam:

Mishra, AK; Ridlo, MO and Higashi, T.2011. Comparison of the Clay Fraction of Ariake Clay Determined by Japanese and ASTM Standards. *Marine Georesources and Geotechnology,* Taylor and Francis Group, LLC ISSN: 1064-119X print/1521-0618 online DOI: 10.1080/1064119X.2012.525138. 29:110-116, 2011

Minty, EJ; Smith, RB and Pratt, DN.1979. Inter-laboratory testing variability assessed for a wide range of N.S.W. soil types. Proceedings of the 3rd International Conference on Applications of Statistics and Probability in Soil and Structural Engineering, Sydney, Vol. 1, pp. 221–235.

Mineralogy. *Journal of the Brasilian Society of Soil Science. Vol 35. No.5. 2011, pp.1589-1596, 2011 Brasil. Available online at http://www.redalyc.org.*

Monye, PK; Stott, PR and Theron, E. Assessment of reliability of the hydrometer by examination of sediment. *1St Southern African Conference on Geotechnical Engineering.* Sun City May 2016.

Nettleship, I; Cisko, L and Vallejo, LE. 1997. *Aggregation of clay in the hydrometer test*. Canadian Geotechnical Journal 34:621-626 (1997)

Nishimura, T. 2013. “Application of micro-porous membrane technology for measurement of soil-water characteristic curve.” Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, pp. 1171-1174.

Nelson, JD; Miller, DJ.1992. Expansive Soils – Problems and Practice in Foundation and Pavement Engineering. John Wiley and Sons, New York, Chicester, Brisbane, Toronto, Singapore, 1992.

Nyak, NV and Christiensen, RW. 1974. Swelling characteristics of compacted expansive soils. Clays and clay minerals, 19(4) pp 251-261.

Paige Green, P.2004. *The Geology and Engineering Geology of Roads In South Africa* Proceedings of the 23rd Southern African Transport Conference (SATC 2004) Pretoria, July 2004

Paige Green, P and Ventura, D.1999.The bar linear shrinkage test – More useful than we think!*12th Reginal Conference for Africa on Soil Mechanics and Geotechnical Engineering.* Durban. 1999. pp 379-387.

Peng, X Horn, R. 2005. “Modeling soil shrinkage curves across a wide range of soil types,” *Soil Science Society of America Jourrnal,* Soil Science Society of America, April 11 2005 Online

Phoon, KK and Kulhawy, FH. 1999. Evaluation of geotechnical property variability. *Canadian Geotechnical Journal*, 36: 625–639.

Phoon, KK and Kulhawy, FH. 1999a. “Characterization of geotechnical variability.” Canadian Geotechnical Journal, 36: 612–624.

Phoon, KK and Kulhawy, FH. 1999b. “Evaluation of geotechnical property variability.” Canadian Geotechnical Journal, 36: 625–639.

Phoon, KK. 2008. Reliability-based design in geotechnical engineering. Taylor and Francis Abingdon and New York. 2008.

Phoon, KK. 2008. Numerical recipes for reliability analysis – a primer.In KK ,Phoon (Ed). *Reliability-Based Design in Geotechnical Engineering.* Oxford: Taylor and Francis.

Phoon, KK and Ching, J. 2013. “Beyond coefficient of variation for statistical characteriszation of geotechnical parameters”. Geotechnical and Geophysical Site Characterization 4 – Couinho and Mayne (eds). Taylor and Francis Group London.

Pidgeon, TJ.1980. A Comparison of existing methods for the design of stiffened raft foundations on expansive soils. *Seventh Regional Conference for Africa on Soil Mechanics and Foundation Engineering,* Accra, June 1980. pp. 277-289.

Pidgeon, JT. 1987. The results of a large scale field experiment aimed at studying the interaction of raft foundations and expansive soils. *International Conference on soil structure Interactions. Paris. May 1987.* Paris: ENPC Press.

Pidgeon, JT and Pellissier, JP. 1987. The behaviour of an L-shaped raft subjected to non-uniform support conditions. *International Conference on Soil Structure Interactions. Paris. May 1987.* Paris: ENPC Press.

Reis, RM; Soboya, F; Tibana, S; Mardiano, CR; Ribeiro, AB; Sterck, WN; Avanzi, E.D. (2013) “Determination of soil-water retention curve for a young residual soil using a small centrifuge”. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, pp. 1175-1178.

Rodrigues, C; de Oliveira, V A; da Silveira, PM and Santos, GG. 2011. Chemical Dispersants and Pre-Treatments to Determine Clay in Soils with Different Mineralogy. *Journal of the Brasilian Society of Soil Science.* Vol 35. No.5. 2011, pp.1589-1596, 2011 Brasil. Available online at http://www.redalyc.org.

Rolfe, BN; Miller, RF and McQueen, IS. 1960. Dispersion Characteristics of Montmorillonite, Kaolinite, and Illite Clays in Waters of Varying Quality, and Their Control with Phosphate Dispersants*. Geological Survey Professional*

Paper 334-G. U.S. Government Printing Office in cooperation with Colorado State University, Washington, 1960.

Robinet. JC; Coelho, D and Altmann S. 2011. State of the art on cation diffusion in clay mineral systems. *European Commission Community Research. Cat Clay Report. 1/5/2011*

Rogers, JD; Olshansky, R and Rogers, RB.2013, Damage to Foundations from Expansive Soil. *Missouri University of Science and Technology web site*: web.mst.edu/ accessed 2013 10 04.

SABS .2011. South African National Standard SANS 3001:2011 Edition 1.1*. Civil Engineering Test Methods. Part GR3,* South African Bureau of Standards. Pretoria. 2011.

SABS. 2011*. Civil Engineering Test Methods. Part GR3*. *South African National Standard*

SABS. 2011. South African National Standard SANS 3001:2011 Edition 1.1*. Civil Engineering Test Methods. Parts GR1, GR2, GR3, GR5, GR10, GR11, GR12*; South African Bureau of Standards. Pretoria. 2011.

Sampson, LR and Netterberg, F.1984. A cone penetration method for measuring the liquid limits. of South African soils. *Proceedings of the Eighth Regional Conference for Africa on Soil Mechanics and Foundation Engineering.* Harare, 1984. Vol 1 pp 105-114.

SANS 3001.2011. Edition 1.1. Pretoria: South African Bureau of Standards

Savage, PF. 2007. *Evaluation of possible swelling potential of soil.* Proceedings of the 26th South African Transport Conference (SATC 2007), 9-12 July 2007, DTT, Pretoria.

Schreiner, D and Habte, T. 2008. A universal method of assessing the intrinsic expansiveness of clays. SAICE Conference: Problem Soils in South Africa. ESCOM conference centre, Midrand, South Africa, 3-4 November 2008.

Singh, A. and Lee, K.L. (1970) “Variability in soil parameters,” Proceedings of the 8th Annual Engineering Geology and Soils Engineering Symposium, Idaho, pp. 159–185

Skempton, AW.1953. The Colloidal “Activity” of Clays. *Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering*, Zurich 1953.

Sparks, ADW. 2010 a. Using Atterberg Limits to predict the expansion of clays. *Civil Engineering*. April 2010

Sparks, ADW. 2010. Modelling of reconstituted clays for city planning. In V.P.Petrushkin, V.M. Ulitsky, I.V. Kolybin, M.B. Lisyuk and M.L. Kholmyansky (eds), Proceedings of the International Geotechnical Conference – Geotechnical Challenges in Magacities, June 2010. pp7-10

Sparks, ADW and Pidgeon, JT. 2011. Simplifying expansion of clays. *Civil Engineering* April 2011.

Sridharan, A and Prakash, K.2000. *Classification procedures for expansive soils*. Proc. Inst. Civil Engineers Geotechnical Engineering, 143, October 2000. pp 235-240

Stott, PR and Theron, E. 2015a. Some shortcomings in the standard South African testing procedures for assessing heaving clay. *Journal of the South African Institution of Civil Engineering. Vol 57 No 2, June 2015, pp. 36–44.*

Stott, PR and Theron, E. 2015b. Assessment of clays by small-scale suction tests. *Proceedings of the 16th African Regional Conference on Soil Mechanics and Geotechnical Engineering. Tunisia 27-30 April 2015.* Tunis: ATMS.

Stott, PR and Theron, E. 2016. Shortcomings in the estimation of clay fraction by Hydrometer*. Journal of the South African Institution of Civil Engineering (in print).*

Stott, PR; Monye, PK and Theron, E. 2016. “Assessment of reliability of the hydrometer by examination of sediment”. *Proceedings of the 1st Southern African Conference on Geotechnical Engineering.* Sun City 5-6 May2016. Rotterdam: Balkema.

Stott, PR and Theron E. 2016. Shortcomings in the estimation of clay fraction by hydrometer. *Journal of the South African Institute of Civil Engineering.* Vol 58 No 2, June 2016 pp.14-24.

Taboada, MA.2003. “Soil Shrinkage Characteristics in Swelling Soils*”. Lecture given at the College on Soil Physics, Trieste*, March 2003

Tariq, A and Durnford, D S.1993. “Soil volumetric shrinkage measurements: a simple method,” *Soil Science* 155, 1993. 325-330.

Turoz, M and Tosun, H. 2011. “The use of methylene blue test for predicting swell parameters of natural clay soils”. *Scientific Research and Essays* Vol. 6(8), pp. 1780-1792, 18 April, 2011. Available online at .org/SRE ISSN 1992-2248 ©2011 Academic Journals

Van der Merwe, DH. 1964. The prediction of heave from the plasticity index and percentage clay fraction in soils. The Civil Engineer In South Africa, June 1964, pp 103-107.

Van der Merwe, DH. 1976. “Plasticity index and the percentage clay fraction of soils” *Proceedings of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering*, Durban, Volume 2, pp 166-167..

Weinert, HH.1980. The natural road construction materials of southern Africa. Pretoria: Academica.

Weston, D. 1980. Expansive roadbed treatment for Southern Africa. Proceedings of the 4th conference on expansive soils, pp 339-360.

White, DJ. 2003. PSD measurement using the single particle optical sizing (SPOS) method. *Geotechnique* 53(3), pp 317-326.

Whitlow, R.2000. Basic Soil Mechanics. *Fourth Edition, first edition published in 1983.* Prentice Hall, Pearson Education.

Williams, A.A.B and Donaldson, G.W. (1980). “Building on expansive soils in South Africa, 1976-1980”, *Proceedings of the 4th International Conference on Expansive Soils,* ASCE Publication, Volume 2.

Zschimmer-Schwarz. 2014. *Mechanisms of action of deflocculants and dispersants in ceramic bodies.* Ceramics. Available at http://www.zschimmer-schwarz.com/en/simon/zschimmer-schwarz/media/site/downloads/fachinfo/eng/Fach/E_WirkmechVerfluess.pdf